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AN ASSESSMENT OF THE ZERO-RISE ORDINANCE IN KING COUNTY

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16. ABSTRACT

Floodplain regulation strives to balance the benefits of reducing flood hazards to human life and property against the costs of limiting encroachment on areas that, under normal conditions, have appeal for many kinds of development. The 1990 enactment of a zero-rise ordinance in King County tightened floodplain regulations enough that agencies responsible for maintaining safe bridges and roadways are now hampered by excessive design and construction costs. Agencies responsible for maintaining transportation safety must now work in a highly restrictive regulatory environment in which bridge replacements and improvements are prohibitively expensive or impossible without variances from one or more regulations.

The general purpose of this project was to evaluate the implications of the zero-rise regulation for bridge builders in the King County Roads Division and the Washington State Department of Transportation. The study found the following:

Economic trade-off of compliance vs. noncompliance: The results of the impact analysis showed that the potential benefits of maintaining the state's infrastructure by replacing, widening, or building new bridges and roads should be considered when development is restricted within the floodplains. The additional costs for compliance with the zero-rise ordinance would be uniformly much greater than the marginal cost of damages due to additional flooding for the bridges studied.

Ability to predict, model, and measure changes: Because of the uncertainties in input parameters, it was not usually possible to predict changes in the water surface elevations to within 0.01 ft. The range of variation for predicted backwater elevations, given realistic levels of uncertainty in inputs, were up to an order of magnitude larger than the 0.01 ft limit. A steady, one-dimensional hydraulic model, which is an economical and commonly used tool for assessing water surface profiles, was not generally accurate enough to delineate the floodplain to within 0.01 feet.

Alternatives to the zero-rise ordinance: An analysis of alternatives to a zero-rise ordinance suggested that, for a limitation law set to a technically enforceable tolerance, exemptions for public structures could be evaluated on the basis of economics.

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Zero Floodplain Rise

AN ASSESSMENT OF THE ZERO-RISE ORDINANCE IN KING COUNTY TECHNICAL REPORT

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EXECUTIVE SUMMARY

In 1990, a floodplain management regulation, entitled Zero Floodplain Rise, was adopted by King County to minimize flooding and reduce flood-related damages by restricting development within the floodplains. This regulation has two major requirements (King County, 1990):

- Development within the floodplain may not increase the water surface elevation of the 100-year storm event by more than 0.01 ft.
- Development within the floodplain may not cause a net reduction in the 100-year floodplain storage capacity.

This report evaluates two issues associated with compliance with a zero-rise ordinance within King County:

- the economic tradeoffs between compliance and non-compliance with the zero-rise ordinance
- the ability of engineers to accurately predict, model and measure changes in the water surface elevation to within 1/100 of a foot

These issues, which concern water surface elevation increases, address only the first part of the zero-rise ordinance adopted by King County. The second portion of the ordinance, which deals with floodplain storage capacity, is not addressed in this report.

Five case studies were conducted to study the feasibility of applying the zero-rise ordinance and its effects on bridge construction and maintenance. These case studies included highway bridge projects located in King County, administered by either King County or the Washington State Department of Transportation (WSDOT). For these case studies, the Hydraulic Engineering Center model HEC – 2 was used to predict the change in water surface elevation caused by the bridge project. A sensitivity analysis was carried out to assess the sensitivity of the model results to the hydraulic input parameters of discharge and Manning roughness coefficient. In addition, for case studies 1 and 2, the sensitivity of the model results to pier location were analyzed. Case studies 1, 2 and 3 were used to estimate the incremental flood damage potential created by each bridge project. The results of case study 3 were used to evaluate some alternative options to the zero-rise ordinance.

The five case studies were:

- Case Study Number 1 - SR-169 Bridge over Cedar River
- Case Study Number 2 - 149^{th} Ave. SE (Elliot) Bridge over Cedar River
- Case Study Number 3 - Meadowbrook Bridge over Snoqualmie River
- Case Study Number 4 - SR-18 Bridge over Green River
- Case Study Number 5 - Preston - Fall City Road Bridge over Raging River

A. Economic Trade-off of Compliance and Non-Compliance

To quantify the impacts associated with non-compliance with the zero-rise ordinance, the cost of additional damages due to flooding was compared with the cost of constructing a single span bridge in compliance with the ordinance. The following conclusions were drawn from the three case studies used in this analysis:

¶ When the cost of not complying with the zero-rise ordinance was assessed as the additional structural and property damages incurred by bridge modifications from a single 100-year flood event, the costs of compliance for Case Studies 1, 2, and 3 exceeded the costs associated with non-compliance.

- For the Cedar River Bridge, Case Study 1, an additional \$1.66 million would be required for compliance with the zero-rise ordinance. The one-time 100-year flood damages associated with constructing a bridge in the floodplain would be approximately \$5,500.
- For the Elliott Bridge, Case Study 2, an additional \$2 to \$4 million would be required to construct a single span bridge. The one-time 100-year flood damages associated with constructing a bridge in the floodplain would be approximately \$33,000.

- For the Meadowbrook Bridge, Case Study 3, it would not be practical to construct a single span bridge across the floodplain because of its great width near the town of Snoqualmie. The maximum damages associated with the 100-year flood event would be approximately $55,000.

¶ When the cost of not complying with the zero-rise ordinance was assessed as the additional structural and property damages incurred by bridge modifications, given expected damage value from 36 years of historical peak flows, the additional annual damage cost for the Meadowbrook Bridge, Case Study 3, was $1250. This represents a 2.3% increase from the annual damage cost without bridge modifications of $54,700.

¶ The relative amount of land damages versus structural damages depended on the particular location of the case study. In Case Studies 1 and 2, the costs associated with land damages would be very small in comparison to structural damages. Maximum land damages would be on the order of hundreds of dollars for the first case study, and there would be no land damages associated with the second case study. In the third case study, on the other hand, the costs associated with land damages would be very large in comparison to structural damages. The maximum land damages would be on the order of hundreds of thousand dollars because the relatively flat floodplain would allow wide overflow to undeveloped areas.

The results of the impact analysis showed that, for all three case studies, the potential benefits of maintaining the state's infrastructure by replacing, widening, or building new bridges and roads should be considered when development is restricted within the floodplains. The additional costs for compliance with the zero-rise ordinance would be uniformly much greater than the marginal cost of damages due to additional flooding.

The results of this preliminary impact analysis were only an assessment of the marginal damages associated with the construction of a bridge in the floodplain. This analysis provided a rough estimate of the tradeoffs associated with the zero-rise ordinance. To complete a more detailed investigation of the tradeoffs of this ordinance, the damage assessment should include the entire reach of river under study.

B. Ability to Predict, Model, and Measure Changes

The Zero Floodplain Rise Ordinance requires a high degree of accuracy in modeling, predicting, and measuring changes in the water surface elevation caused by development in the floodplain. The following conclusions were drawn from the case studies:

¶ Because of the uncertainties in input parameters, it is difficult for engineers to predict changes in the water surface elevations to within 0.01 ft.

- Extremely precise predictions of channel roughness are difficult to obtain because it is costly and time-consuming to verify roughness values with extensive field data and numerical simulations.
- Estimates of discharge in rivers rely on limited time series data for peak flows. As a result, the predicted values for low probability events such as the 100-year peak flow are subject to a significant amount of uncertainty.

¶ The range of variation for predicted backwater elevations, given realistic levels of uncertainty in inputs, can be an order of magnitude larger than the 0.01 ft limit.

- By using a 95 % confidence interval for a range of discharge values for the 100-year peak flow, maximum variations for predicted rise of 0.08 ft., 0.42 ft., 0.04 ft. and 3.05 ft. could be expected for case studies 1, 2, 3, and 4, respectively.
- The maximum water surface variations associated with varying the channel roughness for case studies 1 to 5 were 0.16 ft., 0.08 ft., 0.10 ft., 1.9 ft., and 0.03 ft., respectively.

- The maximum variations in predicted rise using a combination of uncertainty in discharge and in channel roughness were 0.29 ft., 0.67 ft., 0.11 ft., 1.7 ft., and 0.03 ft. for case studies 1 to 5.

¶ An additional point is that, for Case Study 4, the construction of a bridge that improved the flow conveyance would cause local changes in the water surface that could inundate areas that were not previously in the 100-year floodplain. While the wider bridge opening would lower the entire level of the floodplain, the local topography and the existence of levees could change the flooding pattern. Also, it is conceivable that the removal of a floodplain obstruction such as a bridge may cause a rise in the floodplain downstream of the bridge. This was not observed in the five case studies in this report but is nonetheless a possibility.

C. Modeling Limitations

The choice of a numerical model to predict water surface elevations introduces limitations in the accuracy of the predictions. The topography of the river reach and the timing of the flood flow hydrograph must fall well within the assumptions of the numerical model for it to yield reasonable results.

The U.S. Army Corps of Engineers, Hydraulic Engineering Center, the authors of HEC-2, have reported that this model is only accurate to within 0.5 ft. for the types of survey data and the characteristics of rivers typically found in King County (USACOE, 1986). For Case Study 5, in which the Manning coefficients were estimated by engineering judgement, the level of the floodplain was greater than 0.5 feet. The model used for these case studies, HEC-2, is a one-dimensional model for steady, gradually spatially varied flow. Because this type of flow is not fully representative of the flows found in the case studies, HEC-2 can not be expected to model changes in the water surface elevation to 0.01 ft.

As demonstrated in the second and fourth case studies, some river reaches are either not well-modeled with HEC-2 or are extremely sensitive to small changes in input.

These include areas with sharp bends or multiple channels; areas where the bridge design causes flow asymmetry in the channel cross-section, or reaches that have abrupt transitions between leveed and unleveed cross-sections. The use of more complex two-dimensional models, while feasible, requires that more time and expense be devoted to the collection of field data and model calibration. Although input data for one-dimensional models such as HEC-2 are usually available for a site, the data may not be sufficient to construct an accurate two-dimensional model.

D. Alternatives to the Zero Rise Ordinance

The alternative options to the zero-rise ordinance fall into one of three categories:

- limitation based laws
- laws based on economic factors
- broad-scale watershed management laws.

Although a specific evaluation of watershed management was not performed in this study, regulations based on economic analysis were compared to limitation-based regulations for five metrics of performance. The laws based on economic factors rated higher than the limitation based laws for four of the five metrics. The laws based on economic factors proved superior for technical feasibility, cost effectiveness, environmental protection, and maintenance of public safety. Limitation-based laws, however, performed better at reducing the amount of flooding. The analysis suggested that, if a limitation law is set to a technically enforceable tolerance, exemptions based on economics should be made for public structures.

1.0 INTRODUCTION

Because of the high rate of growth experienced in Western Washington state, development in this region has often encroached into floodplains. This development limits the floodplain's capacity to store excess flow and can result in extensive flooding and costly damages. Floodplains are the areas used by the river to accommodate large storm events that cannot be contained within its normal channel banks. Floodplain boundaries are delineated by the Federal Emergency Management Agency (FEMA) and reflect the limits of the predicted 100-year storm event, a storm that has a 1 percent probability of occurring in any given year.

In 1990, a floodplain management regulation, entitled Zero Floodplain Rise, was adopted by King County to minimize flooding and to reduce flood-related damages by restricting development within the floodplains. This regulation has two requirements:

- Development within the floodplain may not increase the water surface elevation of the 100-year storm event by more than 0.01 ft.
- Development within the floodplain may not cause a net reduction in the 100-year floodplain storage capacity.

Before 1990, counties and jurisdictions in the State of Washington adhered to FEMA's floodplain regulations, which establish standards required for participation in the National Flood Insurance Program (NFIP). The zero-rise ordinance varies from NFIP standards in two aspects. First, the NFIP standards allow development within the floodplain to increase the water surface elevation of the 100-year storm event by up to 1 ft. (See Figure 1.1.) Second, the zero-rise regulatory floodway NFIP regulations encompass a larger portion of the floodway. Development is not allowed in the regulatory floodway for either regulation; it is only allowed in the flood fringe. The zero-rise regulatory floodway significantly reduces the definition of the zero-rise flood fringe and thereby imposes more stringent limitations on development in the floodplain.

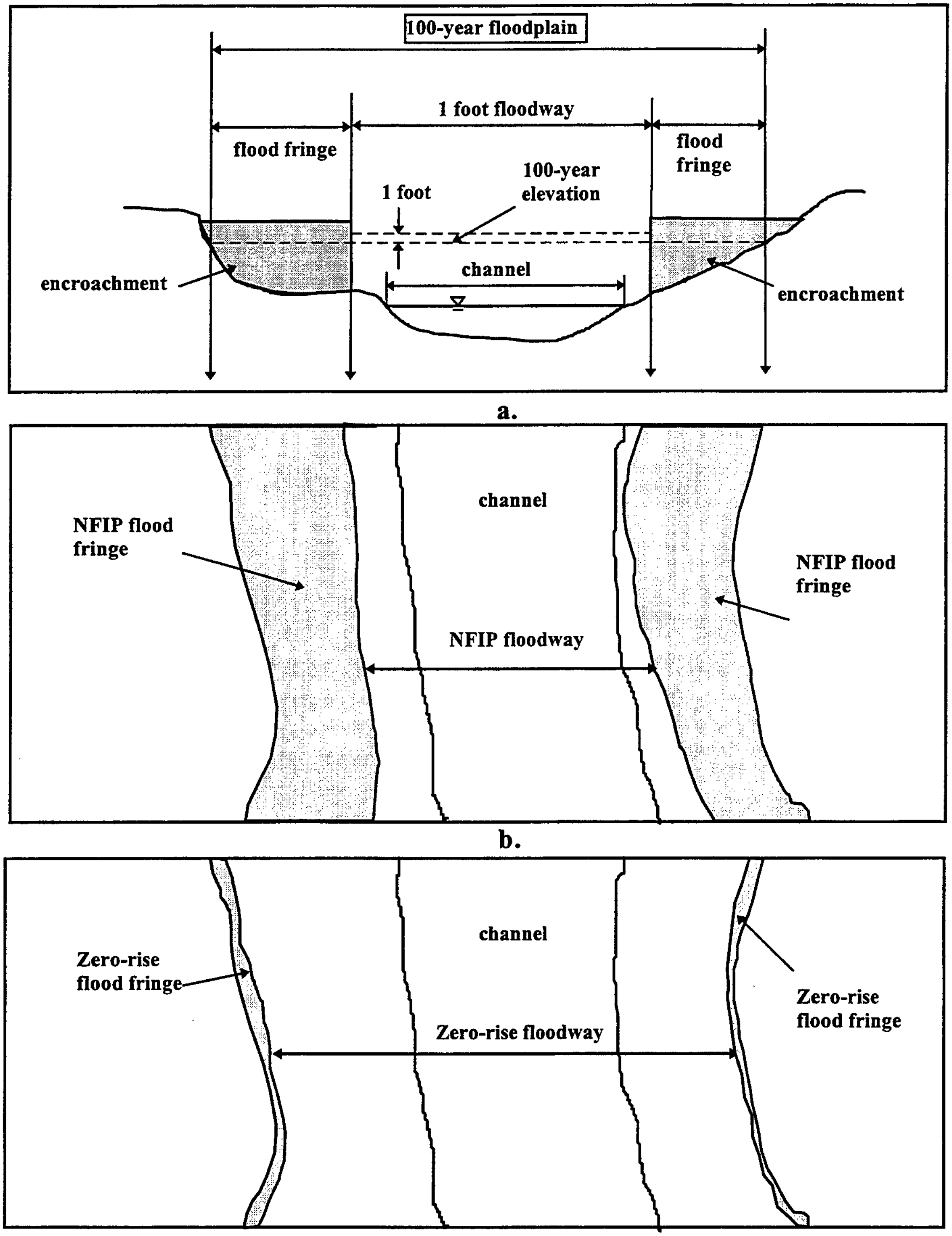

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Figure 1.1. Comparison of NFIP and Zero-Rise Floodplain Regulations.
a) Profile of channel and floodplain; b) plan view of the NFIP 1 ft. floodway; and c) plan view of the zero-rise floodway.

This report evaluates two issues associated with compliance with a zero-rise ordinance within King County:

- the economic tradeoffs between compliance and non-compliance with the zero-rise ordinance
- the ability of engineers to accurately predict, model and measure changes in the water surface elevation to within 1/100 of a foot.

These issues, which concern water surface elevation increases, address only the first part of the zero-rise ordinance adopted by King County. The second portion of the ordinance, which deals with floodplain storage capacity, is not addressed in this report.

To adhere to a requirement limiting water surface elevation increases to .01 foot, an engineer must be capable of predicting, measuring, and estimating floodplain impacts extremely accurately. The impacts were investigated in this study included the increase in water surface elevation resulting from the construction of a bridge or road within the floodplain, and the effect of this increase upstream of the encroachment.

1.1 Floodplain Management

The purpose of floodplain management is two-fold: to minimize flood damage and to preserve and protect natural resources from degradation. Poor management practices in parts of a watershed, such as clear-cutting or excessive development within floodplains, can cause flooding in other parts. Damages can be reduced by several methods (Heijne, 1992):

- increasing the capacity of the river channel, which in turn will reduce high water marks for each flood
- raising banks in order to prevent flood water from escaping the channel
- building retention ponds upstream of the area affected and releasing flood waters at a gradual rate after the event
- preventing additional growth in critical floodplain areas.

Although these methods can help protect property from flooding locally, protection can only alleviate flood damages and not entirely prevent them. The methods do not necessarily preserve and protect human and natural resources from degradation. In fact, potential problems may arise as a result of damage reduction measures. For example, raising banks not only reduces natural flood storage and attenuation, but also increases the velocities within the river channel during a flood event. This in turn can potentially increase scour of the channel bed or cause sediment deposition downstream.

Damage reduction measures can also change the geomorphology of the rivers. Raising banks tends to constrain the natural tendency of rivers to meander. The resulting changes to the river environment can cause loss of natural habitat such as river pools and riffles, plant life along the riverbanks, and the re-supply of nutrients to the alluvial plain.

1.2 Washington Statutes and the Zero-Rise Ordinance

Washington state statutes have been established to address the issues of minimizing flood damages while preserving and protecting natural resources from degradation. Because rivers and watersheds cross city, county, and state boundaries, careful consideration and cooperation among jurisdictions are required in the application of state laws and regulations.

Three principal interrelated Washington statutes address floodplain management activities. Section 86.12 RCW - Flood Control by Counties enables counties to take steps necessary to reduce and prevent damages resulting from flooding, such as condemning properties or levying taxes. As of 1991, this section was amended by the Engrossed Substitute Senate Bill 5411 (ESSB 5411). This bill authorizes all counties to adopt comprehensive flood control management plans for any drainage basin within its jurisdiction. Basins are to be reviewed on a watershed-wide basis. The need for any flood control activity outlined in these plans must be evaluated on the basis of a benefit-cost (B/C) ratio between public benefits, which include economic productivity and resource management, and project costs. This bill also requires that non-structural alternatives to in-stream flood control measures be investigated.

The second Washington state statute is Section 86.16 RCW - Floodplain Management. To lessen continual flood damages and advocate public health and safety, the State of Washington undertook the responsibility of full regulatory control over state waters. To emphasize that preventing flood damages should be a statewide concern, Washington State Department of Ecology was made responsible for the regulatory control over floodplain management. This statute requires cities and counties to adopt floodplain management ordinances. These ordinances are typically designed to reduce the risk of flood damage by strategic land use planning, specifically, restricting development within the floodplain. All floodplain ordinances must comply with the minimum standards required for participation in the NFIP.

The third statute, Section 86.26 - State Participation in Flood Control Maintenance, establishes the Flood Control Assistance Account Program to provide funding for comprehensive flood hazard management planning. This statute also outlines the guidelines for using such funds (Owens and Rudd, 1991).

King County's zero-rise sensitive area ordinance is in accordance with Section 86.16 RCW - Floodplain Management. This ordinance does not allow any bridge piers, abutments, roadway fills, or other development within the floodplain that will increase the 100-year water surface elevation by more than 0.01 ft. Nor does it allow any development activity to reduce the effective storage volume of the 100-year floodplain. However, it does allow an activity such as grading or roadway fill to occur if the ability is mitigated through the creation of compensatory storage of an equal amount, preferably upstream of the development.

The underlying goal in each of these three statutes is to prevent damages caused by flooding. These statutes imply that restricting development in flood prone areas is preferred to constructing flood control measures. Encroachment into floodplains and the damages due to flooding have been reduced as a result of these statutes. However, the extent to which damages have been reduced and to what degree they are preventable should be assessed. With the existing development within the floodplains today, one can not expect to reduce or prevent flood damages unless the development is removed from the floodplain. At best, today's regulations can only hope to prevent the amount of damages from increasing.

The difference between essential development to support infrastructure and commercial or residential development is also a key issue. Section 3 (2a) of ESSB 5411 requires a cost-benefit analysis be completed for any flood control activity. If the flood control activity were to restrict building bridges within the floodplain, the cost to benefit ratio would appear favorable because project costs would be nonexistent and the public benefits of reduced flood damages would be great. However, if the negative benefits of building longer bridges to span floodplains were considered, the cost to benefit ratio would not appear to be as favorable. In fact, the disadvantages of high bridge costs and reductions in highway safety could far exceed the costs associated with flood damages. It seems that the public's willingness to be burdened with the additional costs of a single span bridge or with the possibility of trading one hazard (i.e., flooding) for another (i.e., increased traffic accidents) should be assessed. A cost comparison between the additional costs of building a single span bridge and the damages due to flooding for each of two case studies is discussed in Section 4.

The remainder of this report is organized into four sections. Section 2 discusses the methodology used in conducting the sensitivity analysis and damage assessment of the case studies. Descriptions of the case studies and the water surface profile model, HEC-2, used in this analysis are also provided. Sections 3 and 4 present the results and discussion of the sensitivity analysis and the damage assessment of the case studies, respectively. A discussion of alternatives to a zero-rise ordinance follows in Section 5. Conclusions are summarized in Section 6.

2.0 METHODOLOGY

2.1 Case Study Description

Five case studies were conducted to assess the feasibility of applying the zero-rise ordinance and its effects on bridge construction and maintenance. These case studies included highway bridge projects located in King County, administered by either King County or the Washington State Department of Transportation (WSDOT). They were chosen on the basis of the impact that the zero-rise ordinance has had on the design of the project, the number of people using the bridge or road, and the potential non-bridge damages due to flooding.

The five case studies were as follows:

- Case Study Number 1 - SR-169 Bridge over Cedar River
- Case Study Number 2 - 149th Ave. SE (Elliot) Bridge over Cedar River
- Case Study Number 3 - Meadowbrook Bridge over Snoqualmie River
- Case Study Number 4 - SR-18 Bridge over Green River
- Case Study Number 5 - Preston - Fall City Road Bridge over Raging River

The project locations for the five case studies are shown in Figure 2.1.

The methodology used in the case studies involved:

- selecting a model
- predicting the change in water surface elevation by a sensitivity analysis
- estimating the incremental flood damage potential created by each bridge project.

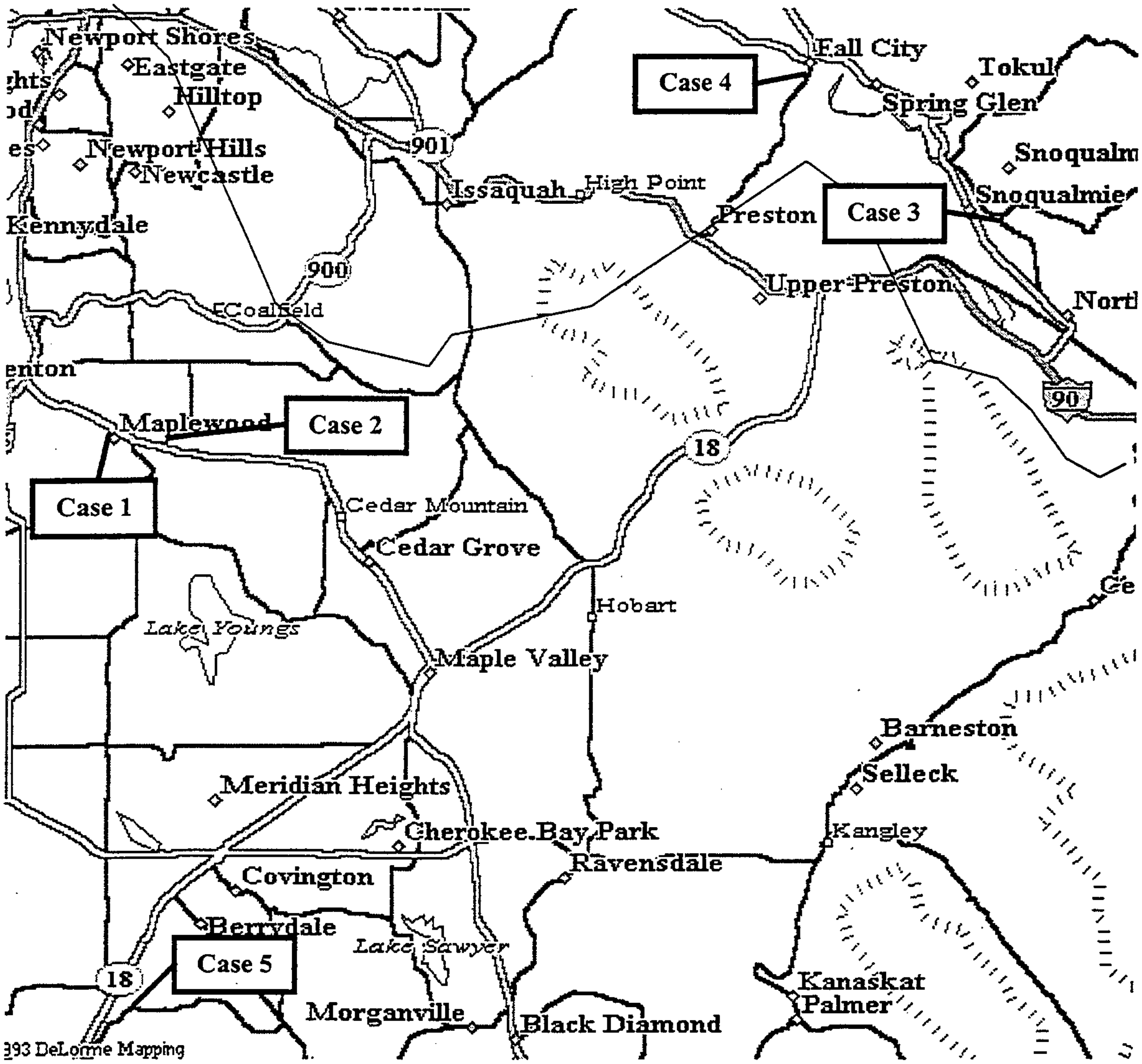


Figure 2.1. Map Showing Project Locations for the Five Case Studies.

2.2 Water Surface Profile Model, HEC-2.

The U.S. Army's Hydrologic Engineering Center's (HEC) water surface profile model, HEC-2, was used to complete a sensitivity analysis for each of the five cases. HEC-2 is a one-dimensional model commonly used to calculate flows and the impacts of changes in the alluvial environment. The model was chosen for this study because of its wide use by federal, state, and local governments in flood damage evaluations and because it reports water surface elevation to the nearest 0.01 foot.

HEC-2 calculates the water surface profile for a specific reach of channel with the step method. This method is commonly used for irregular channels such as rivers for two reasons:

- Channel properties are usually only available from survey data at specific cross-sections along the river.
- Even though channel properties are known at given sections along the reach of the river under study, the variations in channel properties due to irregularities of the channel make it difficult to calculate a reach length along the river from known depth variations (Henderson, 1966).

To model a river reach, HEC-2 requires several input parameters. These include the following:

- discharge
- downstream water surface elevation
- cross-section geometry
- reach lengths
- Manning's roughness coefficients.

Discharge and surface elevation are related. During each time step, HEC-2 evaluates a single discharge rate. For this discharge, the corresponding water surface elevation must be entered into the model at the farthest downstream section. When available, rating curves relating water surface elevation to the discharge rate at known gauging stations were used for this study.

Cross-section geometry and reach lengths define the physical characteristics of the river channel and the surrounding floodplain or "overbank." Cross-sections are taken at locations where discharge, slope, or roughness change, and at control structures or bridges. The reach length defines the distance between each cross-section for the both the right and left overbanks and the main channel.

To ensure that output is representative of actual conditions, models are calibrated on the basis of storm flow measurements near the river reach under study. Known high water marks are entered as input, and the roughness coefficients are then calculated on the basis of the measured discharge during the event. Once the roughness coefficients from the storm flows have been determined, the model is considered "calibrated" for all flows. It is recognized that roughness coefficients vary for different discharge rates and that roughness values for a 10-year discharge may not be the same as those for a 100-year storm.

For the sensitivity analysis, the two primary outputs of interest are

- the computed water surface elevations
- the extreme limits of the channel width.

The computed water surface elevations in each reach determine the maximum rise in water surface created by adding or modifying a bridge across the floodplain. The maximum rise is then compared to the 0.01-foot rise requirement. The second output, the limits of the channel width, comprises the locations where the water surface intersects the ground surface. This is useful for determining the limits of inundation. The output gives the station and elevation where the water surface meets the ground on both the right and left sides of the channel. The width of overflow past the normal channel banks can then be determined. HEC-2 reports both these output values to the nearest 0.01 foot.

To evaluate the accuracy of the water surface profile computed by HEC-2, it is important to consider uncertainties associated with the model. The three main reasons for variations are

- input parameter uncertainties
- floodplain delineation uncertainties
- model limitations.

2.2.1 Input Parameter Uncertainties

As previously mentioned, the five input parameters for HEC-2 include discharge, water surface elevation, cross-section information, reach lengths, and Manning's roughness coefficient. Small variations in estimating the 100-year discharge and Manning's roughness coefficient can cause the predicted water surface elevations to rise significantly over the maximum 0.01-foot requirement. The sensitivity analysis is discussed in detail in Section 2.3.

The hydraulic input variables with the greatest uncertainty are discharge and Manning's n. The cross-sectional data of ground elevation and station, as well as reach lengths, are obtained by field survey. This is the most accurate method of obtaining cross-sectional data. Larger errors can be expected when aerial surveys or topographical maps are used (USACE, 1986). The rating curves were assumed to be accurate for determining the starting water surface elevation.

2.2.2 Floodplain Delineation Uncertainties

Floodplain delineation requires three steps: establishing the design event (such as a 100-year flood), estimating the flow associated with that event, and calculating the one inundated by that flow. Even if the 100-year discharge and its associated stage can be predicted with some accuracy, the resulting floodplain is still difficult to predict. The inundation limits could vary greatly depending on how long the flood lasted, when the flood occurred, or any sediment transport or debris accumulation during the flood event.

Burges (1979) quantified the uncertainty with which floodplain width can be calculated. Burges estimated the standard deviation of floodplain width to be one-third of the calculated width. For the example in Figure 2.2, this implies that there is a 67 per cent probability of the true width being between 1,270 and 2,570 feet, or a 33 per cent probability of the true width being less than 1,270 or more than 2,570 feet. This uncertainty in the extent of inundation of one standard deviation corresponds to an approximately 1-ft. variance in the water surface elevation at the average floodplain boundary. This simple example indicates the difficulty in attributing the change in elevation to development within the floodplain or inaccurate floodplain delineation.

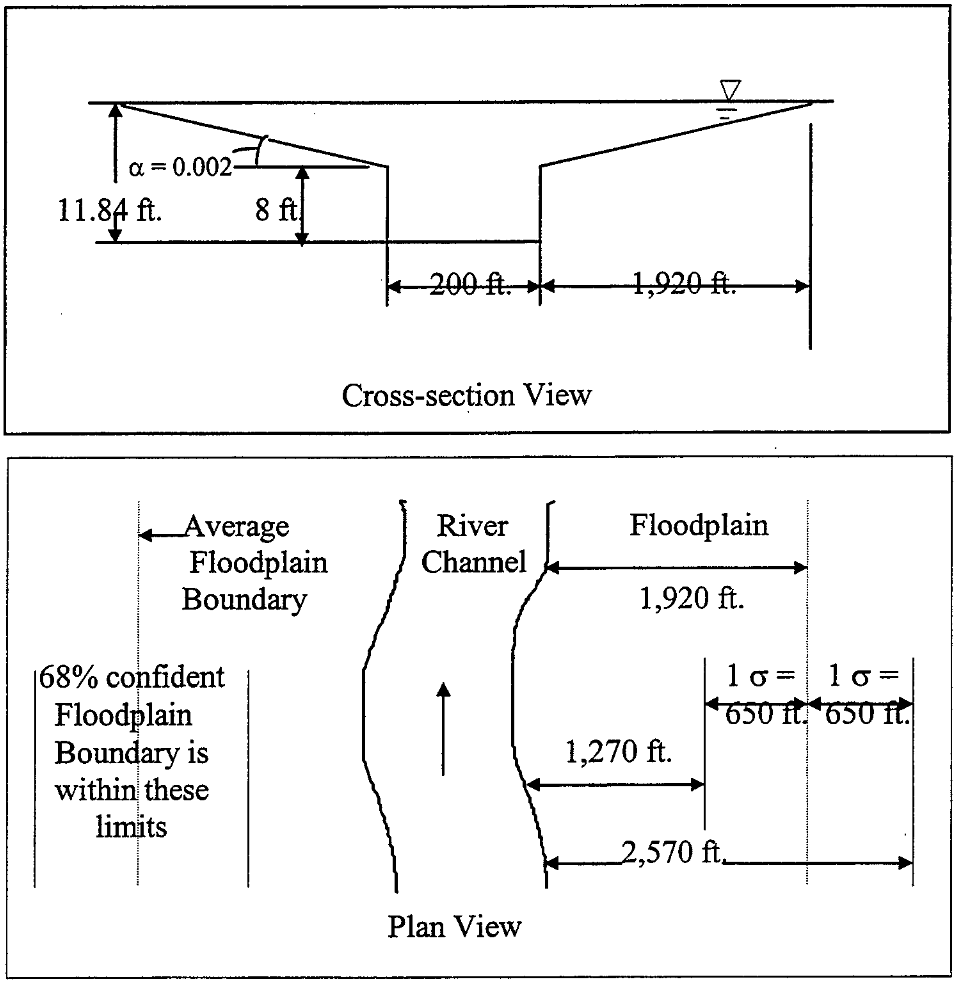


Figure 2.2. Cross-Section and Plan Views of Floodplain Geometry, Illustrating the First Order Uncertainty Analysis.

2.2.3 Model Limitations

The authors of the HEC-2 have reported that for the types of rivers typically found in King County, the model is accurate to only .5 feet for water surface elevation (USACE, 1986). To assess the ability of HEC-2 to accurately predict flood rise to the nearest 0.01 foot, it is important to understand the limitations of the model created by the following assumptions:

- one dimensional flow
- constant water surface elevation for each cross-section
- steady, gradually spatially varied flow
- continuous flow
- linear interpolations of cross-sectional data.

The model is limited to one-dimensional flows because the total energy head is assumed to be constant for all points in a cross-section. HEC-2 cannot accurately model locations with two-dimensional flow, such as river bends and other locations where the main channel overflows into the overbanks. In these locations, the flow of the water contains both lateral and downstream components of velocity.

HEC-2 assumes a constant water surface elevation across each cross-section. Velocity is averaged over the cross-section. This is not representative of actual conditions at river bends, where super-elevation of the water surface can be expected at the interior of the bend. In reality, velocities throughout a river cross-section vary. For a constant total energy head, the water surface should fluctuate to accommodate the changes in velocity heads across the section.

The river reach must have steady, gradually varied flow to be accurately modeled by HEC-2. The energy equation is used to calculate water surface profiles. This excludes any time-dependent terms and assumes a hydrostatic pressure distribution for each cross-section. This is only true if there is no vertical acceleration in the cross-section. River channel slopes should be less than 1:10 to minimize the effect of vertical acceleration (USACE, 1992). The flow should also be constant without a rapidly varying flood hydrograph.

Next, the discharge is assumed to remain constant between cross-sections on the basis of continuity. HEC-2 evaluates only the cross-sections included in the input files and not at flows between sections. The model does not account for any distributed sources or sinks, such as overland flow or exchanges into or out of subsurface soils, nor does it accommodate areas of secondary channel flow outside of the cross-sectional area.

Finally, survey data may be insufficient at some cross-sections where the endpoints (where the water level intersects land) have not been surveyed. For this case, HEC-2 linearly interpolates between survey data points. The flow can only be modeled to the extents of the cross-sectional survey. HEC-2 assumes that an imaginary wall vertical at the first and last station points in the cross-section contains the flow.

2.3 Sensitivity Analysis

Surface water elevation predicted by HEC-2 are sensitive to all input variables, but three are selected here for analysis: discharge, Manning's n, and location. A sensitivity analysis was completed for each case study to quantify the change in water surface elevation caused by a reasonable range of these values. The last of these parameters, pier size and location, were analyzed for Cases 1 and 2 only.

2.3.1 Discharge

Establishing the 100-year flood discharge is difficult for several reasons. Even with good historic data, controversy exists regarding how to calculate the proper design event (Dept. of Army, 1992a). The accuracy of the prediction depends on the quality and length of past records. If 60 years of hydrologic data contain only high-frequency floods, such as 2- or 10-year storm events, the estimate for a low-frequency flood, just as the 100-year storm event, could be quite inaccurate.

The degree of uncertainty in predicting discharge is inversely proportional to the number of observations. For a flood frequency curve with a coefficient of variance of 0.25, the uncertainty (standard deviation of 100-year flood / mean value for 100-year flood) can be reduced by 1/3, from 0.29 to 0.20, when the number of observations increases from 30 to 60. An increase from 30 to 200 observations reduces the uncertainty in estimating the 100-year storm event by two thirds, from 0.29 to 0.09 (Burges, 1979). (See Table 2.1.)

Table 2.1. Approximate Uncertainty Bounds for Estimates of the Magnitude of the 100-Year Flood. (Burges, 1979)

Flood Population: (Gumbel) Extreme Value Type 1 CV = 0.25;		Sample Size	
	30	60	200
Q_{100}/Q =	1.75	1.75	1.75
(68% Confidence Bounds)/Q =	1.30 - 2.25	1.40 - 2.10	1.65-1.90
S_{100}/Q_{100} =	0.29	0.20	0.09

Notes: Q_{100} = 100-year flood; S100 = std. dev. of 100-year flood estimate; Q = mean annual flood (2.33 year recurrence interval); CV = population std. dev./mean annual flood.

To determine the expected value and standard deviation for the 100-year discharge in the five case studies, the available peak flow data were analyzed with Log-Pearson Type III distributions. Because no available analytical formula is available to predict the distribution of the 100-year return interval flows when the Log-Pearson distribution is used, Monte Carlo simulations were performed by using the statistics from the discontinuous annual peak flow data. The values of the 100-year return interval events generated by the simulations were then tabulated and analyzed. Estimated 100-year mean peak flow and standard deviation were calculated for each case study (Table 2.2). The flow records were assumed to represent a stationary process, meaning no watershed changes during the period of record were assigned to change the run-off characteristics of the basins.

Table 2.2. 100-Year Peak Flows and Standard Deviations.

Case Study Number	Number of Years of Data	100-year Peak Flow (cfs)	Standard Deviation (cfs)
1	40	11,632	1198
2	40	11,632	1198
3	32	80,168	8313
4	47	7413	1116
5	N/A	12,000	N/A (Upstream Regulated Flow)

For the sensitivity analysis, five estimates of the 100-year peak flow were used: the expected 100-year peak flow, and both ± 1 and ± 2 standard deviations about the expected 100-year peak flow. This range was assumed to incorporate the uncertainties caused by using historical data to predict the flow rate. This assumption was supported by the different predictions from various state agencies of the 100-year flow rate. Figure 2.3 displays the range of discharge rates used for the first two case studies on the Cedar River, and the discharge rates used by various state agencies. Discharge rates of approximately 9,200 cfs to 14,000 cfs were used in the sensitivity analysis. FEMA and WSDOT use a predicted discharge of 8,530 cfs. This estimate falls below the range modeled in the case studies, but the prediction is considered low because it does not include data from the floods of 1990-91. King County Surface Water Management and the Army Corps of Engineers use a higher predicted discharge rate of 12,000 cfs. This includes some data from these recent floods, but unfortunately, the data were incomplete because the stream gauge was washed out during the flood events.

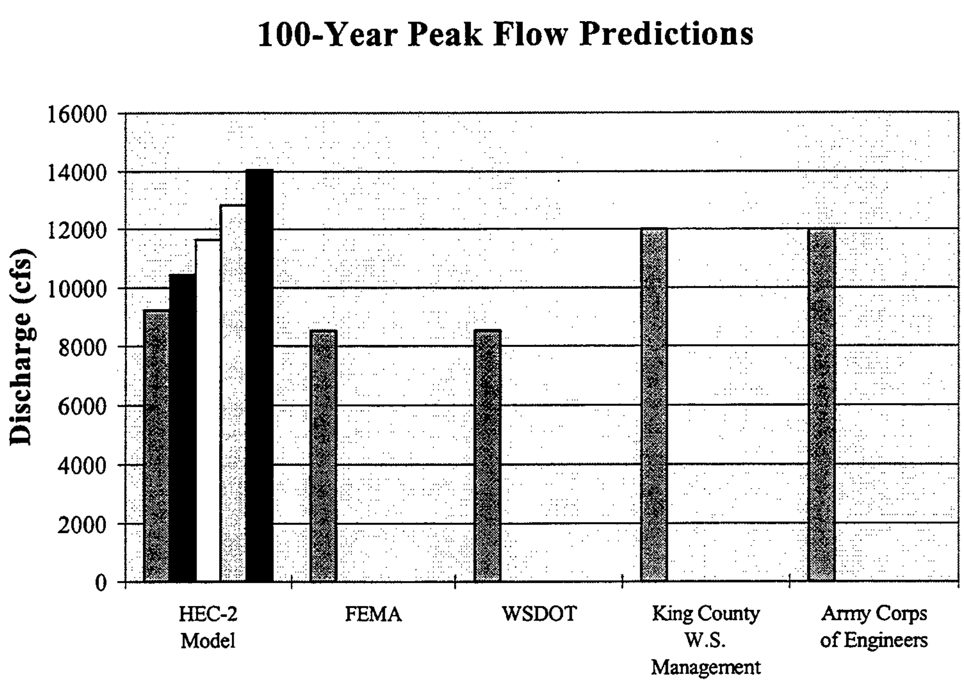


Figure 2.3. Range of 100-Year Discharge Rates Modeled for Cedar River Case Studies and Those in Use by Federal, State and Local Agencies.

2.3.2 Manning's Roughness Coefficient

The roughness coefficient is a measure of the resistance of the river bottom and boundaries to flow. It is used to calculate the energy friction loss created by movement of water across the riverbed. Uncertainties in predicting Manning's roughness coefficient exist because of a lack of information, the cost of acquiring this information, and the non-uniformity of roughness over both time and space. Most methods for determining Manning's roughness coefficients are based on uniform flow, particle size, and particle shape. Additionally, Manning's roughness coefficient varies with the flow rate. An accurate prediction of Manning's n must include effects of cross-section irregularities, channel slope, flow rate, obstructions such as bridges, vegetation, channel meandering, sediment characteristics and transport, and channel and floodplain conditions in urban or agricultural areas (Jarrett, 1984).

Manning's roughness coefficient depends greatly on the bed type. Table 2.3 shows the range of roughness coefficients for various channels. In the case studies, five values of Manning's n were used for input values of the channel roughness in the HEC-2 model at the location of the bridges: 0.025, 0.030, 0.035, 0.040, and 0.045. These values encompass the range of channel roughness values associated with natural earth channels. The overbank roughness was not varied in the sensitivity analysis, and the roughness of the main channel was left unchanged both upstream and downstream of the bridge. This represents a somewhat conservative approach for treating the uncertainty involved in estimating the Manning coefficient. Even a small variation in this coefficient, if applied over the entire modeled reach, would create a significant change in the predicted water surface elevation.

Table 2.3. Variation of Manning's Roughness Coefficients with Bed Type. (Maidment, 1993).

Bed Characteristics	Reference Manning's Roughness Coefficient n
Clean and straight earth channel	**0.025-0.04**
Winding earth channel with some weeds	**0.03-0.05**
Mountain streams with boulders	**0.04-0.1**
Short grass floodplains	**0.02-0.04**
High grass floodplains	**0.03-0.05**
Dense willow, brush, etc. in the floodplains	**0.05-0.20**

2.3.3 Pier Size and Location

Variations in pier size and location can cause changes in the water surface elevation. The size and location of piers are parameters that engineers can control, and they are also significant drivers of total bridge cost. Therefore, a sensitivity analysis was completed to analyze the effect of these two variables on water surface elevation for the first two case studies, in which a new bridge was constructed. Pier sizes investigated in this analysis ranged in diameter from 4 to 10 feet. Model runs were completed to determine the water surface elevation for four sizes of piers falling within this range: 2, 4, 6, and 8 feet. Additionally, runs were completed for piers in three different locations: the left, middle, or right section of the main channel as shown in Figure 2.4. No combinations of pier size and location produced less than 0.01 foot of rise.

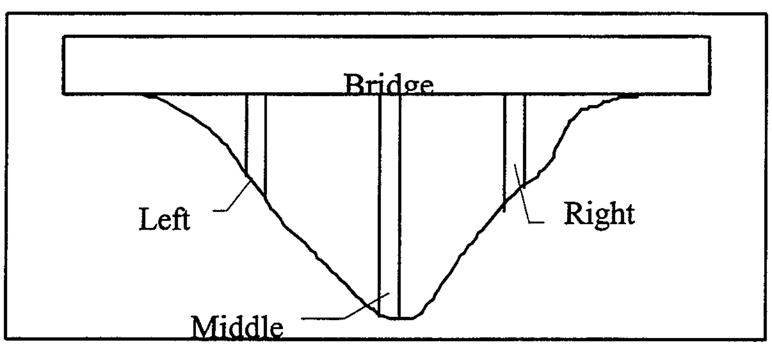


Figure 2.4. Pier Location from Within the River Channel Looking Upstream.

2.3.4 Calculations for Change in Water Surface Elevation

To complete the sensitivity analysis, 25 model runs, combining the five discharge values and five Manning's n values, were completed for both the existing and proposed conditions of the bridge. For the first two case studies, in which the sensitivity analysis included the effects of pier size and location, an additional 12 runs were completed. During each run, the water surface elevation was determined for the existing conditions of the bridge, i.e. the condition of the bridge before construction or modification. Additionally, the water surface elevation was determined for the proposed condition of the bridge.

The water surface elevation at the maximum point of backwater was used for the sensitivity analysis. The greatest change of water surface elevation is expected to occur at this location. The greatest backwater elevation typically occurs at a distance equivalent to one bridge width upstream of the bridge (USCOE, 1995) and is referred to as the critical section.

Figure 2.5 is a schematic of the existing 100-year floodwater surface elevation and the proposed 100-year floodwater surface elevation for the critical section, as modeled by HEC-2. The change in water surface elevation is defined as the difference between the existing and proposed water surface elevations. This change is used to verify compliance with the 0.01-ft. requirement.

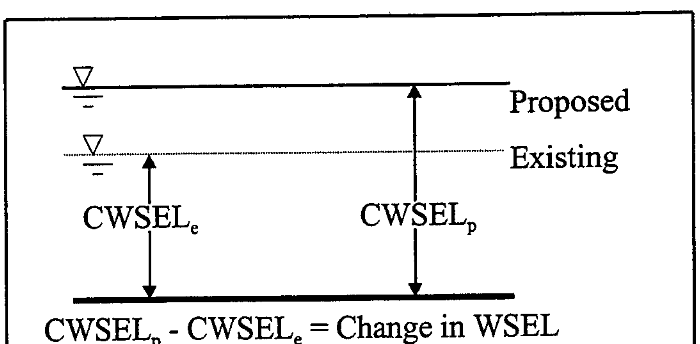


Figure 2.5. Computed Water Surface Elevations (CWSEL) for Proposed and Existing Conditions.

As an example, the water surface elevation for the first case study for a discharge rate of 9,236 cfs and Manning coefficient of 0.33 is 76.36 feet for the existing conditions and 77.18 feet for the proposed conditions (Figure 3.3). The change in water surface elevation is the difference between these two values, or 0.82 feet (Figure 3.4).

2.4 Impact Analysis

When the water surface elevation rises during a 100-year flood, the potential damages from the inundation of the floodplain can be significant. Figure 2.6 shows a map of the town of Snoqualmie, where the third case study was located. The city could be inundated if the river flooded outside of the normal channel banks, as it lies adjacent to the river and at a similar elevation.

The damages caused by an increase in water surface elevation must be evaluated to compare the economics of bridges that comply with the zero-rise requirement to bridges that do not.

The steps for completing a damage analysis include the following:

- determining the extents of inundation
- calculating land damage
- calculating structural damage.

An impact analysis was completed for case studies 1, 2, and 3.

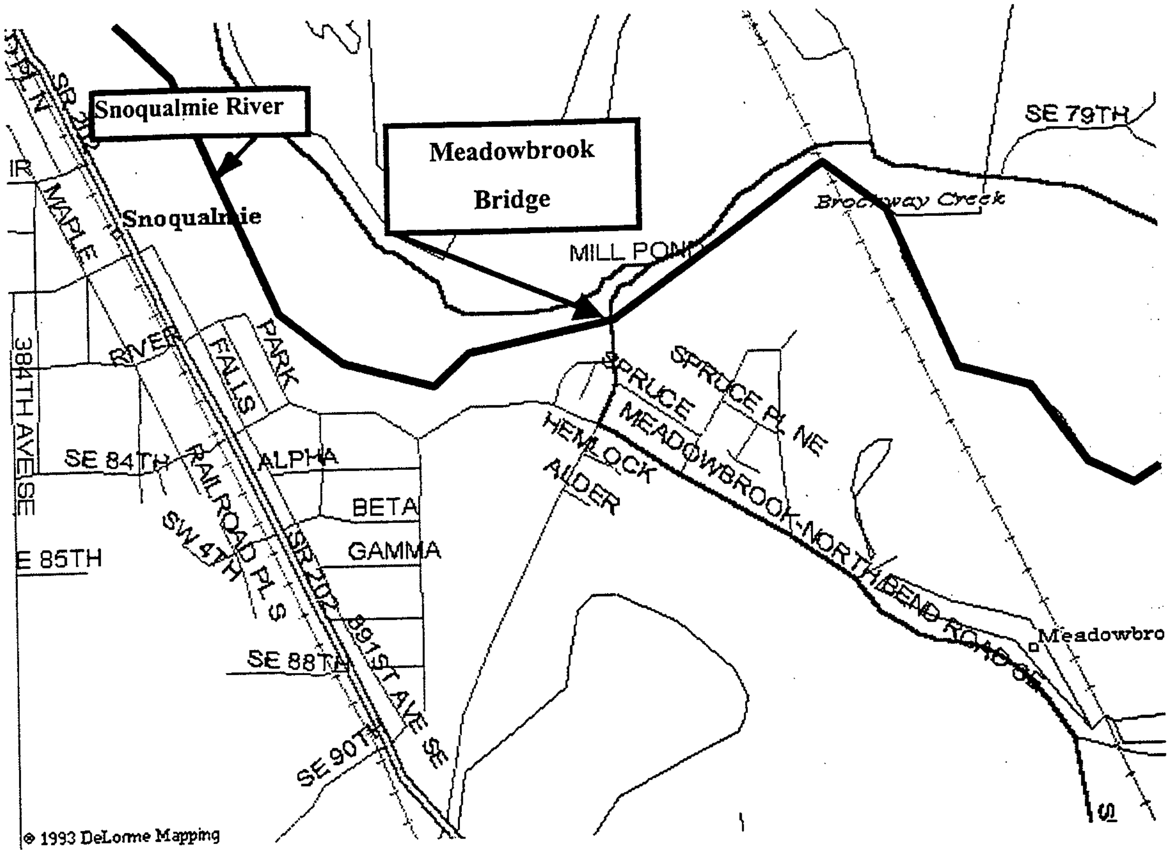


Figure 2.6. Map of Snoqualmie, Washington.

2.4.1 Extent of Inundation.

To determine the damage caused by an increase in water surface elevation, the extent of inundation for both the existing and proposed conditions of the bridge must be determined. The inundation limits correspond to the water surface elevations calculated in the sensitivity analysis. Information from the sensitivity analysis model runs was used to determine the inundation limits for the damage assessment.

In the first two case studies, the analysis focused primarily on the location of maximum backwater. In these case studies, the location of maximum backwater was assumed to experience the most damages. For the third case study, because of the excessive width of flooding during the 100-year storm and the large number of homes in the area, the whole reach of river upstream of the bridge was evaluated for the damage assessment.

The extent of inundation for land damages was determined from surveyed cross-sections and aerial maps. HEC-2 reports the inundation limits (the end point station) as the station of the cross-section where the water surface intersects the ground. To find the extent of inundation, a straight line was drawn from the endpoint of the cross-section immediately upstream of the area of interest to the endpoints of all cross-sections in the region of interest. This line was also extended to the cross-section immediately downstream of the area of interest.

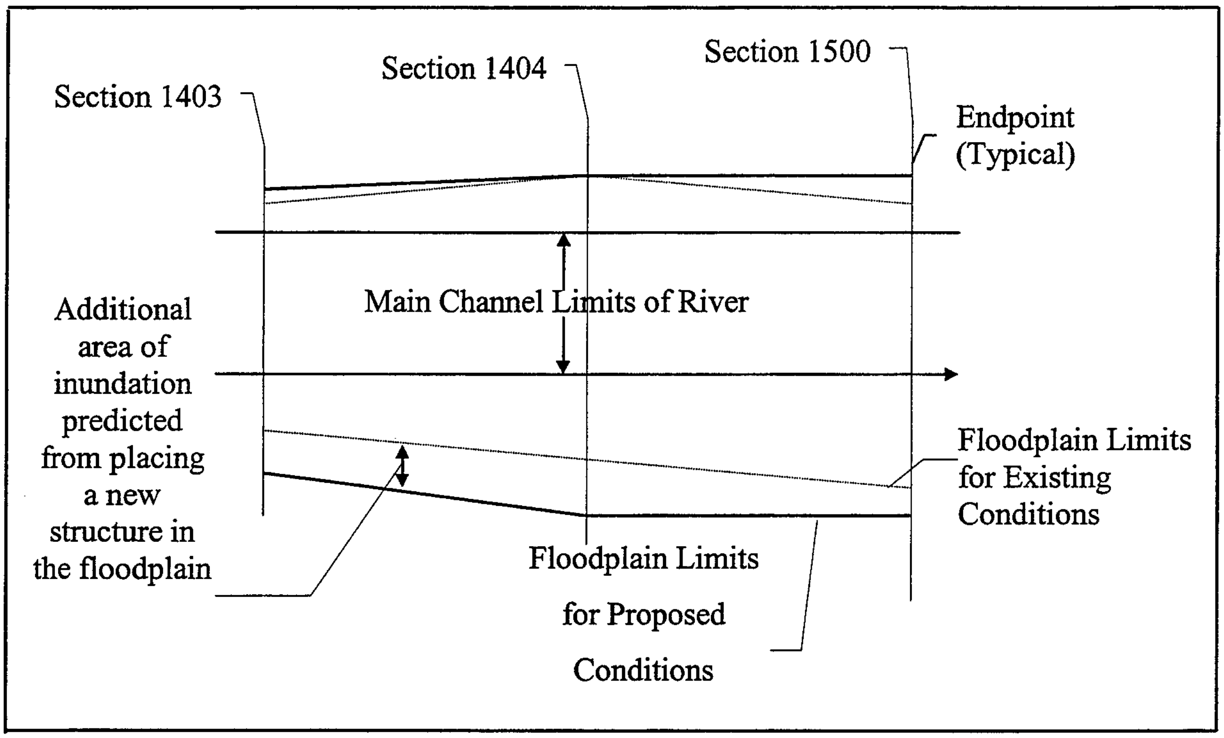


Figure 2.7. Determination of Land Inundation Due to Bridge. Critical section 1404 from the Cedar River bridge case study was used here as an example.

The area bordered by the line and the edge of the river's main channel between cross-sections was calculated for existing and proposed conditions (Figure 2.7). The extent of inundation due to constructing a bridge in the floodplain was calculated as the difference between the area inundated by the proposed conditions and the area inundated by the existing conditions.

2.4.2 Land Damage Assessment

Usually, damage to structures is the primary source of flood damages. Costs are also incurred when land is inundated, such as the costs associated with clean-up of debris and repair of damage to septic tanks or drainage fields. However, in developed areas, land damages are usually small in comparison to structural damages. Therefore, for this analysis land damages were only considered when there were few or no structures on adjacent properties.

The cost of damages due to flooding was calculated as the product of the assessed land value taken from King County Assessor's maps and the acreage of land inundated. For a conservative approach, the entire value of the land that was inundated was assumed to be lost. The increase in land damages due to bridge construction or retrofit was calculated as the difference between the land damages produced by flooding with the proposed bridge conditions and the damages caused by flooding with the existing bridge.

2.4.3 Structural Damage Assessment

Two methods were used to analyze the structural damage. The first method is the typical method for analyzing flood damage. Damages are calculated for the existing conditions and proposed conditions separately. The difference between these two costs is the flood damage produced by the changes to the bridge. However, this method did not always yield reasonable results because, as will be discussed in the next section, inundation costs are calculated in one-foot increments. If the water surface elevation does not increase by one foot, the flood damages calculated by the first method will be zero. The second method provides a way of estimating flood damages for small increments of change in water surface elevation.

2.4.3.1 Method One

The damages caused by the change in water surface elevation due to building a bridge in the floodplain depends on the width of the floodplain, the number of homes within the floodplain limits, and the depth of inundation experienced by each home. This information was estimated from topographical maps, aerial photographs, and modeled water surface profiles. Each factor was analyzed consistently for both existing and proposed conditions to determine the change in damages due to bridge construction or retrofit. The analysis will be described by using an existing condition assessment as an example.

The floodplain near the critical section was determined by using the computed and interpolated water surface elevations and 2-ft. contour maps. Given the water surface elevations and the ground elevations adjacent to the river, the limits of inundation were then estimated. The number of homes located within these limits were tagged and included in the structural damage assessment.

The water surface elevation was used to determine the depth of inundation in this area. Ground elevations were obtained from maps at various locations within the project limits, and the inundation depth was calculated by subtracting the ground elevation from the model output of computed water surface elevation. (See Figure 2.8.) These spot elevations were assumed to be equivalent to the first floor elevations of homes in the area. King County Assessor's records provided the property characteristics including the number of stories, the acreage, and the assessed value for homes in the case studies. For the purposes of this analysis, the homes in the project area were generalized to be single family, wooden frame residences with no basements. The average home value used for each case study was based on the arithmetic average of home values near the critical section

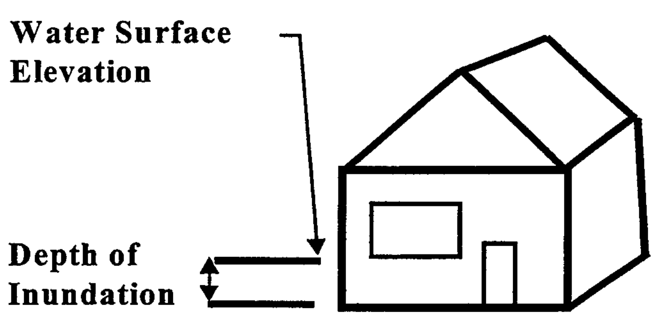


Figure 2.8. Schematic of Inundation Depth for a Home.

With the number of homes exposed to inundation during the 100-year storm event and the predicted depth of inundation, damages were assessed using the 1987 Depth/Damage Curves developed by the Federal Insurance Administration. These curves provide the percentage of structural damages associated with the inundation depth for various types of structures. Table 2.4 describes the curve used in this analysis for inundation depths in 1-ft. increments, with the first floor elevation as the datum. For example, if the first floor of a one-story house without a basement was under 0.2 ft. of water, 7.64% of the structure would be considered damaged, while 11.20% of its contents would be considered damaged. The cost of damages due to flooding is obtained by summing the product of the structure value and the structure percentage and the product of the content value and its percentage.

Table 2.4. Federal Insurance Administration (1987) Depth/Damage Curves for One Story Wooden Homes w/o Basements

Flood Depth (ft.)	Structure (% Value)	Contents % Value)
-0.5 to +0.5	7.64	11.20
+0.5 to +1.5	13.55	22.84
+1.5 to +2.5	20.61	31.39
+2.5 to +3.5	26.85	34.09

This procedure was repeated for proposed conditions. The damages directly associated with the rise in water surface elevation were then determined by taking the difference in damages between existing and proposed conditions.

2.4.3.2 Method Two

As will be shown in Section 3, the greatest change in water surface elevation between existing and proposed conditions of the bridge in Case Study 3 for reasonable estimates of discharge and roughness coefficient was estimated to be 0.11 feet. Because the depth-damage curves used to determine percentage of structural damage only measure in 1-foot increments of inundation, this change in water surface elevation was not large enough to change the level of damage. With the 100-year flood, the cost of structural damage would be very high. However, no increase in structural damage resulted from modifying the bridge when Method One was used.

For Method Two, structural damage was first assessed for the water surface elevation with the existing conditions of the bridge, as well as for a 1-foot higher water surface elevation. The difference between these costs was then pro-rated for the incremental change of water surface elevation calculated for the proposed condition of the bridge in the case study.

For instance, suppose the difference in flood damages between the proposed conditions and the existing conditions for a 1-foot higher water surface elevation was \$1000. Then the flood damage cost between proposed and existing conditions for a 0.1-foot higher water surface elevation would be 0.1 multiplied by \$1000, or \$100.

3.0 RESULTS OF SENSITIVITY ANALYSIS

A sensitivity analysis was completed for five case studies to determine the effects of uncertainties in input values on the estimation of water surface elevation. The results for each case study are described in detail in the following sections. Each section contains:

- a case study description
- the effects of discharge uncertainties on the estimated change in water surface elevation
- the effects of variations in Manning's n on the estimated change in water surface elevation
- the effects of combing discharge and roughness coefficient uncertainties on the estimated change in water surface elevation
- the effects of pier size on the estimated change in water surface elevation
- summary of the case study results.

3.1 Case Study No. 1 - SR 169 Bridge at Cedar River

3.1.1 Case Study Description

The first case study was a bridge construction project due east of Renton, Washington along State Route (SR) 169. This project, administered by WSDOT, involved the construction of a new bridge across the Cedar River. The existing SR 169 Cedar River Bridge (169/24) was a two-lane, 153-ft. long, steel-girder and concrete T-beam bridge. Because of the 27,500 vehicles observed to use SR 169 daily in 1992, this section of SR 169 (from 140th Place SE to Maplewood Golf Course) required widening. A new bridge was designed to accommodate the realignment and widening of SR 169. The existing bridge was to remain in place and be used as a part of the Cedar River Trail Crossing.

A new five-lane bridge, with 156-ft. long pre-stressed concrete girders, was constructed just upstream of the existing SR 169 Cedar River Bridge (169/24). WSDOT determined that one bridge pier would be placed within the 100-year floodplain. Therefore, the new bridge would have two spans connecting to the one intermediate pier in the 100-year floodplain above the high water mark. WSDOT anticipated that the placement of this bridge pier would increase the upstream flood elevation by approximately 1.5 inches. An existing railroad bridge just upstream of the existing SR 169 Cedar River Bridge was removed before the construction of the new bridge. The project site layout along the river was as shown in Figure 3.1.

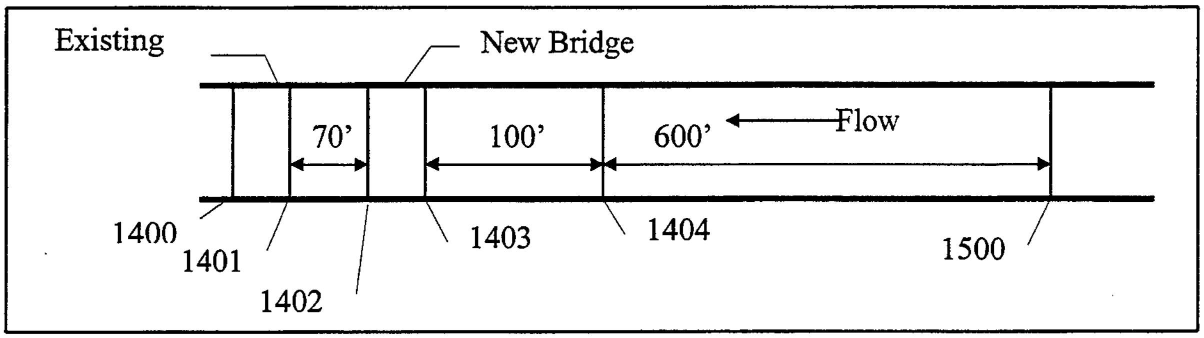


Figure 3.1. SR-169 at Cedar River Project Site Layout with Cross-sections.

A single span bridge was required to comply with the zero-rise ordinance at a cost of approximately $2.6 million. The two-span bridge, which was constructed in lieu of the single span bridge, cost $940,000. WSDOT determined that the pier would have an insignificant effect on the base flood elevation and that the net change in storage was negligible because of the removal of an existing railroad bridge.

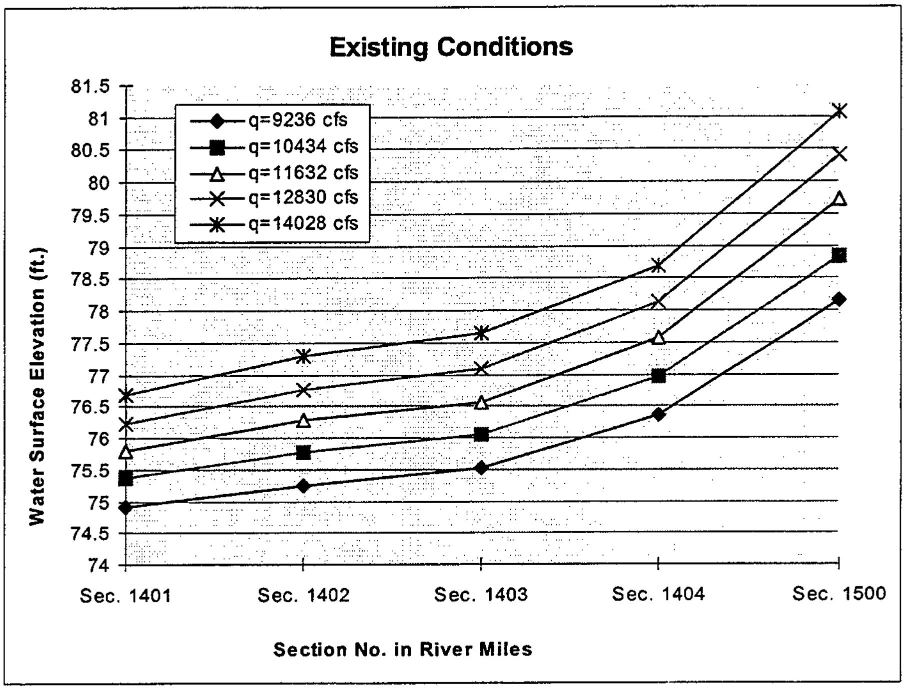
3.1.2 Change in Water Surface Elevations for Varying Discharge

As discussed in Section Two, five different discharge values were modeled to determine the effects of uncertainties in discharge on the computed water surface elevation. The gage used to predict the 100-year peak flow is located on the Cedar River in Renton, Washington and has a drainage basin of 104 sq. miles. The 100-year peak flow is 11,632 cfs, and the standard deviation is 1198 cfs.

The water surface profiles computed by HEC-2 for both existing and proposed conditions are shown in Figure 3.2. Each line represents the water surface profile computed for a specific discharge. The general trend implies that an increase in the discharge caused the water surface profile to rise for both existing and proposed conditions. This is because for a given channel cross-section, the depth of flow would increase to accommodate the additional volume of flow.

The difference between the water surface profiles for existing and proposed conditions was determined. (Refer to Figure 2.5.) A positive value represented a rise in the water surface elevation, and a negative value indicated a drop in the water surface elevation.

The change in water surface due to the construction of a bridge in the floodplain increased as the discharge increased. (See Figure 3.3.) Between Section 1401 (upstream face of the existing SR 169 bridge) and Section 1403 (upstream face of the new SR 169 bridge), the water surface elevation dropped. This drop is attributed to the energy losses that occurred at the new bridge. For this case study, an increase in discharge of one standard deviation caused, on the average, a drop in the water surface of 0.06 ft.



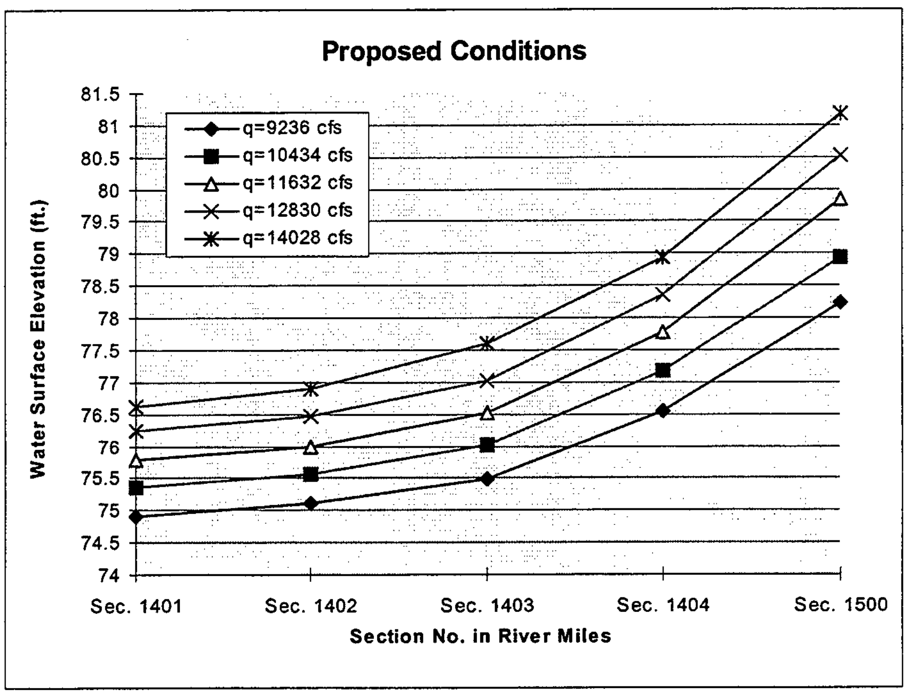


Figure 3.2. Water Surface Profiles for Varying Discharge: Existing and Proposed Conditions. Case 1.
Section Nos. refer to cross-sections along the river. This figure compares the effects various discharge values have on the water surface profiles for existing and proposed conditions. Proposed conditions include a new bridge.

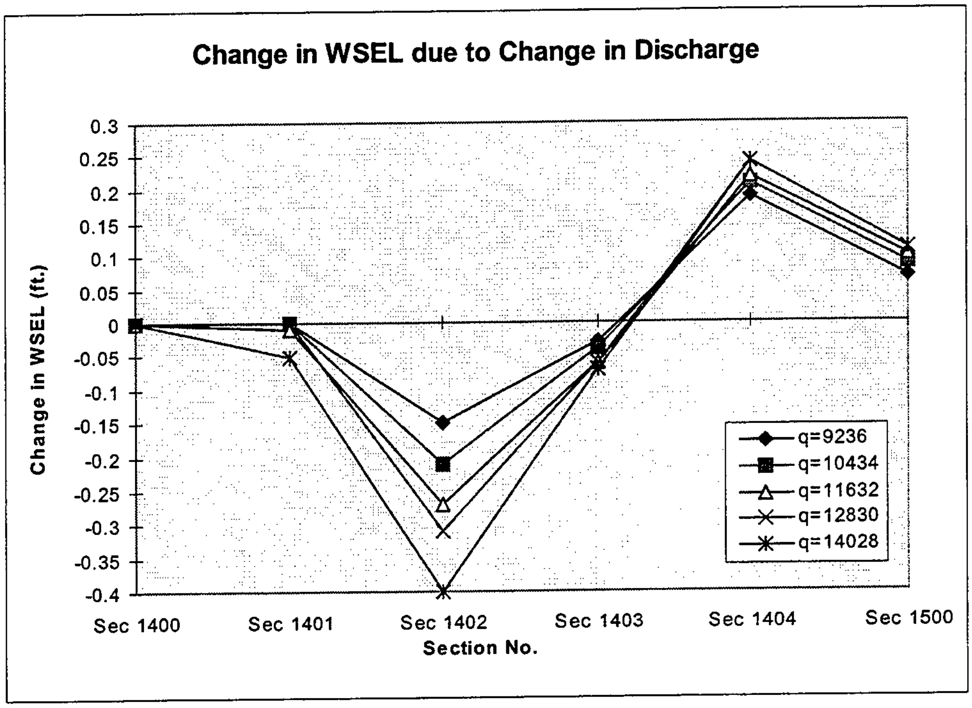


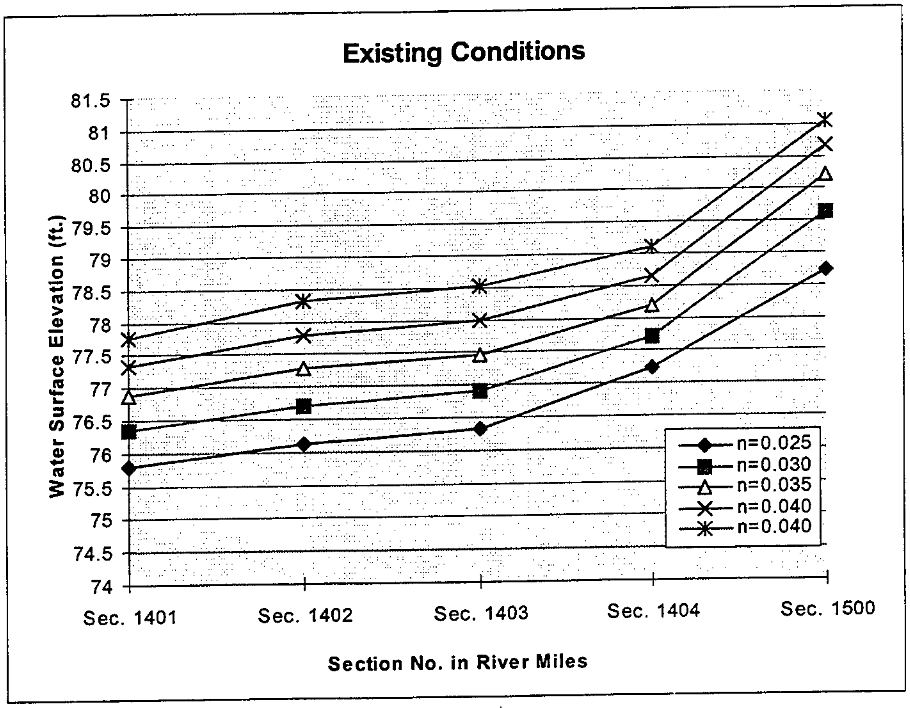
Figure 3.3. Change in Water Surface Elevation (WSEL) for Varying Discharge. Case 1. Section Nos. refer to cross-sections along the river. The change in WSEL is a comparison of WSEL with and without the new bridge for a given discharge.

The greatest backwater appeared at Section 1404, which was 100 ft. upstream of the new SR 169 bridge. Numerous runs with various discharge and roughness values verified that the critical section occurred at Section 1404. The additional backwater created from constructing a bridge in the floodplain for the lowest discharge modeled was 0.19 ft. at the critical section. At the same section, the inclusion of a bridge for the highest discharge modeled caused a 0.24 ft. increase in water surface elevation. These results show that for this case study, the possible outcomes provided by HEC-2, when five reasonable values of discharge were used, fell within a band of 0.05 ft. Even if the estimates of the 100-year discharge were limited to one standard deviation about the 100-year peak flow, the expanse of variation would still be as much as 0.03 ft. Small changes in the estimate of the 100-year peak flow could cause the predicted rises to vary by more than 0.01 ft. of rise.

3.1.3 Change in Water Surface Elevation for Varying Roughness Coefficients

To determine the effects of Manning's roughness coefficient on the computed water surface elevation, the discharge was held constant, and values for Manning's n were varied. Discharge was held constant at the mean 100-year peak flow value, 11,632 cfs, so variations in discharge would not also produce changes in water surface elevation. The roughness coefficient was varied to five different values of 0.025, 0.030, 0.035, 0.040, and 0.045--the typical range for earth channels noted in reference tables for Manning's roughness coefficients. Once again, the computed water surface profiles were graphed for both existing and proposed conditions. (See Figure 3.4.) Each line represents the water surface profile computed for a specific roughness coefficient.

The general trend shown by Figure 3.4 was similar to that found when discharges were increased. The water surface elevations increased as the roughness value increased. This was expected because with an increased in channel roughness, there would be more resistance to the flow along the wetted perimeter of the channel. The additional resistance would slow the flow. For a constant discharge, if the velocity was reduced, the flow area would have to be increased, resulting in a rise in water surface elevation. Increasing Manning's roughness coefficient by 0.005 caused an increase in the water surface elevation of approximately 0.5 ft. under existing conditions. Under proposed conditions, the increase in the water surface elevation was approximately 0.4 ft.



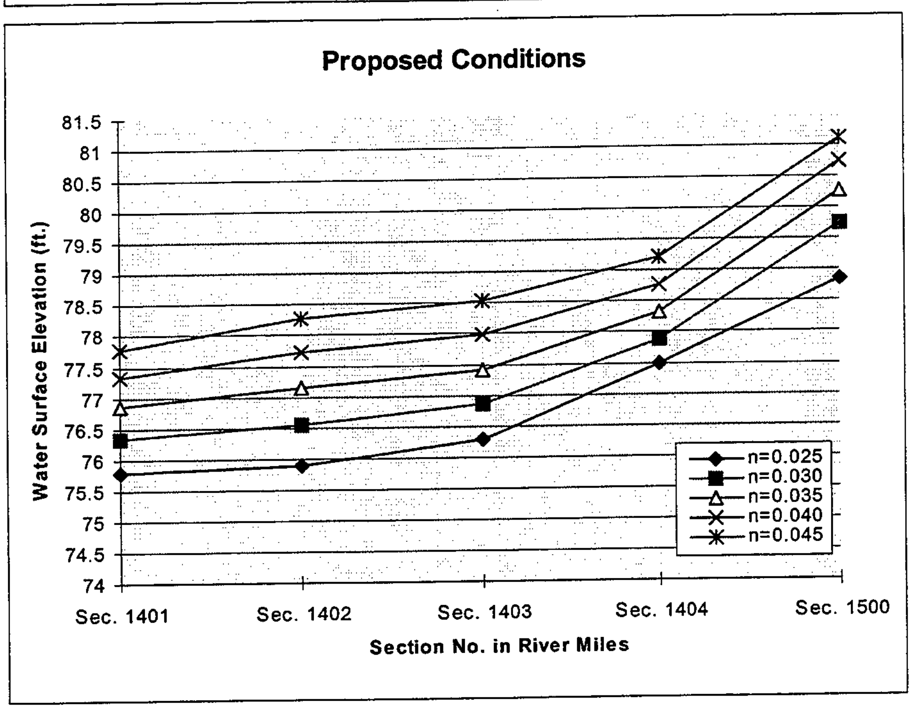


Figure 3.4. Water Surface Profiles for Varying Manning's Roughness: Existing and Proposed Conditions. Case 1.
Section Nos. refer to cross-sections along the river. This figure compares the effects that varying roughness values of Manning's n have on the water surface profiles for existing and proposed conditions and a constant discharge of 11,632 cfs.

However, Figure 3.5 indicates that the change in water surface elevation resulting from the inclusion of a bridge decreased as the roughness coefficient increased. The actual water surface profiles for both existing and proposed conditions rose as the roughness was increased. It was the change in water surface profiles that decreased when roughness was increased.

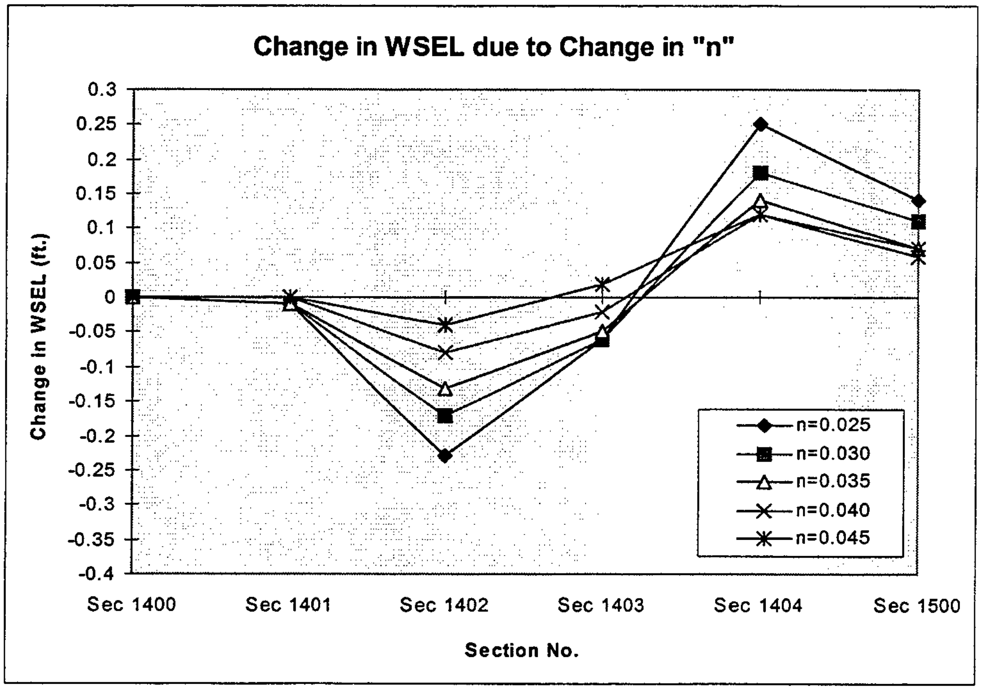


Figure 3.5. Change in Water Surface Elevation (WSEL) for Varying Manning's Roughness. Case 1.
Section Nos. refer to cross-sections along the river. The change in WSEL is a comparison of WSEL with and without the new bridge for a given roughness coefficient.

The results for this case study show that the possible outcomes provided by HEC-2, when five reasonable values of channel roughness were used, ranged from a 0.25-ft. to a 0.12-ft. rise in water surface elevation. A variation of as much as 0.13 ft. could be expected if exact roughness coefficients were not known. This variation far exceeds the 0.01 ft. value and is associated with very small variations in the channel roughness, 0.005.

3.1.4 Change in Water Surface Elevation for Combined Discharge and Manning's n Variation

The first two sets of HEC-2 runs showed the possible variations associated with different estimates of the 100-year peak flow and the channel roughness, independently. The third set of runs combined the five discharge values with the five roughness coefficients, totaling to 25 runs for existing conditions and 25 runs for proposed conditions. The purpose of combining the different values of discharge and channel roughness was to identify the range of possible outcomes when reasonable values of both input parameters were used. Using this information, the range of variation in backwater rise for changes in discharge and roughness coefficient were calculated. (See Table 3.1.)

Table 3.1. Change in Water Surface Elevation (in feet) for a given Discharge and Roughness Coefficient (Critical Section 1404). Case Study 1.

Discharge	n=.025	n=.030	n=.035	n=.040	n=.045	Range of Variation*
9,236 cfs	0.21	0.16	0.13	0.12	0.10	0.11
10,434 cfs	0.23	0.17	0.14	0.12	0.11	0.12
11,632 cfs	0.25	0.18	0.14	0.12	0.12	0.13
12,830 cfs	0.28	0.20	0.14	0.13	0.12	0.16
14,028 cfs	0.29	0.19	0.15	0.14	0.13	0.16
Range of Variation*	0.08	0.04	0.02	0.02	0.03	

*Range of variation is defined as the difference between the largest and smallest possible outcome.

For the first case study, the range of possible outcomes modeled by HEC-2, given a reasonable estimate for discharge and channel roughness, far exceeded the 0.01-foot accuracy desired for computing water surface elevation. The range of variation for a fixed roughness coefficient varied anywhere between 0.02 ft. and 0.08 ft. This implies that if the engineer had an exact estimate of Manning's roughness coefficient and a reasonable estimate of the 100-year peak flow within two standard deviations of the mean peak flow, a variation of as much as 0.08 ft. could be expected in predicting the change in water surface elevation. Even if the estimate of the 100-year peak flow was limited to one standard deviation about the mean, the variation would still be as much as 0.05 ft., which would be 400 percent over the criterion of 0.01 ft. Similarly, for a particular

3.1.5 Change in Water Surface Elevation for Varying Pier Size and Location

Combinations of pier size and location were also reviewed in this sensitivity analysis. The pier size investigated in this analysis ranged from 4 to 10 ft. in diameter. Piers were located either in the left, right, or center of the river channel. This analysis was completed to identify a possible bridge design that would comply with the zero-rise ordinance. Because the ordinance applied to the entire stretch of river under study, the 0.01-ft. rise requirement had to be met at all cross-sections upstream and downstream of the new bridge. This analysis focused on the critical section where the maximum backwater was predicted to occur.

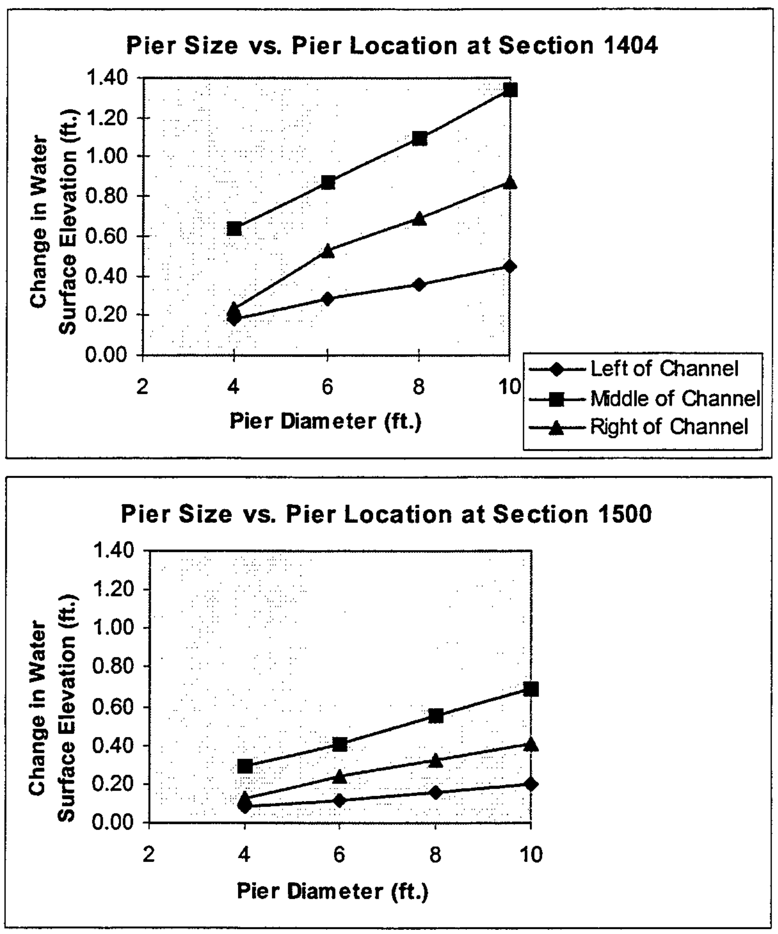


Figure 3.7. Change in WSEL for Varying Pier Size and Location. Case 1. Section Nos. refer to cross-sections along the river. The change in WSEL is a comparison of WSEL with and without the new bridge.

The results showed that at Section 1404, the bridge with a 4-ft. diameter bridge pier located in the left section of the channel caused the least change in the computed water surface elevation. (See Figure 3.7.) However, this particular design still caused more than 0.01 ft. of increase in the computed water surface elevation. No combination of pier size and location used in this analysis complied with the requirements of the zero-rise ordinance at this critical section.

3.1.6 Summary for Case Study Number 1

In summary, the results of the sensitivity analysis for this case study showed that a small variation in the estimate of the hydraulic input parameters could cause the predicted rises to vary by more than 0.01 ft. As discharge values increased, the change in water surface elevation from existing to proposed conditions increased. However, when channel roughness was increased, the change in water surface elevation decreased. When both parameters were varied, the results showed that the water surface elevation computed by the HEC-2 water surface profile model was more sensitive to changes in the roughness coefficient than to changes in discharge. Accurate roughness coefficients are required for accurate predictions of the change in water surface elevation resulting from the inclusion of a bridge. However, as stated previously, the exact roughness coefficients are rarely known and are usually inferred when the model is calibrated to a known discharge. See Table 3.2 for a summary of the results.

Without exact values of input parameters, it appears that it is not possible to model the change in water surface profiles from existing to proposed conditions to within 0.01 ft. using the HEC-2 model. The model should not be used as a tool for assessing conformance to such a tolerance. For Case Study 1, the variation in rise associated with the five values of discharge was 0.05 ft., and the variation associated with the five roughness coefficients was 0.13 ft. Combining the variations associated with discharge and channel roughness individually could produce a variation ranging between 0.02 ft. and 0.16 ft.

Table 3.2. Summary of Results for Case Study 1.

	Value
Drainage Basin Area	104 sq. miles
Slope	0.0006
100-year Peak Flow	11,632 cfs
Standard Deviation of Discharge	1198 cfs
Critical Section	1404
Maximum Change in Water Surface Elevation	0.29 ft
Maximum Range of Variation in Water Surface Elevation Prediction due to Q	0.08 ft
Maximum Range of Variation in Water Surface Elevation Prediction due to Manning's n	0.16 ft
Maximum Range of Estimate of Water Surface due to Pier Size at Mid-Channel per ft. Change in Pier Size	0.13-0.16 ft
Maximum Range of Estimate of Water Surface due to Pier Location for 6 ft. Pier	0.10-0.40 ft
Hydraulic Input Variable of Most Sensitivity	Manning's n
Input Variable of Most Sensitivity	Pier Location

Even though a variation of 0.02 ft. is small, it is still twice the allowable value of 0.01 ft.

These results also showed that bridge design choices are extremely limited when designers are required to adhere to the zero-rise ordinance. Out of the 12 combinations of pier size and location, not one combination of bridge design for a bridge built in the floodplain met the 0.01-ft. requirement of the zero-rise ordinance.

3.2 Case Study No. 2 - Elliott Street Bridge at Cedar River

3.2.1 Case Study Description

The second case study was a King County project to replace the Elliott Bridge, which is approximately 1.5 miles upstream of the SR 169 Cedar River Bridge. Elliott

Bridge was constructed in 1951 and is 204 ft. long with 37-ft. trestles at each end, two 10-ft. lanes, and a cantilevered 3-ft.-wide sidewalk. The existing bridge is narrow and does not provide the vertical clearance required to meet the current highway standards. This bridge also fails to meet the King County Surface Water Management required clearance over the 100-year flood elevation. It is considered to be a safety hazard.

A new four-lane, 400-ft.-long bridge is proposed to accommodate the 22,300 vehicles expected to use this bridge daily in 2010. The new bridge will have two spans, with an intermediate pier on the south bank of the normal low water channel. The placement of this bridge pier is anticipated to increase the upstream flood elevation by approximately 5 inches. Excavation along the north bank will compensate for this upstream rise in flood elevation. If excavation is not undertaken, an additional \$2 to \$4 million to construct a single span bridge would be required to comply with the zero-rise ordinance. The project site layout along the river is as shown in Figure 3.8.

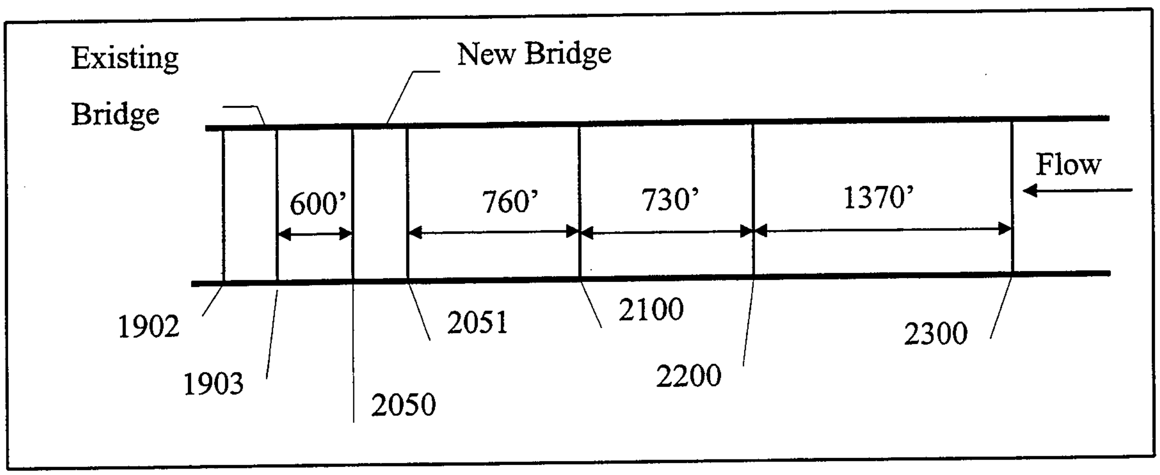


Figure 3.8. Elliott Street Bridge at Cedar River Project Site Layout with Cross-sections. Distances between sections are approximate.

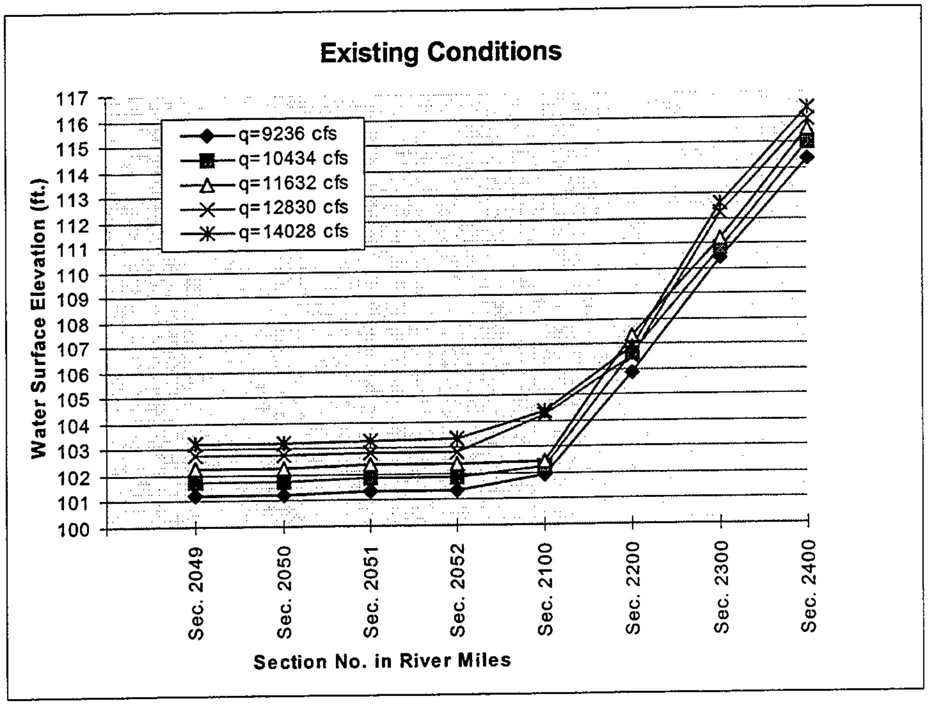
3.2.2 Change in Water Surface Elevation for Varying Discharges

Because Case Studies 1 and 2 were near each other along the Cedar River, the 100-year peak flow was the same for both bridges. As for Case 1, the 100-year peak flow was estimated at 11,632 cfs, with a standard deviation of 1198 cfs.

The water surface profiles computed by HEC-2 for both existing and proposed conditions are shown in Figure 3.9. Each line represents the water surface profile computed for a specific discharge. Once again, Figure 3.9 shows that an increase in the discharge caused the water surface profile to rise for both existing and proposed conditions.

The majority of the model runs followed the trends established in the Cedar River Bridge case study. These results showed that the change in water surface due to the construction of a bridge in the floodplain increased as the discharge increased. (See Figure 3.10.) However, for a discharge of 12,830 cfs, the maximum changes in water surface elevation from existing to proposed conditions occurred at different locations than for other discharges. In a sense, the water surface profile at this discharge can be described as out of phase with the other profiles.

To determine the cause of this phase shift, another set of runs was performed in the same manner as the first set of runs. The five new discharge values, closely spaced about the 12,830 cfs value, were 12,780 cfs, 12,830 cfs, 12,880 cfs, 12,930 cfs, and 12,980 cfs. (See Figure 3.11.) The results of those runs were similar to those of the previous runs, shown in Figure 3.10. However, the result for a discharge of 12,830 cfs was still out of phase.



SECNO	w/o bridge q=9236 cfs	w/bridge q=9236 cfs	Elev. Diff.	w/o bridge q=10434 c	w/bridge q=10434 c	Elev. Diff.
Sec. 2049	101.21	101.21	0	101.75	101.75	0
Sec. 2050	101.23	101.3	0.07	101.76	101.8	0.04
Sec. 2051	101.33	101.34	0.01	101.86	101.85	-0.01
Sec. 2052	101.35	101.33	-0.02	101.87	101.84	-0.03
Sec. 2100	101.92	101.91	-0.01	102.25	102.23	-0.02
Sec. 2200	105.83	105.77	-0.06	106.63	106.57	-0.06
Sec. 2300	110.42	110.67	0.25	110.83	111.25	0.42
Sec. 2400	114.39	114.35	-0.04	114.98	114.91	-0.07
Sec. 2500	120.77	120.78	0.01	121.13	121.15	0.02

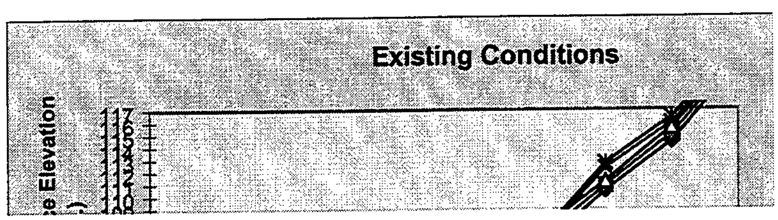


Figure 3.9. Water Surface Profiles for Varying Discharge: Existing and Proposed Conditions. Case 2.
Section Nos. refer to cross-sections along the river. This figure compares the effects that varying discharge values have on the water surface profiles for existing and proposed conditions.

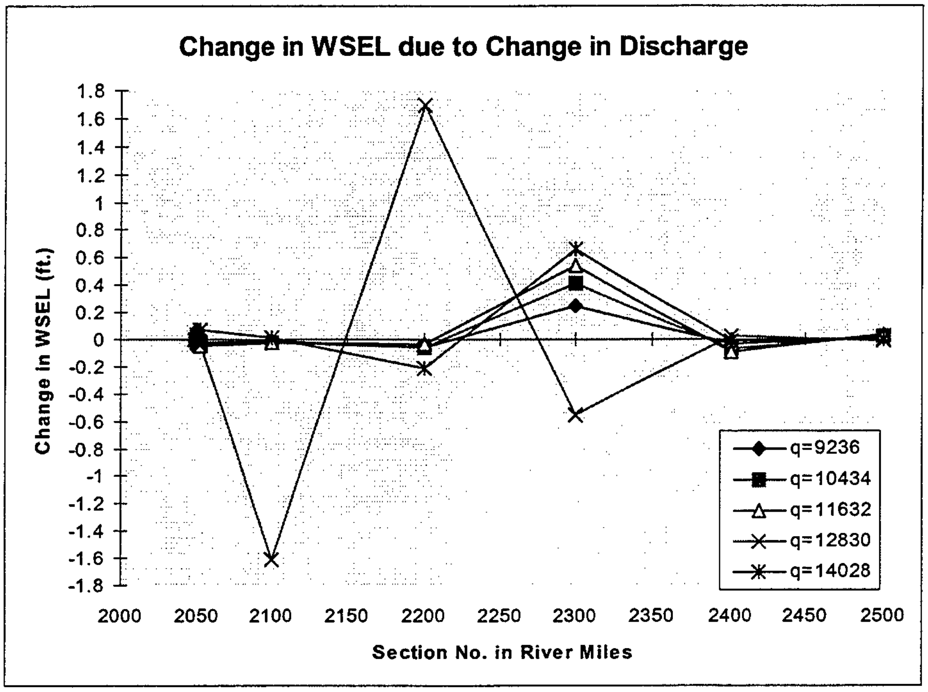


Figure 3.10. Change in Water Surface Elevation (WSEL) for Varying Discharge. Case 2. Section Nos. refer to cross-sections along the river. The change in WSEL is a comparison of WSEL with and without the new bridge for a given discharge.

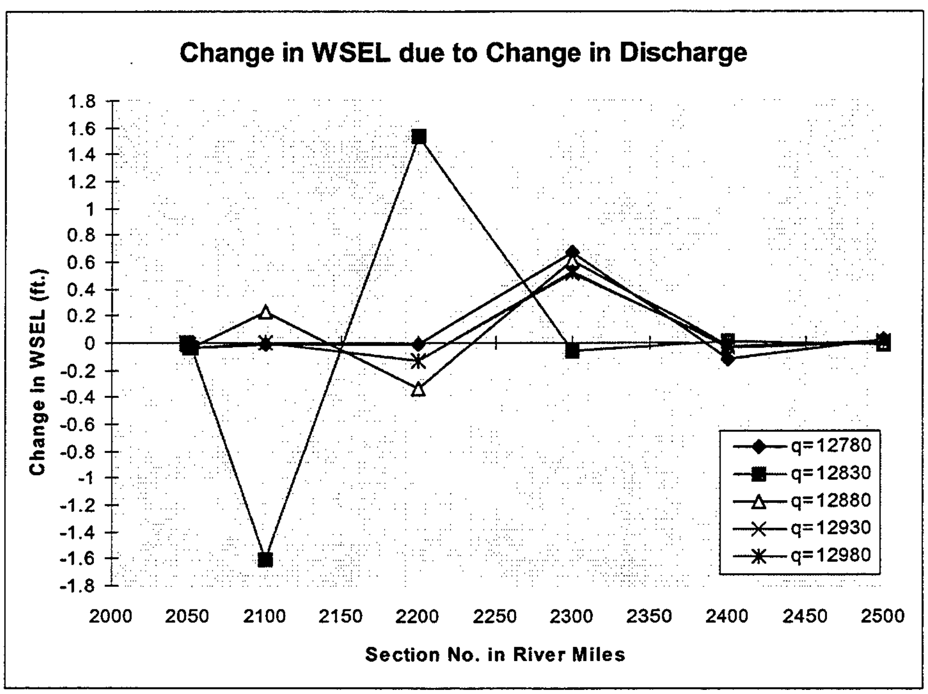


Figure 3.11. Change in Water Surface Elevation (WSEL) for Small Variation in Discharge. Case 2.

A comparison of the water surface profiles at Section 2100 for existing and proposed conditions may help to explain this shift. (See Table 3.3.) Section 2100 was the point at which the shift in phase began. If there was any instability in the model results at this location, all calculations upstream would be affected because the flow is subcritical and HEC-2 uses the standard step method. For a discharge of 12,780 cfs, the water surface elevations for existing and proposed conditions were 102.62 ft. and 102.61 ft., respectively. Similarly, for Q = 12,880 cfs, the water surface elevations for existing and proposed conditions were 104.02 ft. and 104.25 ft., respectively. However, for a discharge of 12,830 cfs, there was a large difference between the water surface elevations for existing conditions (104.23 ft.) and proposed conditions (102.63 ft.). This difference can be attributed to the way in which effective flow areas are used by HEC-2 for each condition.

Table 3.3. Comparison of Existing and Proposed Water Surface Elevations at Section 2100 for Three Discharges.

Discharge (cfs)	Water Surface Elevation Existing Conditions (ft)	Water Surface Proposed Conditions (ft)
12,780	102.62	102.61
12,830	104.23	102.62
12,880	104.02	104.25

HEC-2 allows the user to specify the flow area used in the water surface profile computations. The default option in HEC-2 uses the entire area below the water surface elevation. Under this option, HEC-2 assumes that even discontinuous areas in the overbanks that have a ground elevation lower than the computed water surface are capable of conveying flow. An other option is to confine the flow to the main channel. The user can specify in the HEC-2 input file that only the flow area in the main channel may be used in the computations until the water surface elevation exceeds the elevations of the left and right overbank stations. These stations and elevations delineate the point where the overbanks begin and the main channel ends and are specified by the user.

Once these elevations have been exceeded, HEC-2 computes the water surface elevations by using the default option. This effective flow area can be constrained even more with the use of encroachments. If the river overflows into its overbanks, the user can limit the extent of the overbank by specifying another station and elevation landward from the left and right overbank stations. This second limit is referred to as the encroachment station and elevation. HEC-2 will not include the flow area outside of this station until the water surface elevation exceeds the top elevation of the encroachment.

At Section 2100, the effective flow area for this case study was confined to the main channel. If the river exceeded the elevation of the left overbank station, the flow area in the left overbank would be restricted to a point 550 ft. landward of the left overbank station. If the river exceeded the right overbank, the entire flow area in the right overbank would be used in the computations.

It appears that the flow reacted differently in the existing and proposed conditions because of this constraint, which, in turn, caused the large change in water surface elevation. If the flow area is confined, the flow can react in two ways. HEC-2 confines the flow by reducing the available width of the overbank. For a constant discharge, if the velocity is kept constant, then the area must remain constant. With a reduced width, the depth of flow must increase to maintain this constant area. This is what occurred for the model for existing conditions at the 12,830 cfs discharge. When the flow was confined by the left bank encroachment, the flow depth increased. The river had to build up head to cover the losses experienced downstream of the constraint. When sufficient depth was reached to overtop the overbanks, the flow utilized the overbanks and channel for passing the discharge. The velocity was reduced because the area was then larger.

The second way in which the flow can react when its flow area is confined occurred under proposed conditions, after a bridge had been constructed within the floodplain downstream of Section 2100. Inclusion of the bridge downstream caused an increase in depth at Section 2100. The river had sufficient energy to cover the losses downstream of this constrained flow. The flow depth for this case never exceeded the overbank elevation. If the depth of flow does not change significantly, then the flow area has been reduced because of the reduction in floodplain width. In this case, the flow velocity increased to maintain a constant discharge.

Additional runs verified this reasoning. HEC-2 output showed that for existing conditions and a discharge of 12,830 cfs, the velocity head was reduced and the flow depth increased when the flow area was constrained. For proposed conditions, the velocity head increased and the water surface dropped. If the constraint on the left overbank has been removed, the change in water surface profile for a discharge of 12,830 cfs would have been in phase with the other discharges.

This analysis implies that a comparison of existing to proposed conditions may not accurately portray the effects of building a bridge in the floodplain. A comparison of the energy grade line may be a better comparison since it takes into account the changes in velocity head, as well as the changes in water surface elevation. However, to be consistent with the first case study and the zero-rise ordinance, the change in water surface elevation was used as an indication of the effects that bridges in the floodplain have on the water surface. The water surface elevations determined from a discharge of 12,830 cfs and a channel roughness of 0.040 were not included in the interpretation of results and general trends for this case study. This should not compromise the results because this was only one point out of 25.

The greatest backwater generally occurs at a distance equivalent to one bridge width upstream of the bridge (USACOE, 1995). However, for this case study, the channel geometry dictated that the location of the greatest backwater would occur approximately 2,800 ft. upstream of the new Elliott Bridge. Immediately upstream of the new bridge, the floodplain was wide and shallow. Flow was not constrained to remain in the main channel. Between Sections 2100 and 2300, the floodplain narrowed because of topography, and the flow was confined to the main channel. Except for the case mentioned above, the maximum backwater always occurred at the critical Section 2300.

The backwater created by constructing a bridge in the floodplain for the lowest discharge modeled was 0.25 ft. at the critical section. At the same section, the inclusion of a bridge at the highest discharge caused a 0.66-ft. increase in water surface elevation.

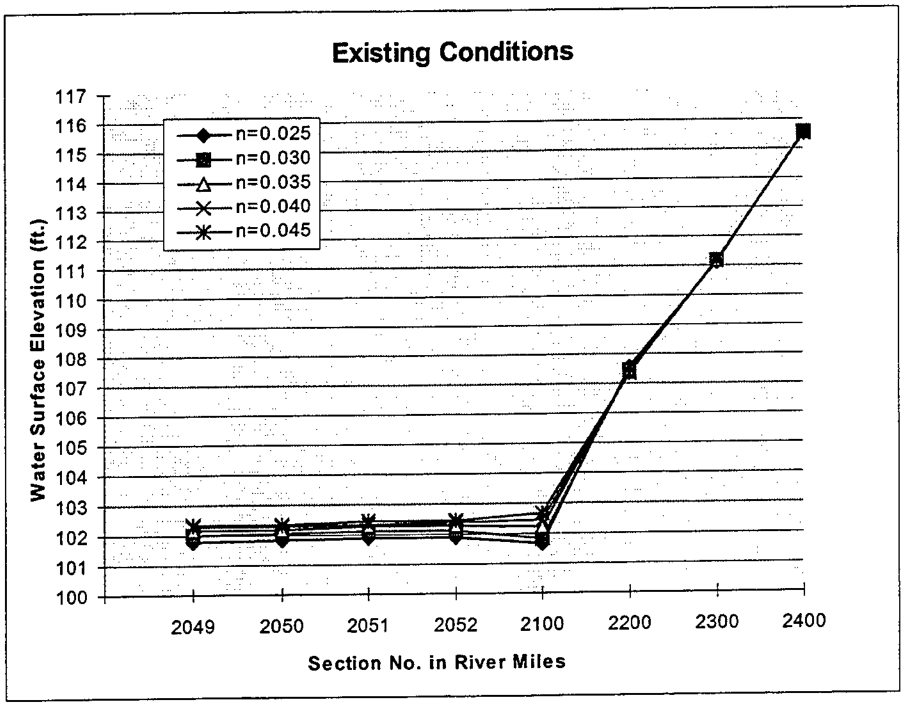
These results show that for this case study, the possible outcomes provided by HEC-2, when five reasonable values of discharge were used, fell within a band of 0.41 ft. Even if the estimates of the 100-year peak flow were limited to one standard deviation about the mean, the expanse of variation would still be as much as 0.19 ft., far exceeding the 0.01-ft. criterion. A small variation in the estimate of the 100-year discharge could cause the predicted rises to vary by more than 0.01 ft. of rise.

3.2.3 Change in Water Surface Elevation for Varying Roughness Coefficients

As in Case Study 1, the effect of Manning's n on the computed water surface elevation was determined by holding the discharge constant while values for Manning's n were varied. Once again, the computed water surface profiles were graphed for both existing and proposed conditions. (See Figure 3.12.) As in the previous case study, the water surface elevations increased as the roughness value increased. Increasing Manning's roughness coefficient by 0.005 caused, on the average, an increase in the water surface elevation of approximately 0.3 ft. for this case study.

Similarly to the first case study, Figure 3.13 indicates that the change in water surface elevation resulting from the inclusion of a bridge decreased as the roughness coefficient increased. The effects of the bridge piers were felt less as the channel roughness increased.

For this case study, the results showed that the possible outcomes using five reasonable values of channel roughness yielded a maximum rise in water surface elevation ranging from 0.55 ft. to 0.57 ft as a result of the new bridge. A small variation of 0.02 ft. in the predicted amount of rise could be expected when the exact roughness coefficient at the bridge was not known.



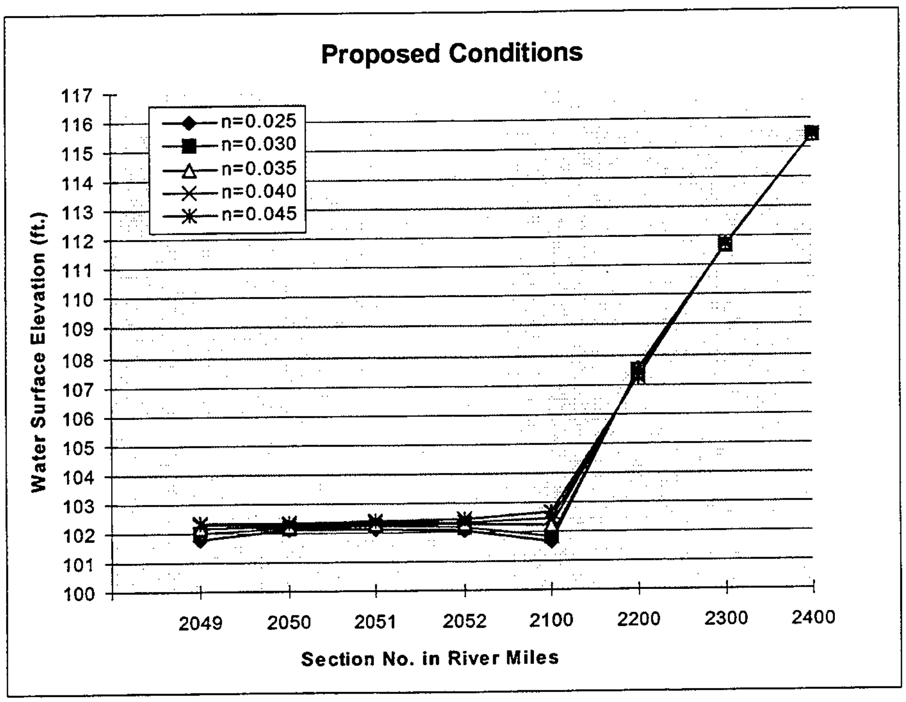


Figure 3.12. Water Surface Profiles for Varying Manning's Roughness: Existing and Proposed Conditions. Case 2.
Section Nos. refer to cross-sections along the river. This figure compares the effects that various roughness values of Manning's n have on the water surface profiles for existing and proposed conditions and a constant discharge of 11,632 cfs.

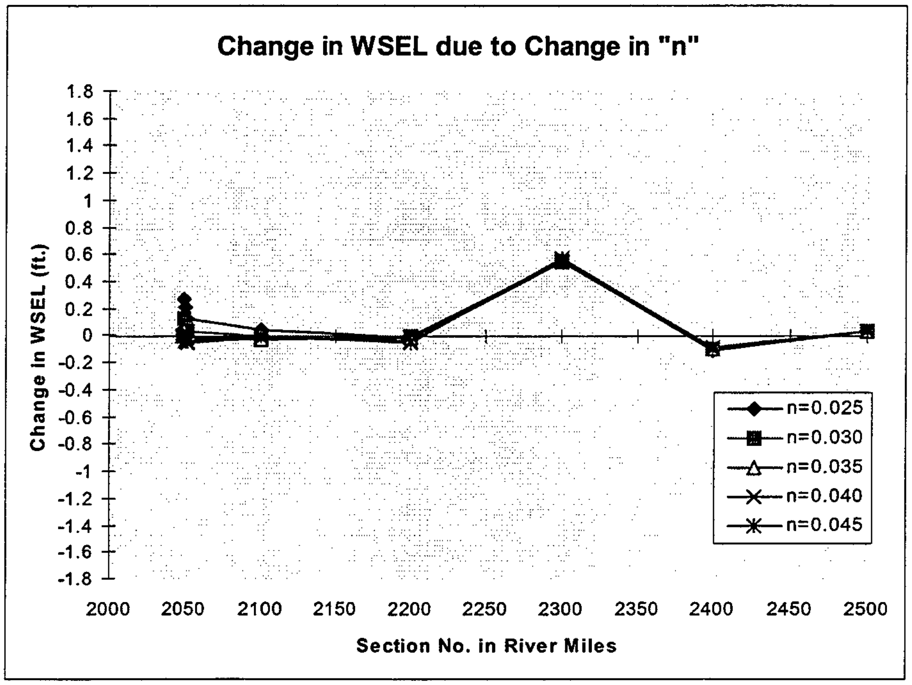


Figure 3.13. Change in Water Surface Elevation (WSEL) for Varying Manning's Roughness. Case 2.
Section Nos. refer to cross-sections along the river. The change in WSEL is a comparison of WSEL with and without the new bridge for a given roughness coefficient.

3.2.4 Change in Water Surface Elevation for Combined Discharge and Manning's n Variation

For the 25 runs combining discharge and roughness coefficients, the change in water surface elevation between the proposed and existing conditions was calculated. Table 3.4 displays the range of variation in backwater rise for changes in discharge and roughness coefficient.

For Case Study Number 2, the rise of water surface elevation, given a reasonable estimate for discharge and channel roughness, was between 0.25 ft. and 0.67 ft. If the channel roughness was known exactly and the estimate of the discharge was within two standard deviations of the 100-year peak flow, a variation of as much as 0.42 ft. could be expected when the change in water surface elevation was predicted for this case study.

Table 3.4. Change in Water Surface Elevation (in feet) for a Given Discharge and Roughness Coefficient. Case Study 2.

Discharge	n=.025	n=.030	n=.035	n=.040	n=.045	Range of Variation*
9,236 cfs	0.27	0.25	0.25	0.25	0.25	0.02
10,434 cfs	0.44	0.43	0.42	0.42	0.42	0.02
11,632 cfs	0.56	0.55	0.56	0.55	0.57	0.02
12,830 cfs	0.59	0.67	0.67	-0.06	0.60	0.08**
14,028 cfs	0.66	0.64	0.66	0.66	0.66	0.02
Range of Variation*	0.39	0.42	0.42	0.41	0.41	

***Range of variation is defined as the difference between the largest and smallest possible outcome.**
****The change in water surface elevation for Q=12,830 and n=0.040 was not included.**

If the estimate of the 100-year peak flow was limited to one standard deviation about the mean, the variation would still be as much as 0.25 ft. Similarly, for a fixed discharge, the variation of bridge related rise due to uncertainty in roughness would range from 0.02 ft. to 0.10 ft. For example, if the engineer knew the exact value of the 100-year discharge but did not have an exact value for channel roughness, a variation of as much as 0.10 ft. could be expected when change in water surface elevation was predicted.

As shown in Figure 3.14, the predicted change in water surface between proposed and existing conditions was not very sensitive to the Manning coefficient at the bridge, especially for discharge values below 12,000 cfs. The water surface difference was more sensitive to changes in discharge rate, with the greatest sensitivity to changes in discharge rate below 12,000 cfs.

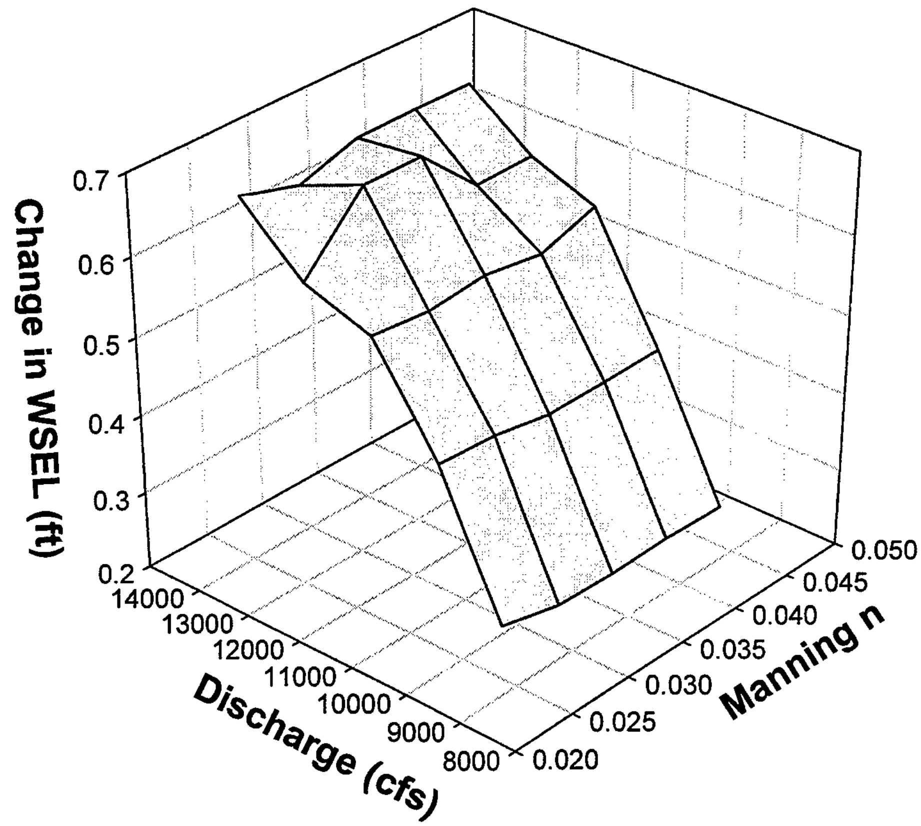


Figure 3.14. Change in Water Surface Elevation vs. Manning's n and Discharge. Case 2.

3.2.5 Change in Water Surface Elevation for Varying Pier Size and Location

Combinations of pier size and location were also reviewed in this sensitivity analysis. The pier size investigated in this analysis ranged from 4 to 10 ft. in diameter. Piers were located either in the left, right, or center of the river channel. This analysis was completed to identify a possible bridge design that would comply with the zero-rise ordinance requirements.

The results showed that locating a 4-ft. diameter bridge pier at any location in the channel would cause the least change in the computed water surface elevation at the critical section, Section 2300. (See Figure 3.15.) However, this particular design still exceeded the allowable 0.01 ft. of increase in the computed water surface elevation.

Once again, no combination of pier size and location used in this analysis produced less than 0.01 ft. rise at the critical section. Therefore, designs for bridges using these pier parameters would not comply with the requirements of the zero-rise ordinance.

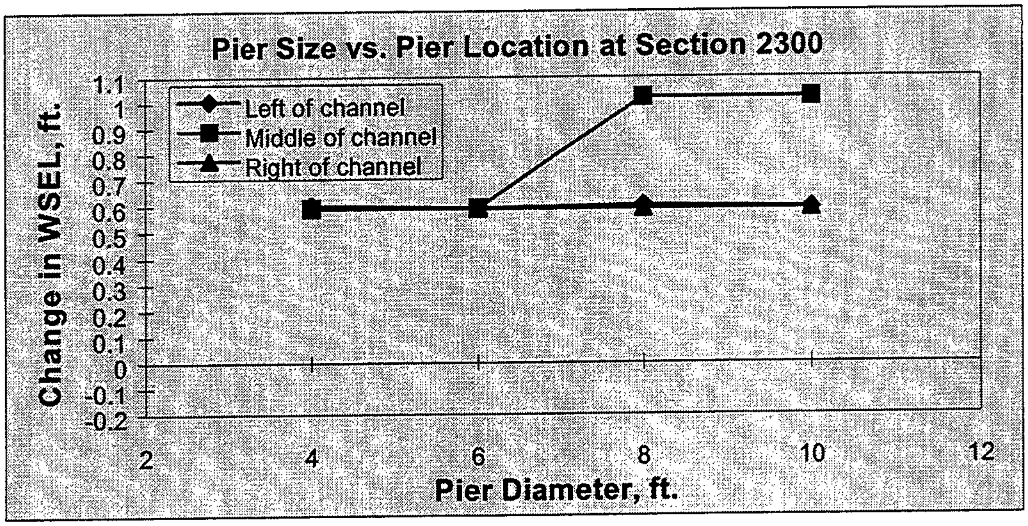


Figure 3.15. Change in WSEL for Varying Pier Size & Location. Case 2. Section Nos. refer to cross-sections along the river. The change in WSEL is a comparison of WSEL with and without the new bridge.

3.2.6 Summary for Case Study Number 2

The results of the sensitivity analyses for this case study showed that a small variation in the estimate of the hydraulic parameters could cause the predicted rises to vary by more than 0.01 ft. These results showed trends similar to those found in the Cedar River Bridge case study. As discharge values increased, the change in water surface elevation from existing to proposed conditions increased. When channel roughness was increased, the change in water surface elevation decreased.

When both parameters were varied, the results differed from the previous case study. For Case Study 2, the water surface elevation computed by the HEC-2 water surface profile model was more sensitive to changes in discharge than in the channel roughness. Accurate predictions of the 100-year peak flow were required for accurate prediction of the change in water surface elevation that would result from the inclusion of a bridge at this location.

For this case, the estimated water surface elevation varied by up to 0.42 ft. for changes in the 100-year peak flow prediction and by up to 0.08 ft. with various Manning's n values. Combining the uncertainties associated with discharge and channel roughness, the change in predicted water surface elevation ranged from 0.02 ft. to 0.42 ft. Even though a variation of 0.02 ft. is small, it is still twice the 0.01 ft. criterion. See Table 3.4 for a summary of the results.

Table 3.4. Summary of Results for Case Study Number 2.

	Value
Drainage Basin Area	**104 sq. miles**
Slope	**0.0006**
100-year Peak Flow	**11,632 cfs**
Standard Deviation of Discharge	**1198 cfs**
Critical Section	**2300**
Maximum Change in Water Surface Elevation	**0.67 ft**
Maximum Range of Variation in Water Surface Elevation Prediction due to Q	**0.42 ft**
Maximum Range of Variation in Water Surface Elevation Prediction due to Manning's n	**0.08 ft**
Maximum Range of Estimate of Water Surface due to Pier Size at Mid-Channel per ft. Change in Pier Size	**0.10-0.15 ft**
Maximum Range of Estimate of Water Surface due to Pier Location for 6 ft. Pier	**0.60 ft**
Hydraulic Input Variable of Most Sensitivity	**100-year Discharge**
Input Variable of Most Sensitivity	**Pier Location**

In addition to variations on the basis of reasonable estimates of input parameters, this case study showed that modeling limitations can cause questionable results. It is possible to choose a particular option in the model that will produce unlikely results.

Because HEC-2 uses a standard step method, the effects of these limitations are carried upstream as the calculations for subcritical flow are carried out.

Similarly to the first case study, this sensitivity analysis showed the bridge design to be limited severely by the zero-rise ordinance. Out of the 12 combinations of pier size and location, not one combination of bridge design parameters created a bridge that would meet the 0.01-ft. requirement of the zero-rise ordinance.

3.3 Case Study No. 3 - Meadowbrook Bridge at Snoqualmie River

3.3.1 Case Study Description

The third case study was a project that involved modifying the Meadowbrook Bridge located on 396th Avenue SE and 82nd St. at Meadowbrook over Snoqualmie River near the City of Snoqualmie, Wash. Meadowbrook Bridge was constructed in 1921 and is 373 ft. long, with two 9 1/2-foot lanes and a 4-foot sidewalk. The existing bridge is narrow and does not provide the King County Surface Water Management required clearance over the 100-year flood.

For this project, the new bridge design had not yet be determined, requiring the researchers to make assumptions e about how the bridge would likely be modified. Because the bridge does not meet the clearance requirements over the 100-year flood, this study assumed that the low bridge chord would be raised by 3 feet. To support the extra force created by raising the bridge, the existing piers would have to be widened, and the abutments would have to be expanded. The increased width of the two existing bridge piers was assumed to be 2 feet. Additionally, the channel width would be decreased by 5 percent, with a 2.5 percent decrease on each side of the channel. For the HEC-2 model, these modifications were assumed to be made at the same location as the existing bridge. This was different from Cases 1 and 2, in which new bridges were installed alongside the old ones. Figure 3.16 shows the project site layout along Snoqualmie River.

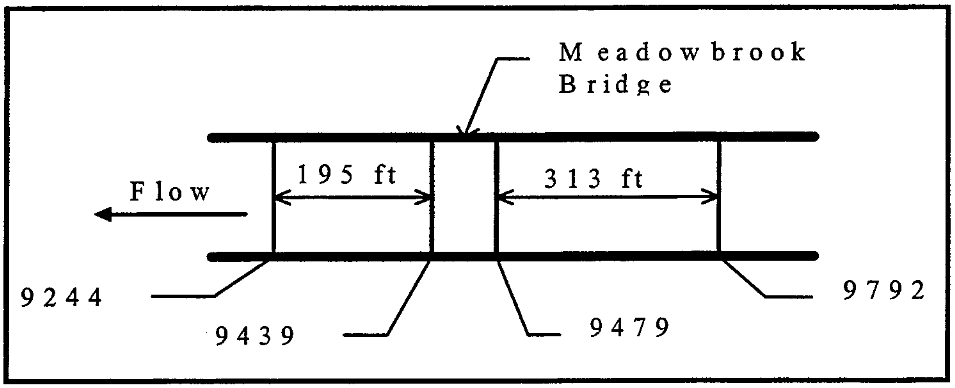


Figure 3.16. Meadowbrook Bridge at Snoqualmie River Project Site Layout with Cross-sections.

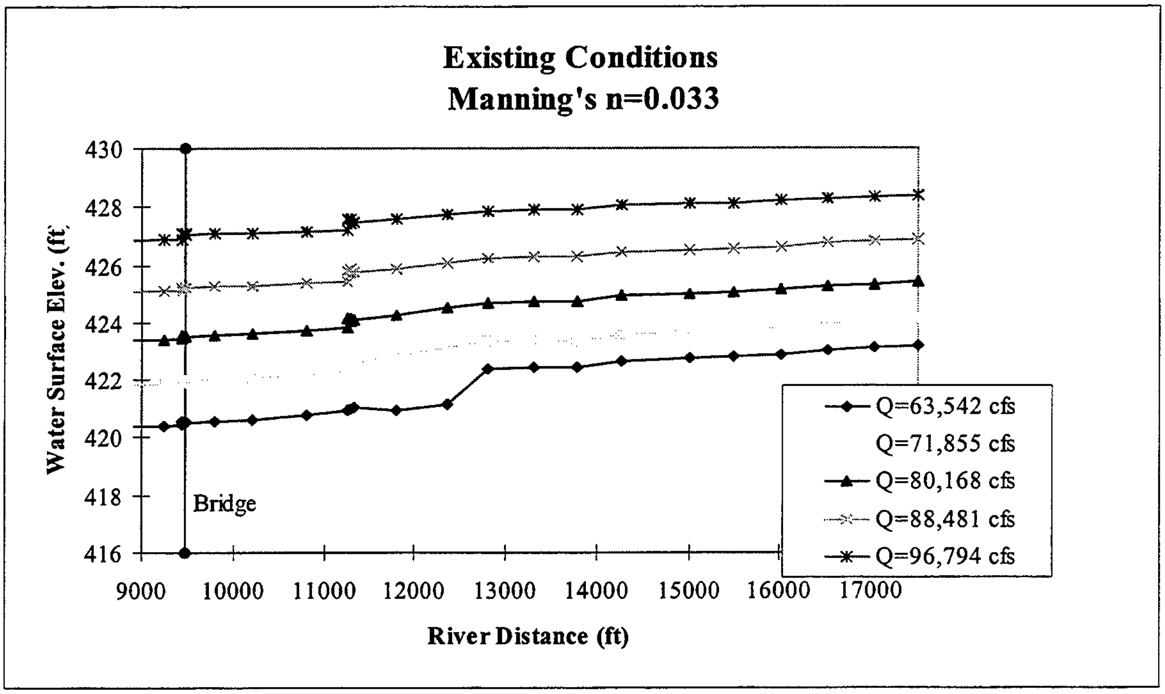
3.3.2 Change in Water Surface Elevations for Varying Discharge

As in the other case studies, five different discharge values were modeled in HEC-2. The gage used to predict the 100-year peak flow was located on the Snoqualmie River near Snoqualmie, Wash., and had a drainage basin of 375 square miles. The 100-year peak flow was estimated as 80,168 cfs with a standard deviation of 8,313 cfs. However, when the highest discharge of 96,794 cfs was modeled, the existing bridge was overtopped. Therefore, this value was not included in the analysis. Figure 3.17 presents the water surface profiles for existing and proposed conditions for all five discharges, and Figure 3.18 presents the change in water surface elevations for the lowest four discharge values.

For this case study, the critical section appeared at river distance 9792 feet, 313 feet upstream of the Meadowbrook Bridge. Theoretically actual maximum backwater should occur closer to the face of the bridge. However, the cross-section at river distance 9792 was the closest section to the upstream face of the bridge. This meant that the actual maximum change in water surface elevation would be greater than what the model predicted.

The main difference between this case study and the previous two case studies was the fact that the change in water surface elevation did not increase as the discharge increased as a result of modifications to the bridge and channel. Instead, the largest modeled discharge rate of 88,481 cfs created the smallest net change in water surface elevation. The discharge rates of 71,855 cfs and 80,168 cfs produced the greatest changes in water surface elevation. In this case study, the bridge did not constrict the flow. This allowed for high volumes of overflow. Because the ground profile of the floodplain contains natural contours, the cross-sectional area did not increase at a constant rate for depth and width.

The backwater created by modifying the existing bridge in the floodplain for the discharges modeled ranged from 0.07 feet to 0.09 feet. These results showed that for this case study, the possible outcomes when four reasonable values of discharge were used fell within a band of 0.02 ft.



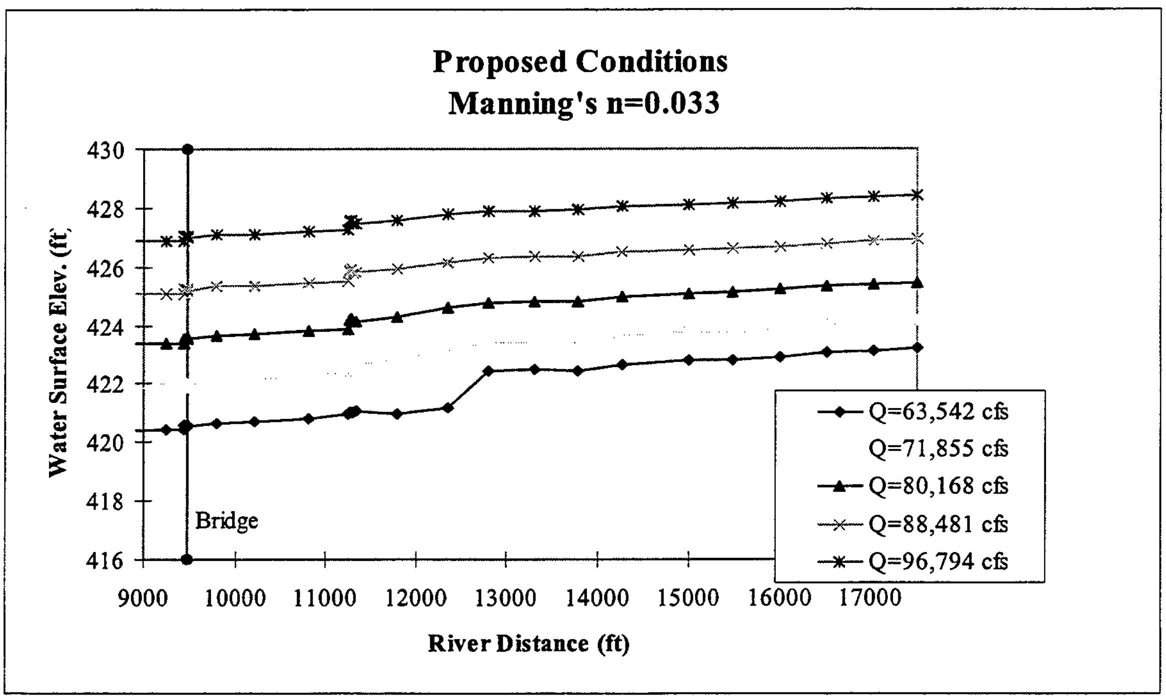


Figure 3.17. Water Surface Profiles for Varying Discharge: Existing and Proposed Conditions. Case 3.
This figure compares the effects that various discharge values have on the water surface profiles for existing and proposed conditions. Proposed conditions include bridge modification.

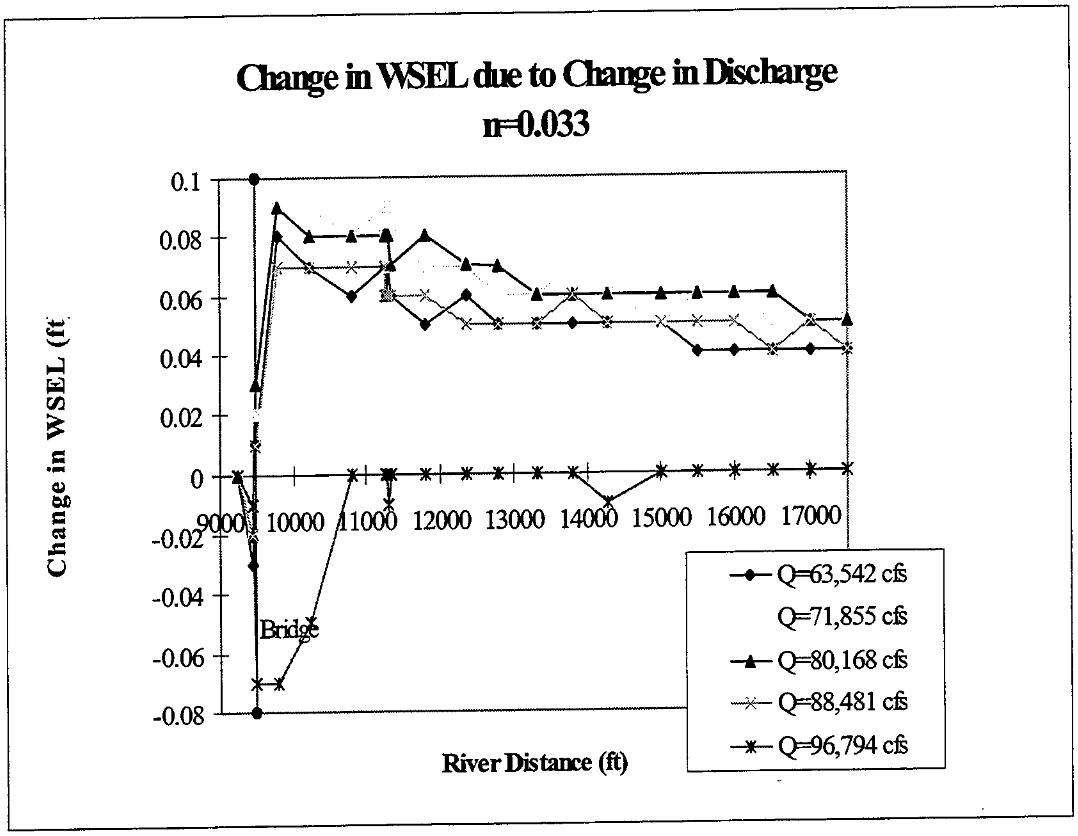
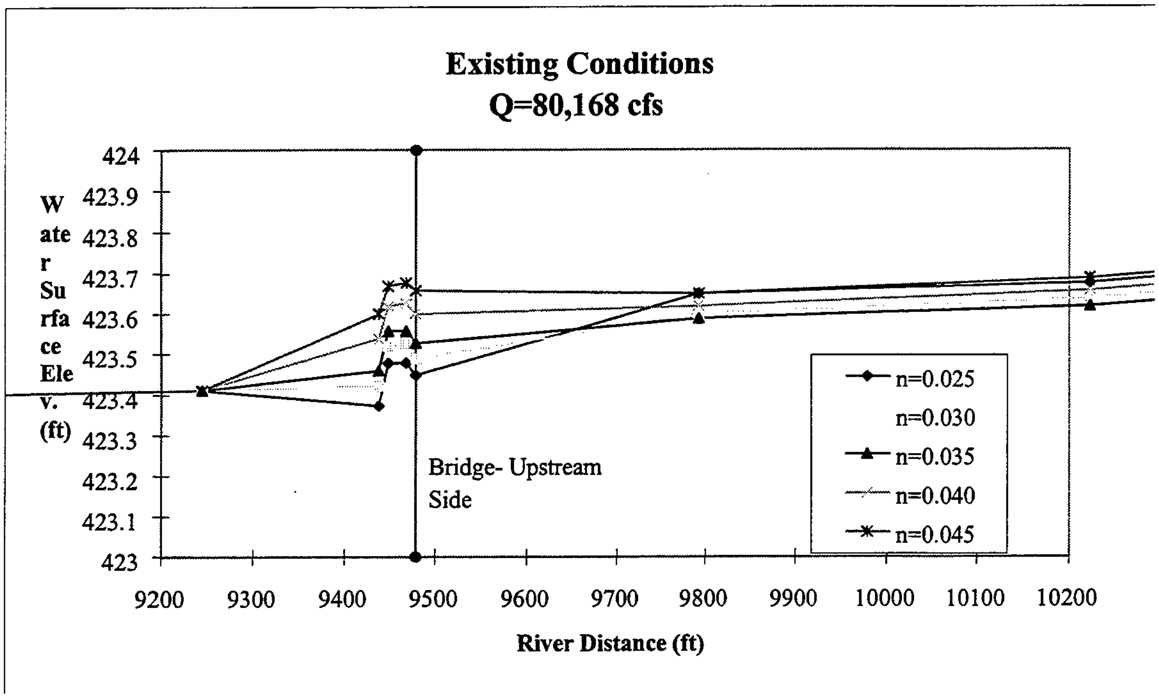


Figure 3.18. Change in Water Surface Elevation (WSEL) for Varying Discharge. Case 3. River distance refers to cross-sections along the river. The change in WSEL is a comparison of WSEL with and without the bridge modifications for a given discharge.

3.3.3 Change in Water Surface Elevations for Varying Roughness Coefficients

The effect of Manning's n on the computed water surface elevation was determined by varying the values for roughness while the discharge was held constant. Figure 3.19 shows the computed water surface profiles for both the existing and proposed conditions. The water surface elevation was expected to increase as the roughness value increased. However, as shown in Figure 3.19, this trend only held true between river distance 9200 feet and 9600 feet. Above the river distance of 9600 feet, the lowest roughness coefficient caused the greatest change in water surface elevation. This discrepancy occurred because the roughness coefficient was varied only for the main channel. The overbank area had a constant roughness coefficient of 0.015.



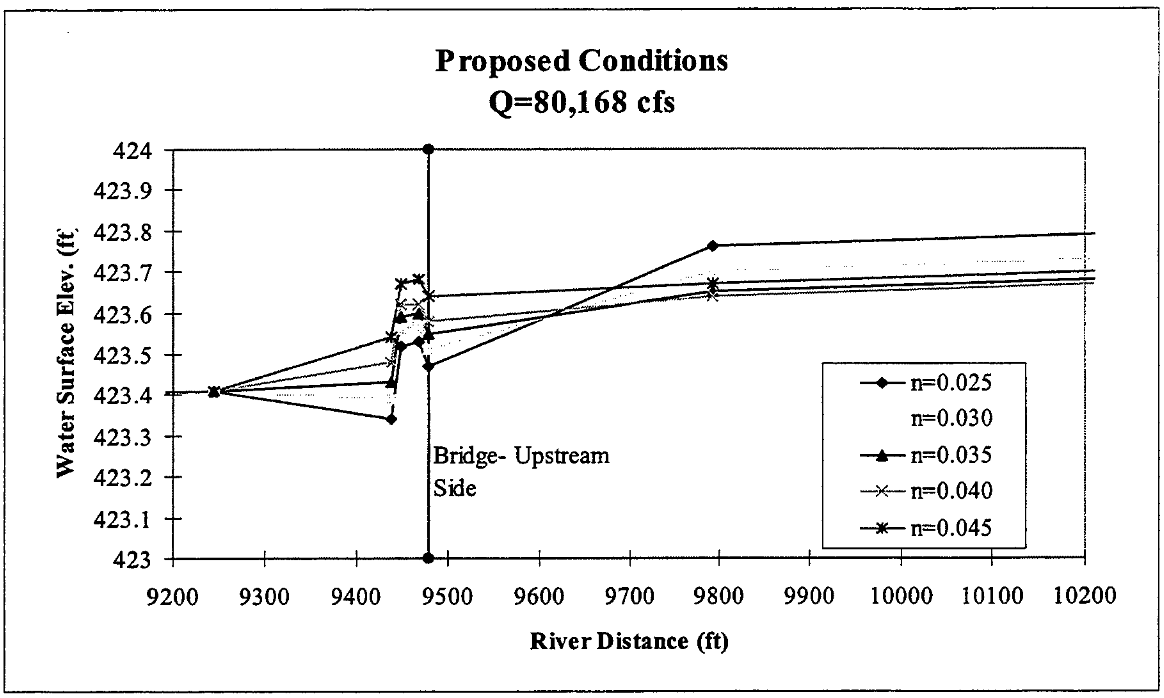


Figure 3.19. Water Surface Profiles for Varying Manning's Roughness: Existing and Proposed Conditions. Case 3.
River Distance refers to cross-sections along the river. This figure compares the effects that various values of Manning's n have on the water surface profiles for existing and proposed conditions and a constant discharge of 80,168 cfs.

The same discharge with different channel roughness produced varying amounts of flow through the banks and channel. The velocity of the flow in the channel decreased for increasing roughness. Usually this would imply that the depth of flow would increase. However, for this case study, the floodplain was wide, allowing wide overbank flow. Because the roughness remained constant in the overbanks and changes for the main channel, the water surface elevation behaved differently for various roughness coefficients. Thus, the lowest roughness coefficient caused the greatest change in water surface elevation between existing and proposed conditions.

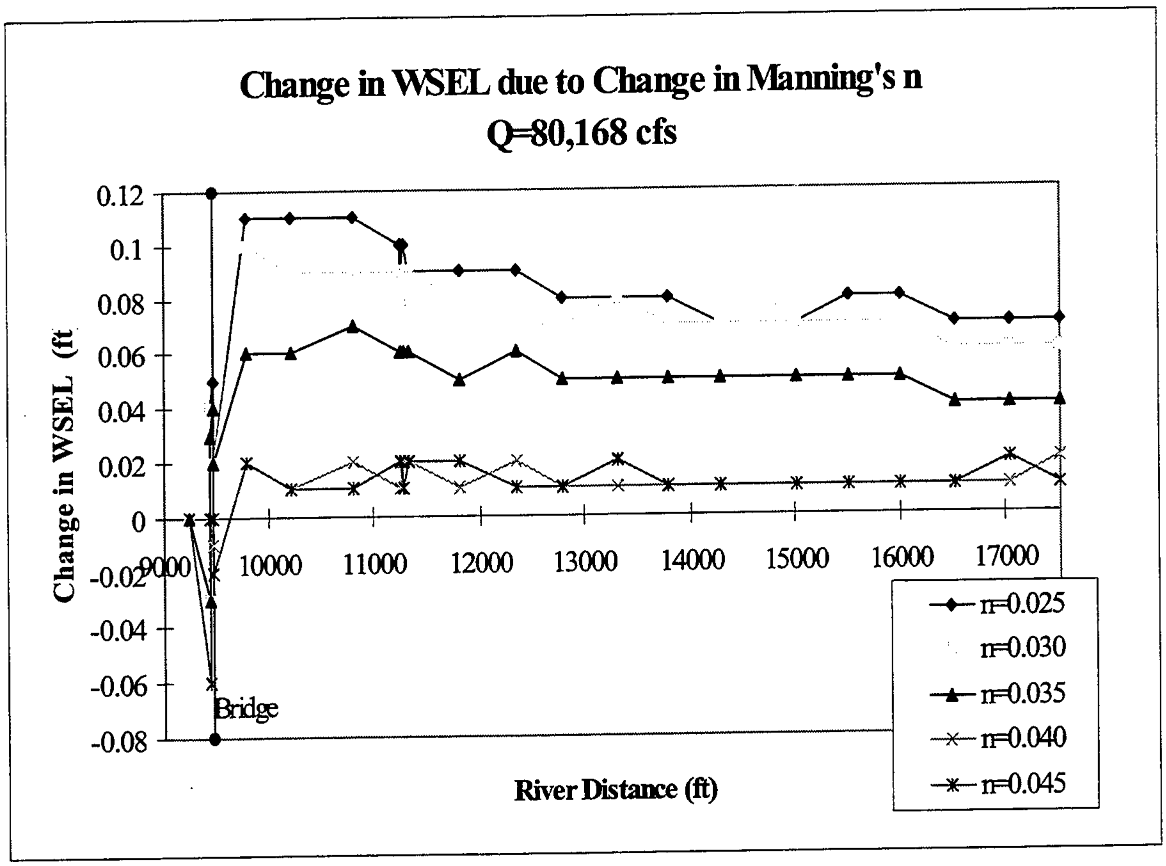


Figure 3.23. Change in Water Surface Elevation (WSEL) for Varying Manning's Roughness. Case 3.
River Distance refers to cross-sections along the river. The change in WSEL is a comparison of WSEL with and without the bridge modifications for a given roughness coefficient.

As observed in the previous two case studies, the change in water surface elevation resulting from modifications to the existing bridge decreased as roughness increased. Refer to Figure 3.20. For this case study, the variation in change in water surface for five reasonable values of channel roughness ranged from 0.02 ft for the highest Manning coefficient to 0.11 ft for the lowest roughness. A variation of as much as 0.09 ft. could be expected if exact roughness coefficients were not known. This error far exceeds the 0.01-ft. criterion.

3.3.4 Change in Water Surface Elevation for Combined Discharge and Manning's n Variation

For this case study, the sensitivity analysis consisted of 20 runs for both the existing and proposed conditions. The 20 runs were produced by combining the four discharge rates with the five Manning's n values. As in the previous two case studies, the change in water surface elevation between the proposed and existing conditions was calculated. Table 3.6 shows the range of variation in backwater rise for reasonable estimates of the 100-year peak flow and Manning's roughness coefficient.

Table 3.6. Change in Water Surface Elevation (in feet) for a Given Discharge and Roughness Coefficient.

Discharge	n=.025	n=.030	n=.035	n=.040	n=.045	Range of Variation*
63,542 cfs	0.11	0.09	0.04	0.02	0.02	0.09
71,855 cfs	0.11	0.11	0.06	0.02	0.02	0.09
80,168 cfs	0.11	0.10	0.06	0.02	0.02	0.09
88,481 cfs	0.09	0.07	0.04	0.00	-0.01	0.10
Range of Variation*	0.02	0.04	0.02	0.02	0.03	

*Range of Variation is defined as the difference between the largest and smallest possible outcome.

Figure 3.21 shows the variation in the change of water surface elevation versus the Manning's n values and 100-year peak flow values. The predicted change in water surface between proposed and existing conditions was most sensitive to the Manning coefficient at the bridge, especially for the highest discharge value. The range of rise estimates for a fixed roughness coefficient varied between 0.02 ft. and 0.04 ft. This is 100 percent to 300 percent over the desired accuracy of 0.01 ft. Similarly, for a fixed discharge, the value for rise varied anywhere from between 0.09 ft. and 0.10 ft. For Case Study 3, that the range of possible outcomes modeled by HEC-2, given a reasonable estimate for discharge and channel roughness, far exceeded the 0.01-ft. criterion.

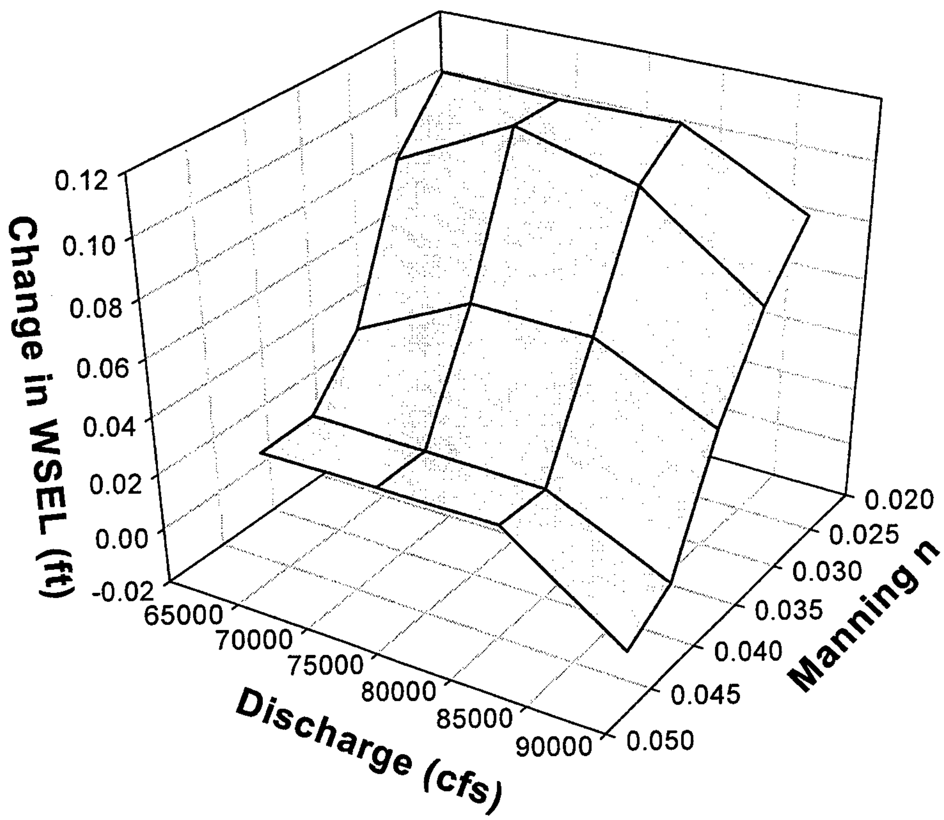


Figure 3.21. Change in Water Surface Elevation vs. Manning's n and Discharge. Case 3.

3.3.5 Summary for Case Study Number 3

As in the previous case studies, variations in the estimation of hydraulic input parameters caused the predicted water surface to vary by more than 0.01 feet. As discharge values were increased, the change in water surface elevation had a variable response. However, when channel roughness increased, the change in water surface elevation decreased. When both parameters were varied, the water surface elevation computed by the HEC-2 water surface profile model was more sensitive to changes in the roughness coefficient than to those in discharge. See Table 3.7 for a summary of the results.

Without exact values of input parameters, it would not be feasible to model the change in water surface profiles from existing to proposed conditions to within 0.01 ft. This variation associated with the four discharge values was 0.02 ft., and the variation associated with the five roughness coefficients was 0.09 ft. Combining the parameters of discharge and channel roughness caused a variation in the water surface elevation ranging from 0.02 ft. to 0.10 ft.

Table 3.7. Summary of Results for Case Study Number 3

	Value
Drainage Basin Area	375 sq. miles
Slope	0.006
Mean 100-year Discharge	80,168 cfs
Standard Deviation of Discharge	8313 cfs
Critical Section	9792
Maximum Change in Water Surface Elevation	0.11 ft
Maximum Range of Error in Water Surface Elevation Prediction due to Q	0.04 ft
Maximum Range of Error in Water Surface Elevation Prediction due to Manning's n	0.10 ft
Input Variable of Most Sensitivity	Manning's n

3.4 Case Study No. 4 - Fall City Bridge at Raging River

3.4.1 Case Study Description

The project of interest for this case study was Raging River Bridge #234A, which crosses over the Raging River near Fall City, Washington. The existing bridge was built in 1919 and reconstructed in 1937. It has two 12-foot lanes, no shoulders, and a 4-foot sidewalk on the west side. The main span is 97-feet long and is connected by a 17-foot transition span to four 22-foot-long concrete approach spans for a total length of 202 feet. The main pier and all four approach pile bents are within the FEMA 100-year floodway.

The proposed bridge will be a post-tensioned, precast concrete girder design consisting of a 115-foot main span over the channel and an 80-foot back span, for a total length of 195 feet. For the new design, the bridge width will be increased from 24 feet to 40 feet, with two 12-foot lanes and two 8-foot shoulders. The main pier location will be used to place a new support structure of the same dimensions. However, the four pile bents will be removed. This is expected to improve river hydraulics by providing a greater waterway opening than with the existing bridge and to reduce the likelihood of debris being caught on the bridge. The new bridge height will be increased to a minimum of 3 feet above the FEMA 100-year floodway. A variance from King County Surface Water Management (SWM) is needed because the 3-foot clearance is less than the 6-foot clearance required by SWM. The total project construction cost was estimated to be $2.1 million in 1994.

The study reach was from the mouth of the Raging River (confluence with the Snoqualmie River) to a location approximately 5.5 miles upstream. The river has a relatively steep gradient with a narrow main channel and steep banks. The geometry of the floodplain is very complex with a system of levees and non-symmetric overbank flow area. The project location and cross-sections along the river are shown in Figure 3.22.

The Raging River is leveed along both sides for the first 1.5 miles. Preliminary hydraulic analysis performed by Haper Righellis, Inc. indicated that the levees did not consistently provide 3 feet of freeboard, and thus were not certifiable to FEMA standards for flood protection. Therefore, a series of hydraulic profiles with varying levee protection conditions were required to develop a "probable risk assessment" floodplain per FEMA regulations.

For the purposes of this bridge analysis, input files for both the existing and proposed condition were based on the assumption that both levees would provide protection. By constraining the flow to the main channel (except in the cases of overtopping), this condition would assure that the effects of the construction would be maximized.

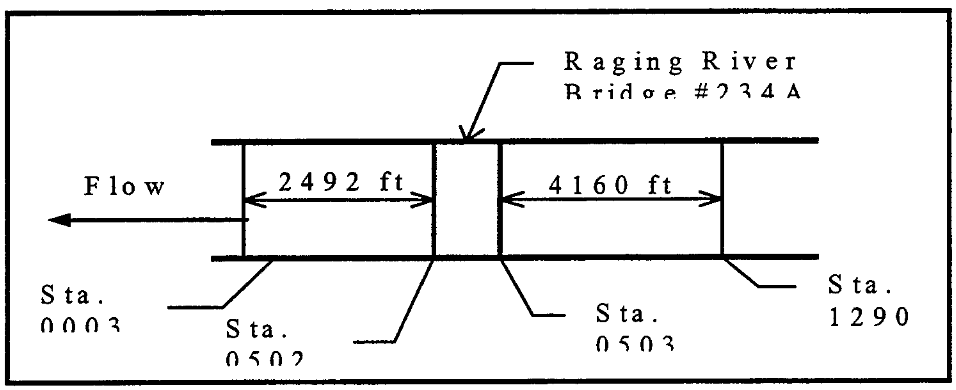


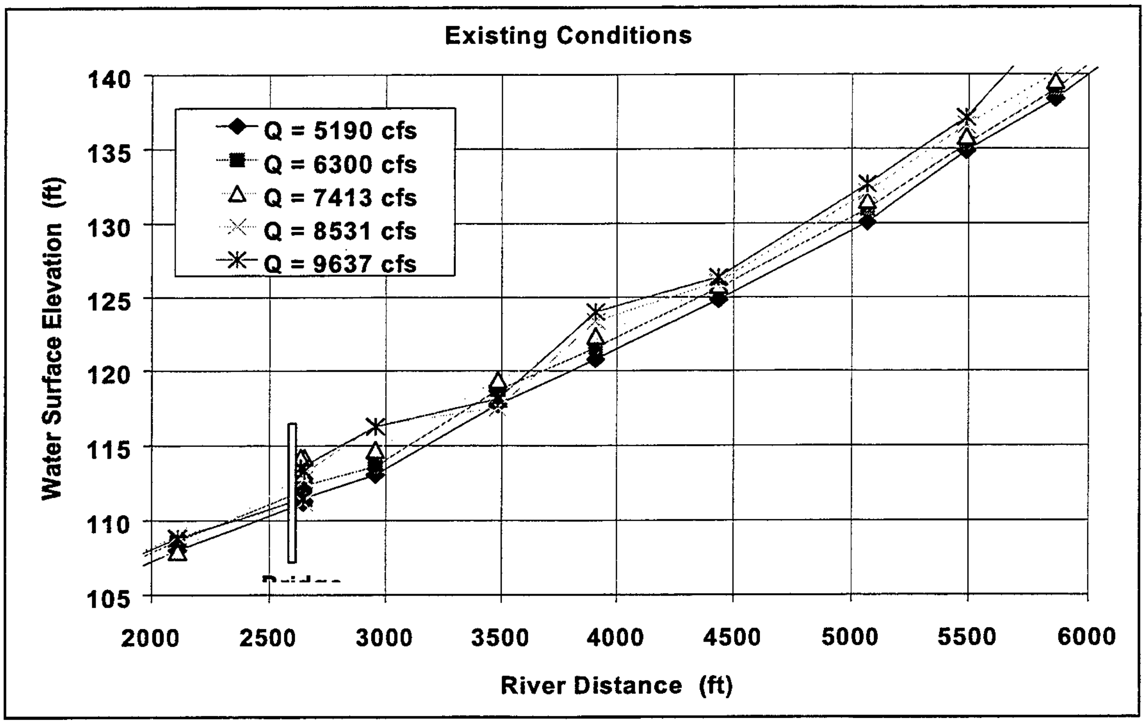
Figure 3.22. Raging River Bridge at Fall City - Project Site Layout with Cross-sections. Stations refer to river miles from confluence with Snoqualmie River.

<u>3.4.2. Change in Water Surface Elevations for Varying Discharge</u>

The mean predicted 100-year flood value of 7413 cfs was on the basis of a hydrologic analysis performed on 47 years of peak flow gage data at river mile 2.75 (68^{th} Street Bridge). The drainage basin at this gage is 30.6 square miles. The four other discharge values used in the HEC-2 model were computed from the summary statistics using the HEC-FFA (Flood Frequency Analysis) log-Pearson Type III distribution scheme, with a resulting standard deviation, σ, of 1118 cfs.

First, the discharge (Q) was varied for the five previous determined values (μ, $\pm 1\sigma$, $\pm 2\sigma$) while the Manning's n coefficient was held constant at 0.045. The profiles for the existing and the proposed conditions are given in Figure 3.23. In general, the existing and proposed condition profiles exhibited increasing water surface elevation for increasing discharge, except for at the higher values of 8531 cfs and 9637 cfs. At these higher values, the levee system and non-symmetric floodplain geometry caused complex flow conditions. When the flow was maintained within the main channel, the water surface elevations were much higher. However, when the flow "spilled" into the overbanks, the profiles became more uniform between cross-sections.

Because the overbank floodplain geometry was quite complex, the profiles for the existing and proposed conditions showed maxima at different locations. For example, at river station 3900, the two highest discharges showed a water surface near an elevation of 124 ft for the existing condition. At this elevation a portion of the river would flow into the overbank areas. As a result, at station 3500, downstream, the water surface was locally lowered to 118 ft. For the proposed conditions, the bridge design provided better water conveyance and a lower backwater. Because the water surface was generally lower, the water surface at station 3900 was at elevation 123 ft. This did not divert flow. As a result, at station 3500 the water surface was at 121 ft. The comparison between existing and proposed conditions showed a local rise of 3 ft at station 3500 even though the proposed design actually lowered the water surface at the bridge. Even though the floodplain elevation near the bridge was lower, the topography upstream resulted in local rises along the leveed portions of the river of up to 3 feet.



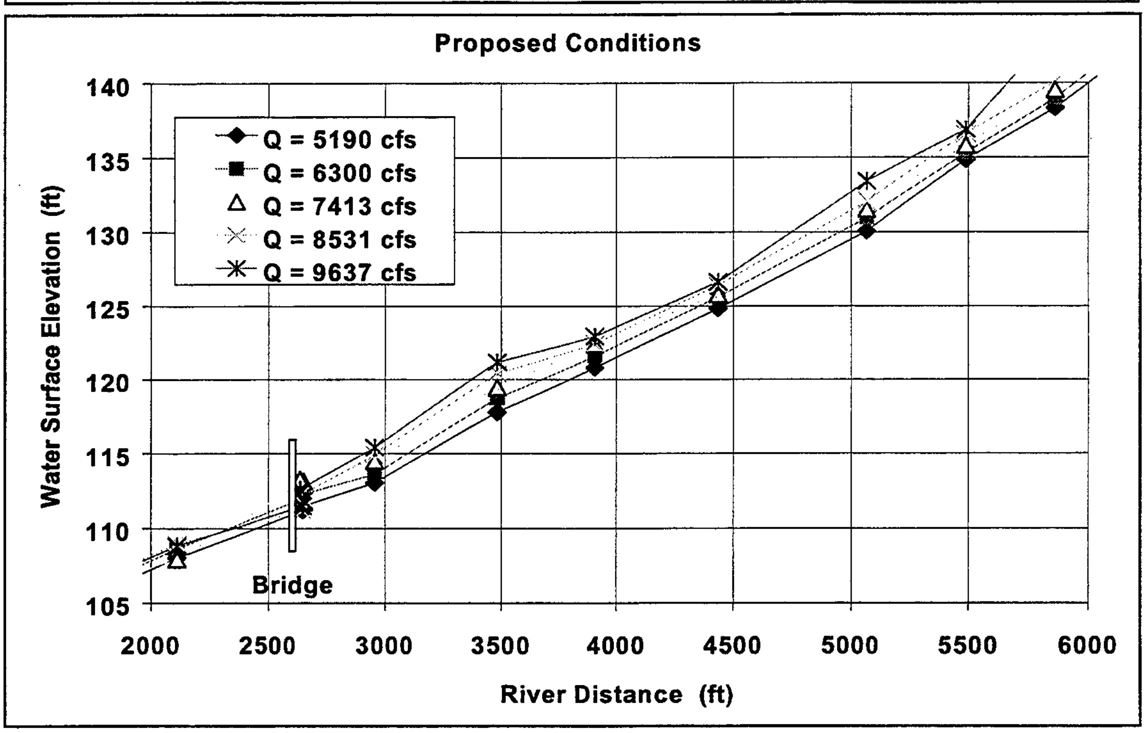


Figure 3.23. Water Surface Profiles for Varying Discharge: Existing and Proposed Conditions. Case 4.
This figure compares the effects that various discharge values have on the water surface profiles for existing and proposed conditions. Proposed conditions include bridge modification.

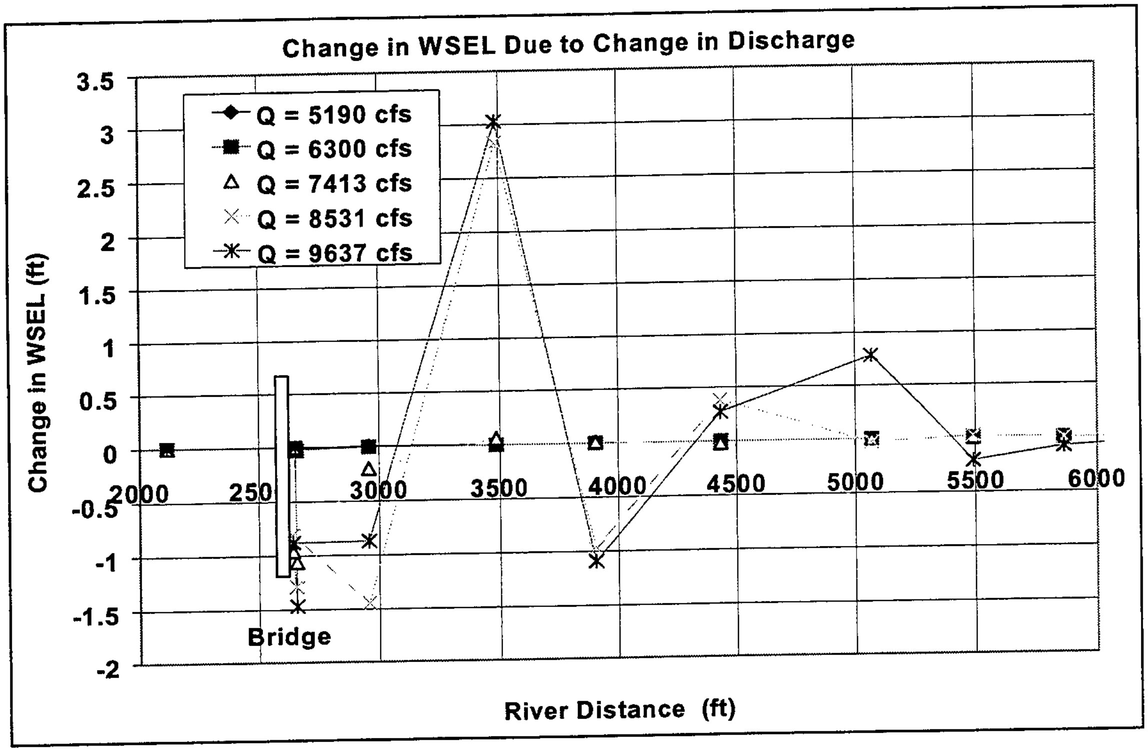


Figure 3.24. Change in Water Surface Elevation (WSEL) for Varying Discharge. Case 4. The change in WSEL is a comparison of WSEL with and without the bridge modifications.

Figure 3.24 shows the change in water surface elevation profile between the existing and proposed conditions. The values in Figure 3.24 are simply the difference between the proposed profile and the existing profile at a given location for a given discharge rate. The large fluctuations in the difference between the two water surface elevations were the result of levee overtopping at the higher discharge rates. As shown in the figure, though the proposed bridge offered a better design for flood conveyance, a reasonable choice for discharge of 8513 cfs yielded a water surface change that locally exceeds the 0.01 foot tolerance in at least three locations. This rise occurs in an area where the river flows between levees so that no impact would be expected from flooding.

3.4.2 Change in Water Surface Elevation for Varying Roughness Coefficients

The values of the Manning roughness coefficient were varied while the discharge was held constant at the predicted mean 100-year event, 7413 cfs. Plots of the existing and proposed profiles, as well as the difference between them, are shown in Figures 3.25 and 3.26. The same behavior that was displayed for varying discharge values is exhibited by the model for varying Manning's n values. While the general water surface profiles were not greatly different between existing and proposed conditions, the water surfaces in a given location varied greatly, depending on the choice of Manning coefficient. The shift in the location of the maximum change in water surface in this case only occurred for a Manning's n value of 0.040.

Again, the proposed design yielded either an unchanged or a lower water surface at the bridge. Upstream, however, the depth introduced by the particular Manning n value of 0.040 caused overtopping of the levees in such a manner that large changes in the water surface level occurred between the proposed and existing conditions. If the zero-rise ordinance were enforced rigidly, the model would be extremely sensitive to the exact choice of roughness coefficient at the bridge. A small variation from the calibrated value of n = 0.045 could result in a predicted local rise in the floodplain level of over 1.5 feet.

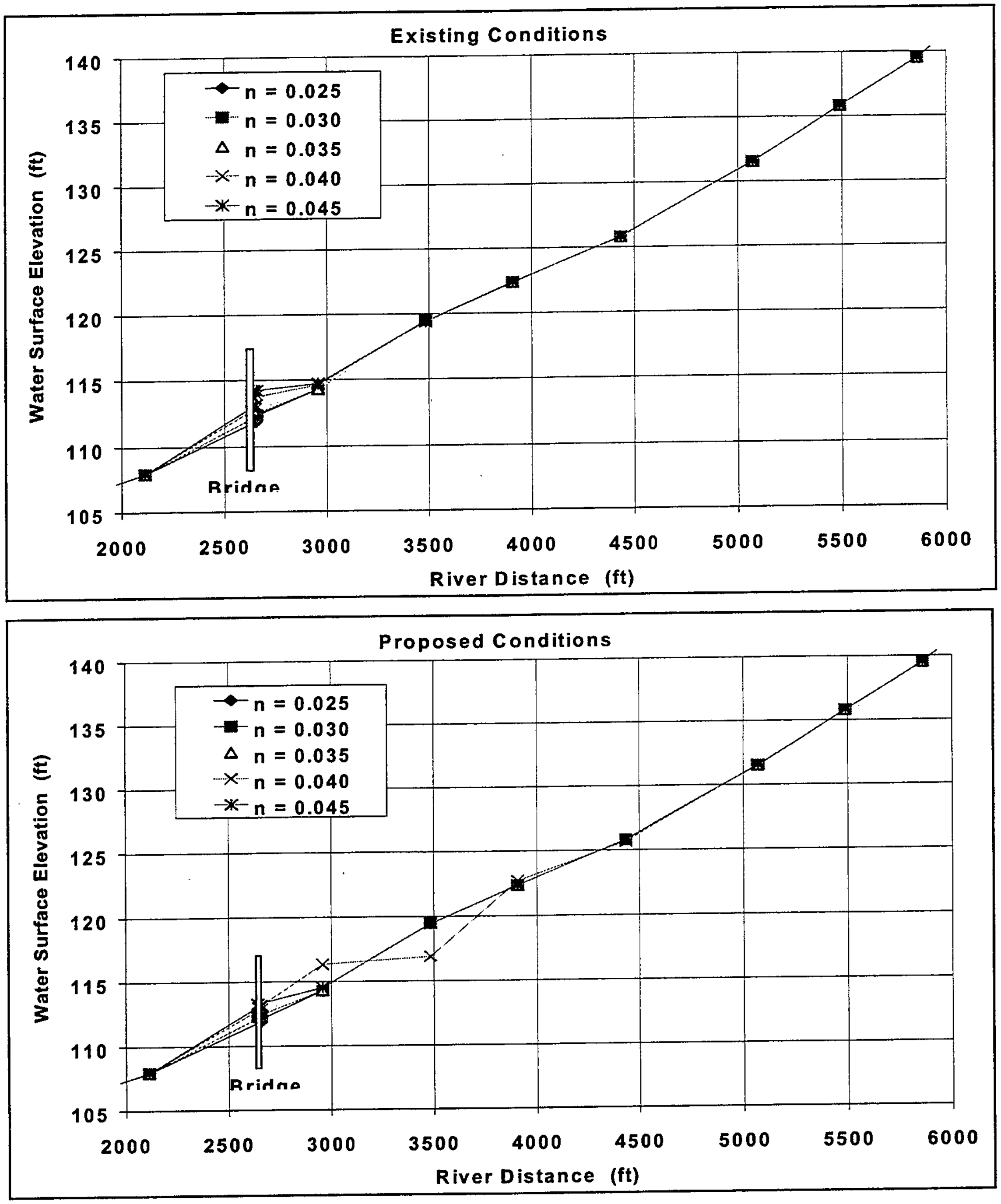


Figure 3.25. Water Surface Profiles for Varying Manning's Roughness: Existing and Proposed Conditions. Case 4.
This figure compares the effects that varying roughness values of Manning's n have on the water surface profiles for existing and proposed conditions and a constant discharge of 7413 cfs.

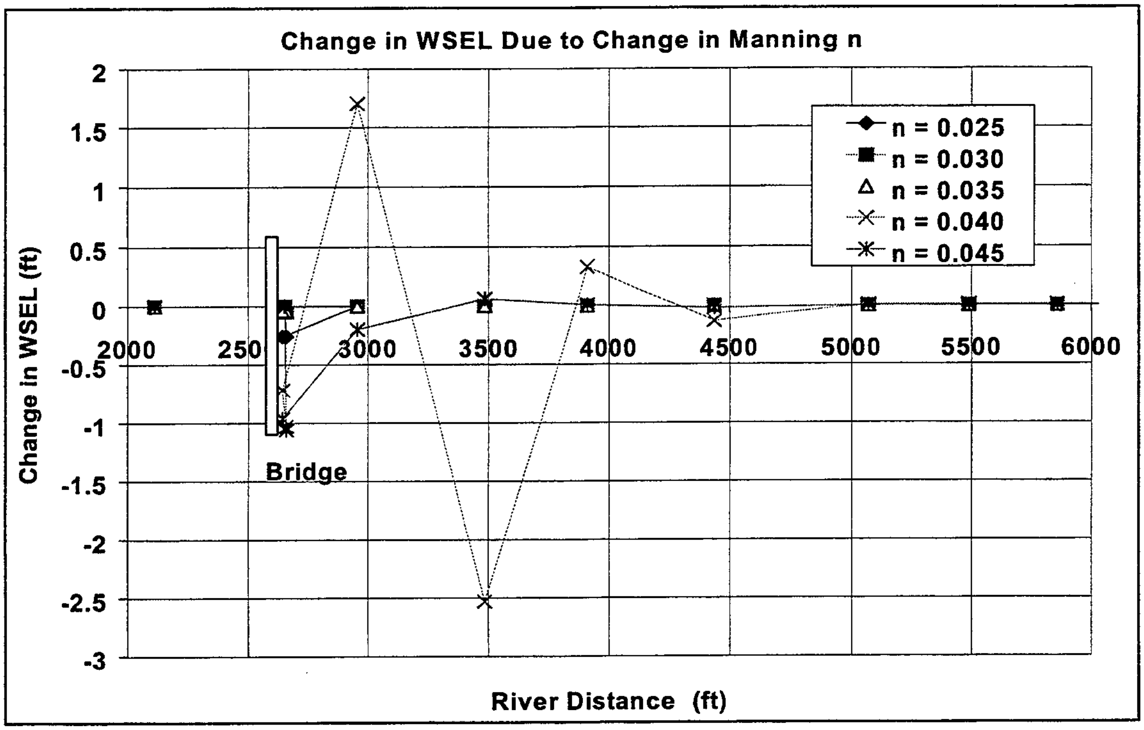


Figure 3.26. Change in Water Surface Elevation (WSEL) for Varying Manning's Roughness. Case 4.
River Distance refers to cross-sections along the river. The change in WSEL is a comparison of WSEL with and without the bridge modifications for a given roughness coefficient.

3.4.4 Change in Water Surface Elevation for Combined Discharge and Manning's n Variation

The range of possible maximum water surface variations for 25 combinations of discharge and Manning roughness is presented in Table 3.8 for the river distance of 3000 ft. Because the proposed design for the Raging River would create a bridge opening larger than that of the existing bridge, a number of the resulting values showed either no impact from the bridge or a significantly lower water surface. The exact choice of design discharge and Manning coefficient played a large role in the predicted amount of rise. The predicted amount of rise for a fixed discharge varied from 0.03 feet to 1.90 feet, depending on the choice on roughness, while the predicted amount of rise for a fixed roughness varied between 0.08 and 3.05 feet. The range of variation was well above the 0.01-foot tolerance, even for small changes in the values of hydraulic variables.

Table 3.8. Change in Water Surface Elevation (in feet) for a Given Discharge and Roughness Coefficient. River Distance 3000 ft.

Discharge	n=.025	n=.030	n=.035	n=.040	n=.045	Range of Variation*
5190 cfs	0.01	-0.02	0.00	0.00	0.01	0.03
6300 cfs	0.03	0.00	0.00	0.00	0.00	0.03
7413 cfs	0.00	0.00	0.00	1.70	-0.20	1.90
8531 cfs	0.02	0.01	-1.33	-1.35	-1.45	1.47
9637 cfs	-0.10	-0.07	-0.73	-0.11	-0.87	0.10
Range of Variation*	0.13	0.08	1.33	3.05	1.46	

*Range of Variation is defined as the difference between the largest and smallest possible outcome.

Figure 3.27 shows the variation of change in water surface elevation with discharge and Manning roughness. The model was not very sensitive to changes in these hydraulic variables for low values of Manning n and discharge. At values near the mean 100-year discharge of 7413 cfs and the calibrated roughness value of 0.045, however, the model was very sensitive to any marginal change in either of these variables. This indicates that the confidence with which the level of rise can be predicted is poorest near the most probable values of hydraulic input variables. For Case 4, use of a 1-D model such as HEC-2 to assess compliance with the zero tolerance ordinance was not successful.

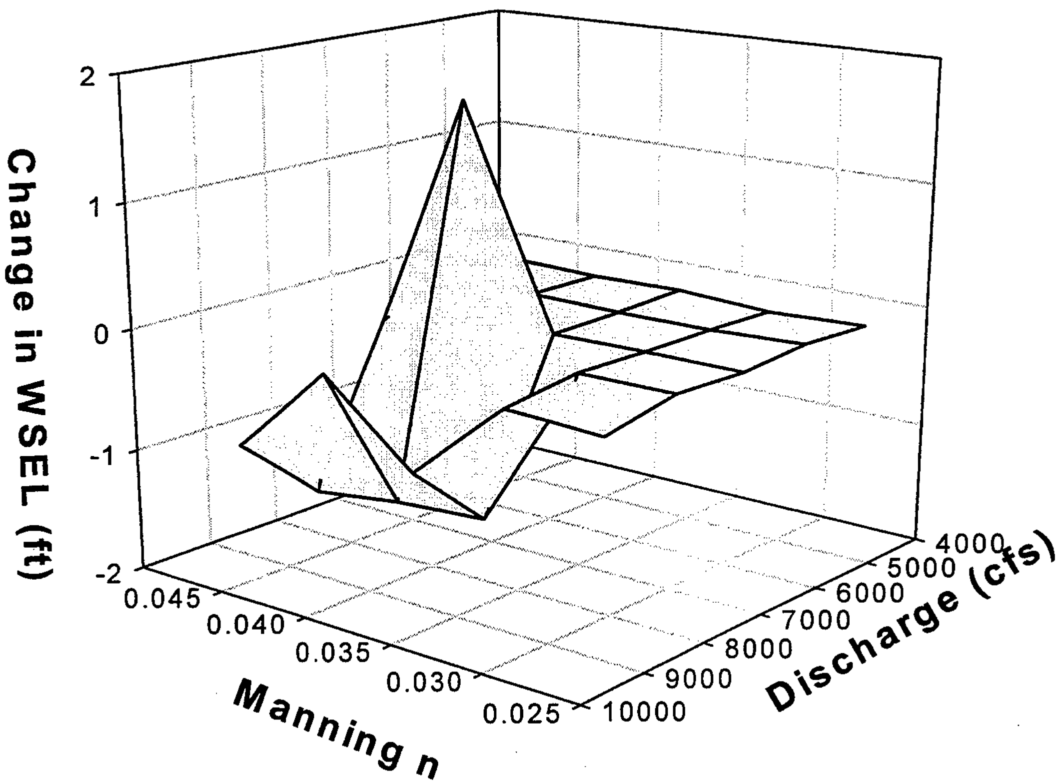


Figure 3.27. Change in Water Surface Elevation vs. Manning's n and Discharge. Case 4.

3.4.5 Summary for Case Study Number 4

The results of this case study showed that, although the current hydraulic model inputs resulting in less than 0.01 feet of rise, a small variation in the estimate of the hydraulic input variables caused the predicted rise to vary by well more than 0.01 ft. Although the proposed bridge modifications would increase the flow conveyance at the bridge location, some combinations of flow variables predicted that the floodplain level would actually increase at certain locations upstream of the bridge. This implies that the new bridge would not be allowed under the zero-rise ordinance. The trends in water surface elevation change were not uniform for either varying discharge or for varying Manning roughness. The sensitivity of the model predictions were greatest near the most likely values of discharge and roughness. Part of this is due to the complexity of the topography at the project site and to the presence of levees. Because of the complex conditions of the Raging River, the use of a 1-D model, such as HEC-2, may not be the appropriate tool for evaluating the dynamics of its floodplain. Table 3.9 summarizes the findings for Case 4.

Table 3.9. Summary of Results for Case Study Number 4.

	Value
Drainage Basin Area	30.6 sq. miles
Slope	0.0004
100-year Peak Flow	7,413 cfs
Standard Deviation of Discharge	1118 cfs
Critical Section	0560
Maximum Change in Water Surface Elevation	1.7 ft
Maximum Range of Variation in Water Surface Elevation Prediction due to Q	3.05 ft
Maximum Range of Variation in Water Surface Elevation Prediction due to Manning's n	1.9 ft
Hydraulic Input Variable of Most Sensitivity	100-year Discharge

3.5 Case Study No. 5 - SR-18 Bridge at Green River

3.5.1 Case Study Description

This case study involved the construction of a new 406-foot-long three-span, composite steel plate girder bridge for the new westbound lanes of SR-18 crossing the Green River. This bridge was constructed north of and adjacent to the existing SR-18 Green River bridge. The existing bridge will stay in place and is used for eastbound traffic. The substructure of the new bridge consists of four piers with a central span length of 171 ft and one pier located within the FEMA regulatory floodway.

In the Floodplain Evaluation Report drafted in 1992 for this crossing, a hydraulic model was run using a 436-foot-long, precast, prestressed girder bridge with a central span of 145 feet and two piers located in the floodway. The results of the analysis indicated a resulting floodplain rise of 0.09 to 0.13 feet. By contrast, the current design resulted in a lower predicted rise of 0.01 to 0.03 when analyzed with the model used for this case study. If a zero-rise bridge were required in this location, it would be designed as a cast-in-place concrete segmental bridge 470-feet long with a central span of 220 feet. This would allow both intermediate piers to be placed outside the 100-year floodplain. The resulting cost increase would be over 50 percent of the $1.4 million cost of the 406-foot steel girder bridge (Wood et al 1997).

For the SR-18 crossing of the Green River, the 100-year event discharge is regulated by the Howard Hanson dam and is assumed to be fixed at 12000 cfs by dam release regulations. For this reason, the sensitivity analysis was only carried out for the Manning n values. Because of the lack of high water marks near the bridge crossing, Manning n values used in the FEMA model for this area were based on engineering judgment. This was the case for the 1992 floodplain study as well. No additional information had been obtained that could be used to calibrate the choice of roughness coefficients for locations upstream of the USGS Auburn gauging station. The project layout is shown in Figure 3.28.

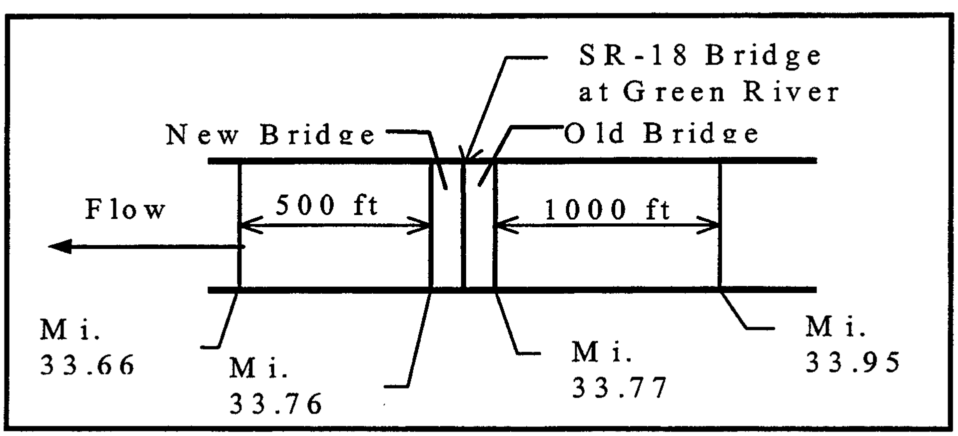
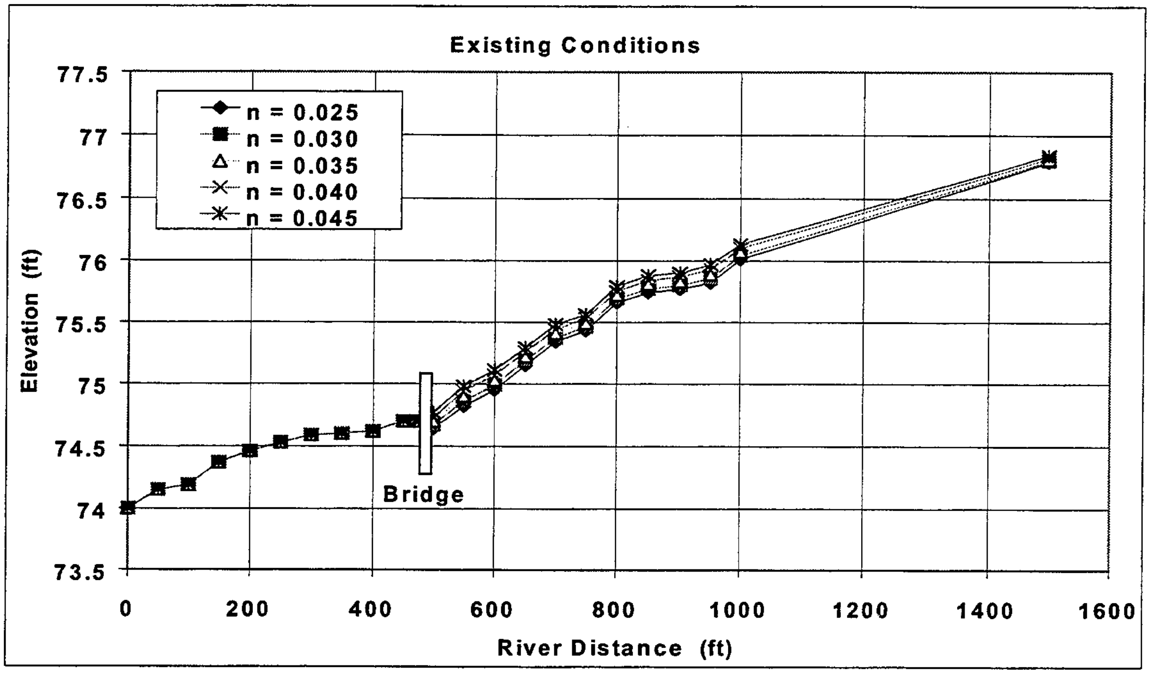


Figure 3.28. SR-18 Bridge at Green River - Project Site Layout with Cross-sections.

3.5.2 Change in Water Surface Elevation for Varying Roughness Coefficients

Both existing and proposed bridge configurations were run for the regulated peak discharge of 12,000 cfs. Five different values of the Manning roughness coefficient n were used, varying from 0.025 to 0.045. In light of the estimated Manning roughness coefficients in the HEC-2 model for this river reach, a sensitivity analysis was particularly useful for this bridge crossing. In addition, the actual roughness at the bridge for the proposed conditions was difficult to predict because bank protection and a Cedar log and root wad will be placed at the crossing.

The water surface profiles for the existing condition and the proposed condition with the new bridge are shown in Figure 3.29. As would be expected, the higher Manning coefficients resulted in a higher water surface elevation. Because of channel modifications proposed for the new bridge and the use of only one pier in the floodplain, the two sets of curves appear almost identical.



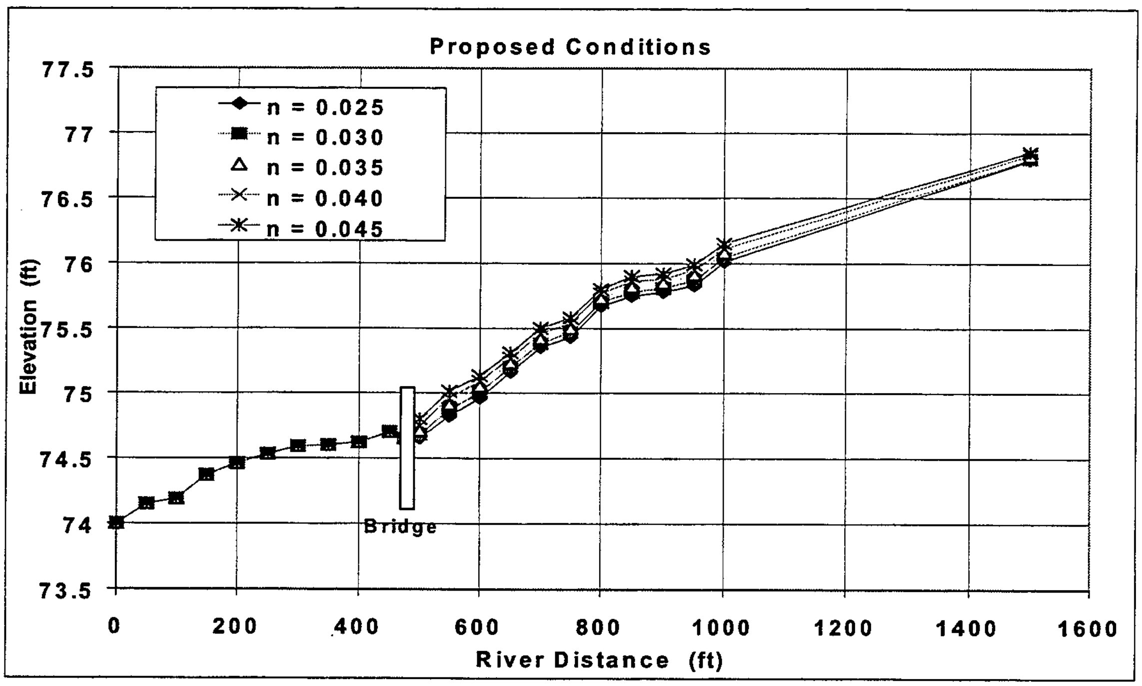


Figure 3.29. Water Surface Profiles for Varying Manning's Roughness: Existing and Proposed Conditions. Case 5.
This figure compares the effects that varying values of Manning's n have on the water surface profiles for existing and proposed conditions and a constant discharge of 12,000 cfs. Proposed conditions include the new bridge.

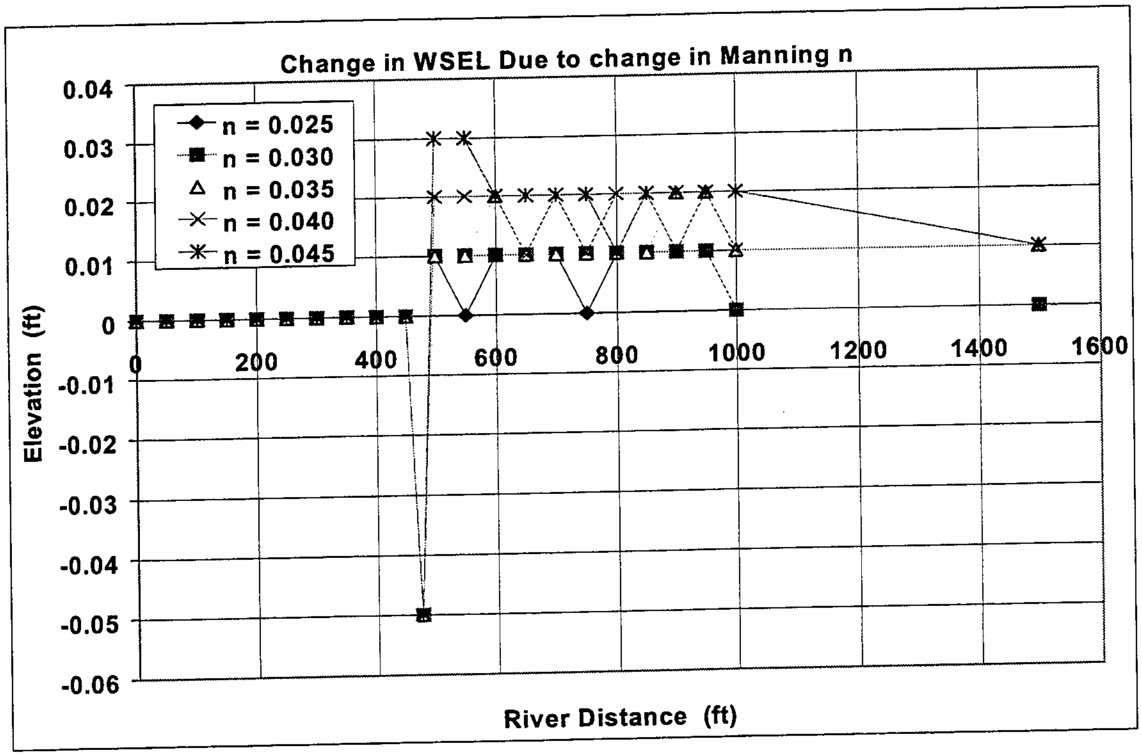


Figure 3.30. Change in Water Surface Elevation (WSEL) for Varying Manning's Roughness. Case 5.
River Distance refers to cross-sections along the river. The change in WSEL is a comparison of WSEL with and without the new bridge for a given roughness coefficient.

The change between existing and proposed conditions at each cross-section is shown in Figure 3.30. A slight drop of 0.05 feet occurred just downstream of the new bridge location as a result of the structure. The backwater upstream of the bridge ranged from 0.01 feet for a Manning roughness of less than 0.035 up to between 0.02 and 0.03 feet for a Manning roughness of greater than 0.035. Although the effects of the new bridge were minimal, the local rise in the floodplain still exceeded the 0.01-foot tolerance. The model was sensitive to the choice of the Manning roughness coefficient, with a larger change in water surface occurring for higher values of Manning n. For the cross-sections located 1000 feet upstream of the bridge, there was no computed effect on the water surface.

3.5.3 Summary for Case Study Number 5

The results of this case study showed that a small variation in the choice of the Manning roughness coefficient at the bridge caused the predicted water surface rise due to the bridge to exceed 0.01 feet. The roughness coefficients for this reach were chosen through engineering judgment rather than as the result of a calibration with known high water marks. It was evident that the effect of the new bridge could not be accurately predicted: a change in roughness of 0.005 at one cross-section could cause the bridge to fall in or out of compliance. Table 3.10 shows that for the maximum regulated discharge of 12,000 cfs, the range of variation for the water surface rise at the critical section (river distance 500 ft) was 0.03 feet. Because of the uncertainty in roughness coefficients, it is not likely that using HEC-2, or any other hydraulic model, would allow an engineer to assess the bridge impact to within 0.01 feet. Table 3.11 summarizes the results of Case Study 5.

Table 3.10. Change in Water Surface Elevation (in feet) for a Given Roughness Coefficient at Critical Section. Case 5.

Discharge	n=.025	n=.030	n=.035	n=.040	n=.045	Range of Variation*
12000 cfs	0.01	0.01	0.01	0.02	0.03	0.03

*Range of Variation is defined as the difference between the largest and smallest possible outcome.

Table 3.11. Summary of Results for Case Study Number 5.

	Value
Drainage Basin Area	399 sq. miles
Slope	0.004
100-year Peak Flow	12,000 cfs
Standard Deviation of Discharge	0 cfs (regulated upstream)
Critical Section	Mile 33.764
Maximum Range of Variation in Water Surface Elevation Prediction due to Manning's n	0.03 ft

4.0 DAMAGE ASSESSMENT

When flooding occurs in a region, the extent of damage depends greatly on the area surrounding the river. A thorough risk assessment should evaluate the probability of flood occurrence together with the associated damage costs related to that magnitude of flood. The analysis should be repeated for a broad range of flood probabilities. The damage analysis carried out in this study only examined the damage associated with the 100-year flood for each of the case studies. Because of the rural environment of these case studies, the estimated damages were minimal. If the case studies had been located in an urban area, the damages might have been larger by orders of magnitude.

4.1 Case Study No. 1 - SR 169 Bridge at Cedar River

As previously mentioned, the new Cedar River Bridge is due east of Renton on SR 169. A housing development of approximately 200 homes is located immediately downstream of the bridge and south of the existing SR 169 Cedar River Bridge. According to King County's floodplain analysis and the HEC-2 analysis, this development would not be exposed to flooding during the 100-year storm event. Just upstream of the bridge crossing and north of SR 169 is a newly constructed golf course. Beyond the golf course, the river meanders through undeveloped land that is highly vegetated with brush and trees. Except for a golf course storage shed, no structures are located along the river until the Elliott Bridge, which is approximately 1.5 miles upstream of the Cedar River Bridge.

In this case study, the possible values of the change in water surface elevation, given the five reasonable values of discharge and the five roughness coefficients, ranged between 0.10 ft. and 0.29 ft. This rise in water surface elevation would not cause significant structural damage. Only one structure, the golf course storage shed, could be inundated. The assessed value of this shed was not recorded in the 1994-95 edition of King County's Parcel List, but a $30,000 value was assumed. If the shed was inundated up to 0.5 ft. during the 100-year storm event, approximately $5,500 of structural damages could be expected. An additional foot of inundation would increase the structural damages by another \$5,500, totaling to \$11,000 in damages.

For this case study, land damages and their associated costs were reviewed in addition to the structural costs because the damages to one structure did not provide a representative measure of the damages due to development in the floodplain. Also, incorporating land damages would provide a better estimate of the expected damages for this case study because one of the largest properties that could be flooded during the 100-year storm event is a golf course. Obviously, the disuse of land resulting from flooding would be a significant cost to this type of business.

According to King County's Assessor's Records, the average land value for properties adjacent to the new Cedar River Bridge and used in this case study is \$10,700 per acre. The average amount of additional land that would be inundated because of the construction of a bridge in the floodplain was approximately 2,100 ft^2 for this case study. The damages to land associated with this flooding totals \$500. The range of values obtained from the 25 comparisons of existing to proposed conditions is listed in Table 4.1.

Figure 4.1 displays the expected damages due to the increase in water surface elevation that would result from constructing a bridge in the floodplain tabulated against the inputs of discharge and channel roughness. The costs of the expected land damages are small, ranging from \$100 to \$1,000. Assessed values are usually lower than the fair market value at the time of a sale. Because the average land value used in this analysis was based on assessed values, the cost of land damages could be greater than previously stated.

Table 4.1. **Land Damages for a Given Discharge and Roughness Coefficient.**

Discharge (cfs), Manning's n		Additional Land Inundated (ft^2)	Damages due to New Bridge ($)
9,236	0.025	1,677	400
9,236	0.030	1,308	300
9,236	0.035	1,056	300
9,236	0.040	2,862	700
9,236	0.045	2,436	600
10,434	0.025	1,905	500
10,434	0.030	1,419	300
10,434	0.035	2,022	500
10,434	0.040	2,082	500
10,434	0.045	1,290	300
11,632	0.025	2,046	500
11,632	0.030	3,792	900
11,632	0.035	1,659	400
11,632	0.040	1,458	400
11,632	0.045	1,383	300
12,830	0.025	4,266	1,000
12,830	0.030	2,325	600
12,830	0.035	1,662	400
12,830	0.040	1,452	400
12,830	0.045	1,365	300
14,028	0.025	3,480	900
14,028	0.030	2,292	600
14,028	0.035	1,647	400
14,028	0.040	1,485	400
14,028	0.045	261	100

However, even if the fair market value had been used in this analysis, the cost of land damages for this case study would still not have exceeded the cost of constructing a single span bridge to comply with the zero-rise ordinance. In fact, the average land value for this case study would have to be $35 million per acre for the expected damages to exceed the cost of constructing a single span bridge.

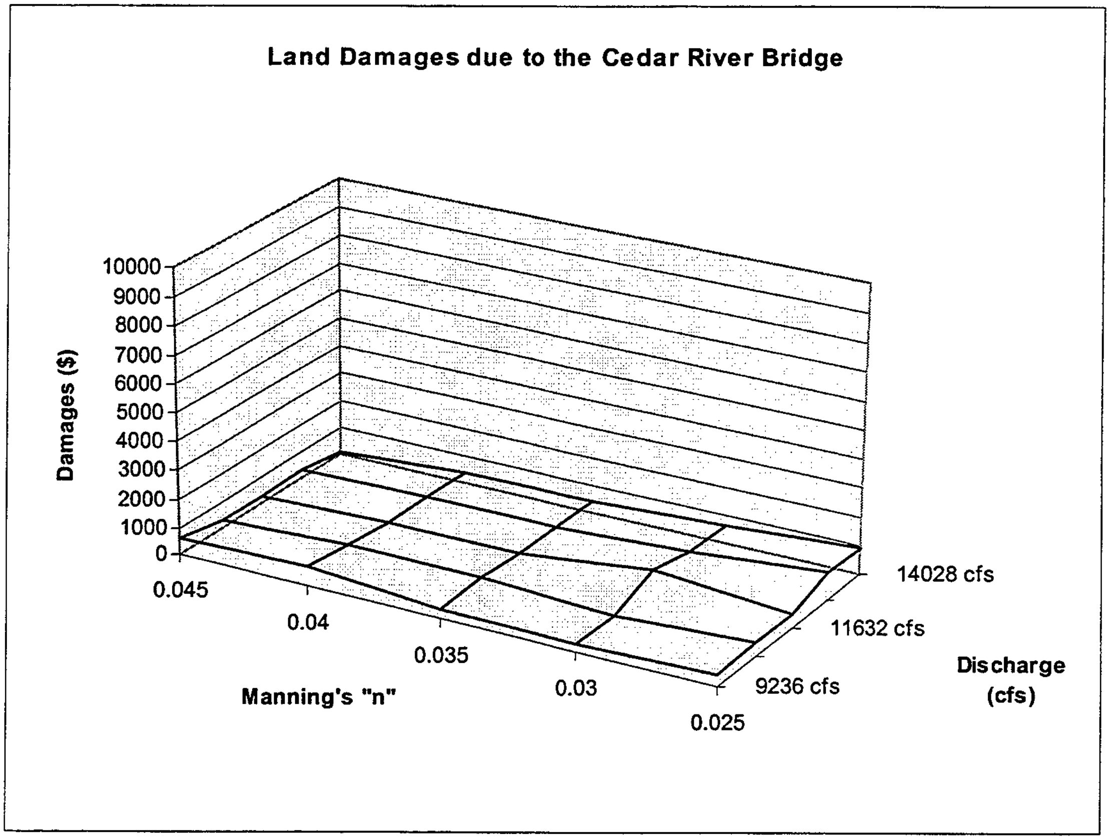


Figure 4.1. Land Damages for a Given Discharge and Roughness Coefficient.

4.2 Case Study No. 2 - Elliott Street Bridge at Cedar River

Several homes are immediately downstream and upstream of Elliott Bridge in the 100-year floodplain. The damage assessment for this case study focused on homes located near the critical section, Section 2300. The area of analysis included 10 private residences along the north side of the Cedar River between the critical section and the cross-section immediately downstream, Section 2200. These homes are in a naturally low area, with the Cedar River to the south and Jones Road to the north. No homes between the critical section and the section immediately upstream, Section 2400, are exposed to flooding. Similarly, no homes are along the south side of the Cedar River between Sections 2200 and 2400. This land is undeveloped and highly vegetated with trees and brush.

In this case study, the possible values of the change in water surface elevation for the critical section ranged from 0.25 ft. to 0.67 ft. This rise would not cause any structural or land damages at this section because a levee along the north side of the Cedar River would confine the flow to the main channel and protect the homes north of Jones Road. If only the critical section and not what happens between cross-sections is considered, one could reason that even though constructing a bridge in the floodplain would increase the water surface elevation by as much as 0.67 ft., this rise would not cause any damages. However, this is not true.

To fully assess the impacts associated with the backwater at the critical section, the computed water surface elevation for the cross-section immediately downstream of Section 2300 was used to determine the water surface elevation at intermediate locations. Downstream of the critical section was of interest because 10 homes could be flooded in that area. According to topographical maps and aerial maps, no homes are in danger of being flooded upstream of the critical section.

All of the 25 combinations of discharge and channel roughness, with the exception of a discharge of 9,236 cfs, produced a water surface elevation that would overtop Jones Road in this area. However, the increase in water surface elevation due to constructing a bridge in the floodplain would not expose any additional homes to flooding. The depth of inundation would increase, resulting in an increase in the cost of structural damages.

Because the water surface elevations are not constant between the cross-sections of interest, three intermediate points were used to estimate the inundation of the 10 homes. As mentioned earlier, damages were assessed in 1-foot increments. The increase in structural damages due to constructing a bridge in the floodplain was calculated as the difference between the damages incurred under existing and proposed conditions. The values of flood damage are shown in Table 4.2. Figure 4.2 displays the flood damage values. The change in structural damage due to flooding ranged from $0 to approximately $33,000, depending on the value used for discharge and Manning's n.

Table 4.2. Changes in Structural Damages from Existing to Proposed Conditions for a Given Discharge and Roughness Coefficient.

	Q=9,236 cfs	Q=10,434 cfs	Q=11,632 cfs	Q=12,780 cfs	Q=14,028 cfs
n = 0.025	$0	$24,117.60	$16,078.40	$16,078.40	$13,719.60
n = 0.030	$0	$33,155.90	$16,078.40	$9,146.40	$13,719.60
n = 0.035	$0	$33,155.90	$16,078.40	$9,146.40	$13,719.60
n = 0.040	$0	$0	$16,078.40	$9,146.40	$13,719.60
n = 0.045	$0	$0	$16,078.40	$16,078.40	$13,719.60

As expected for structural damages, the increase in damages due to construction decreased as the discharge increased. As shown in the sensitivity analysis, higher discharges would result in greater changes in the water surface elevations from existing to proposed conditions. At the critical section, this change would be associated with a rise in the water surface elevation. Higher water surface elevations would result in deeper inundation and thus more damages. However, the increase in inundation depth would not be directly proportional to the increase in structural damages. Under existing conditions, higher discharges would produce a higher water surface profile than lower discharges. Under proposed conditions, the inclusion of a bridge would cause the water surface profile to rise even more. However, the incremental difference in structural damages would be small for higher discharges because damages would already be high under the existing condition. The effects of the bridge would be felt less because the flow itself, without the bridge, would already cause a lot of damages.

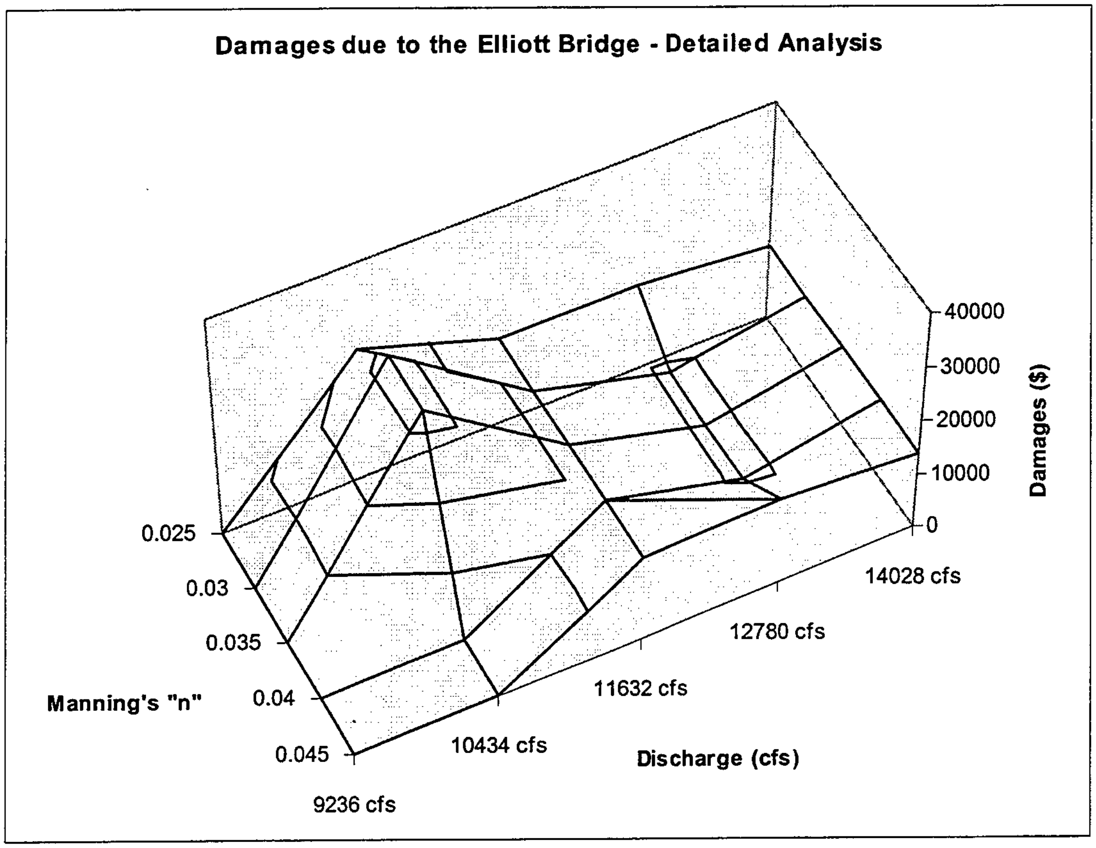


Figure 4.2. Changes in Structural Damages from Existing to Proposed Conditions. This figure displays the damages due specifically to the bridge.

Conversely, lower discharge values would produce a lower water surface profile for existing conditions than higher discharges. The flows produced by low discharges would usually be contained within the main channel. Thus, under existing conditions, the flow would not cause significant damage. However, building a bridge in the floodplain would increase the water surface elevation, allowing the flow to escape from its confines and flood neighboring properties. As a result, the difference in structural damages for lower discharges would be greater than those damages for higher discharges. These trends were not always followed precisely because of the 1-foot assessment interval.

4.3 Case Study No. 3 - Meadowbrook Bridge at Snoqualmie River

The Meadowbrook Bridge site differed significantly from the other two case studies because of the location of the bridge. The bridge is located directly in the town of Snoqualmie. The bridge is upstream of most homes in Snoqualmie, but backwater effects would impact many homes in the neighborhood directly upstream of the bridge on the south banks of the river. On the north side of the river, and further upstream on the south side, there are no housing developments. In this case study, both the increase in structural damage and land damage due to the bridge were considered.

The town of Snoqualmie is at a relatively low elevation with respect to the elevation of the river and frequently floods. In the event of a 100-year peak flow, the town would be inundated, regardless of the condition of the bridge. With increased discharge, the river would expand in width as well as in depth. The HEC-2 analysis showed that the greatest change in river width between existing and proposed conditions would not occur at the critical section. Therefore, for this damage analysis, the entire modeled reach of river upstream of the bridge was used in the calculations.

For this case study, the modifications to the existing Meadowbrook Bridge would cause no increase in structural damage, if calculated with Method One described under Section 2.0 (Methodology). Modifying the bridge would inundate no additional homes, and the change in water surface elevation would not be great enough to cause a cost difference, given the depth-damage curves.

To estimate what the potential structural damages would be if the depth-damage curves were used to greater accuracy, Method Two was used. The structural damage cost was assessed for the water surface elevation with the existing conditions of the bridge. The structural damage cost was also assessed for a 1-foot higher water surface elevation for the proposed condition of the bridge. This cost was then pro-rated for the incremental change of water surface elevation. According to King County Assessor's records, the average value of homes in the area is $45,360. Table 4.3 shows the changes in structural damage calculated with Method Two. The change in flood damage due to structural damage ranged from $0 to approximately $19,000, depending on the values used for discharge and Manning's n.

Table 4.3. Changes in Structural Damages from Existing to Proposed Conditions with Discharge and Roughness Coefficient.

	Q=63,542 cfs	Q=71,855 cfs	Q=80,168 cfs	Q=88,481 cfs
n = 0.025	$18,941	$15,612	$14,081	$6,442
n = 0.030	$15,497	$15,612	$11,947	$5,249
n = 0.035	$6,887	$8,516	$8,107	$3,102
n = 0.040	$2,870	$2,839	$2,133	$0
n = 0.045	$3,444	$2,839	$1,707	$0

For Case Study 3, land damage and its associated costs had to be analyzed. Most of the land in the reach of the river under analysis do not have any existing structures. Therefore, the land damages were essential to consider. As in the first case study, the entire value of the land inundated was conservatively assumed to be lost. The average land value taken from properties adjacent to Meadowbrook Bridge is $95,000 per acre. Using this value, the increase in land damages for the 20 comparisons were calculated. The results are listed below in Table 4.4. The change in flood damage due to land damage ranged from $0 to approximately $140,000, depending on the values used for discharge and Manning's n.

Table 4.4. Land Damages for a Given Discharge and Roughness Coefficient.

Discharge (cfs), Manning's n		Additional Land Inundated (ft^2)	Damages due to Bridge Mod. ($)
63,542	0.025	52,953	115,484
63,542	0.030	45,295	98,785
63,542	0.035	21,565	47,030
63,542	0.040	7951	17,340
63,542	0.045	7776	16,959
71,855	0.025	63,806	139,156
71,855	0.030	59,376	129,494
71,855	0.035	34,471	75,177
71,855	0.040	9956	21,713
71,855	0.045	10,855	23,673
80,168	0.025	59,571	129,919
80,168	0.030	54,817	119,550
80,168	0.035	37,927	82,716
80,168	0.040	20,492	24,842
80,168	0.045	10,101	22,029
88,481	0.025	47,950	104,574
88,481	0.030	39,881	86,976
88,481	0.035	24,948	54,410
88,481	0.040	0	0
88,481	0.045	0	0

Table 4.5. Changes in Total Damages from Existing to Proposed Conditions with Discharge and Roughness Coefficient.

	Q=63,542 cfs	Q=71,855 cfs	Q=80,168 cfs	Q=88,481 cfs
n = 0.025	$134,425	$154,768	$144,000	$111,016
n = 0.030	$114,282	$145,106	$131,497	$92,225
n = 0.035	$53,917	$83,693	$90,823	$57,512
n = 0.040	$20,210	$24,552	$26,975	$0
n = 0.045	$20,403	$26,512	$23,736	$0

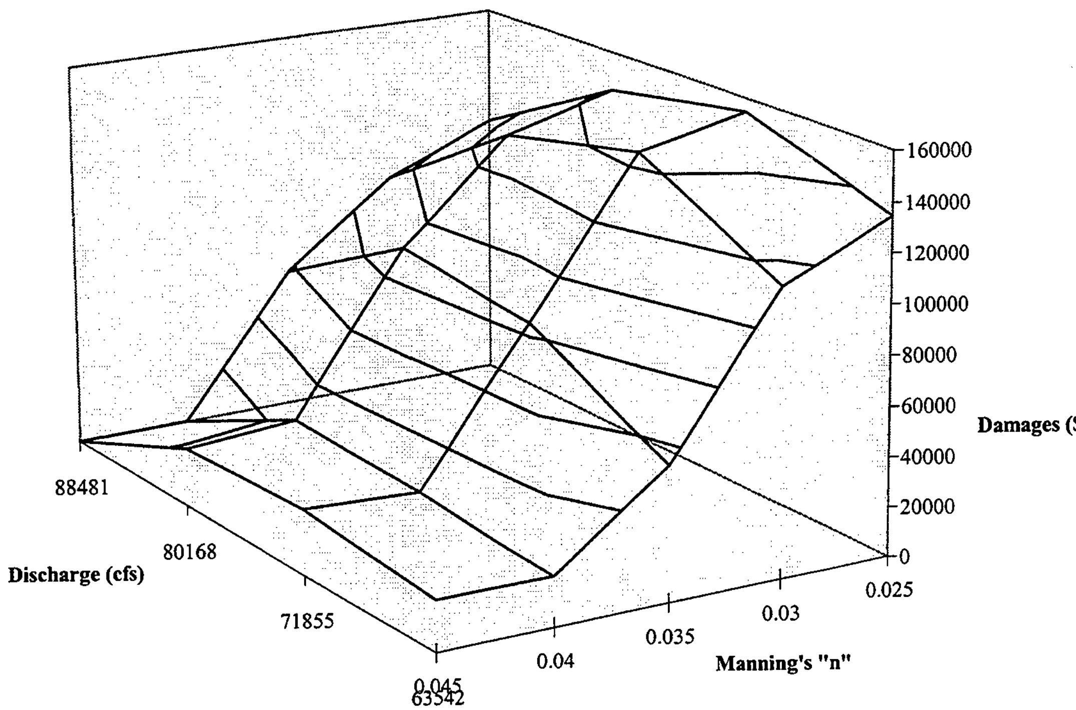


Figure 4.3. Total Damages Due to Increase in Water Surface Elevation from Bridge Modifications vs. Discharge and Channel Roughness.

The change in total damages due to Meadowbrook Bridge modifications was the sum of the land damages and the structural damages. Table 4.5 shows the values of the total damages caused by flooding for Case Study 3. Figure 4.3 graphically displays the total flood damage values. The change in total damage ranged from $0 to approximately $155,000, depending on the value used for discharge and Manning's n.

5.0 ALTERNATIVES TO ZERO-RISE TOLERANCE

Although the zero-rise ordinance seeks to control flooding damages and development in the 100-year floodplain, the law may cause unintentional negative impacts. The cost of maintaining bridges and roadways near rivers has the potential to increase substantially. As a result, bridge improvement projects become less viable, and some projects may become prohibitively expensive (Wood et al, 1997). The ordinance addresses only one part of a multifaceted problem. It does not balance the safety of the public with the cost of flood damages, nor does it require an assessment of the actual impacts to the environment of the bridge modifications.

This section describes and evaluates some alternative options to the zero-rise ordinance. These options fall into one of three categories:

- limitation based laws
- laws based on economic factors
- broad-scale watershed management laws

Sections 5.1 through 5.3 explore these categories further.

5.1 Limitation Based Laws – Limiting the Energy Grade Line

Limitation based laws are often legislated for floodplain protection. For example, the zero-rise ordinance limits the allowable increase in water surface elevation during the 100-year flood. An alternative approach would limit the maximum change in the energy grade line caused by construction in the floodplain. The energy grade line is the elevation head plus the velocity head, or $z + v^2/(2*g)$, where v is the cross-sectional averaged velocity and z is the river stage. Laws that limit only the change in water surface elevation assume that all the damages are caused by increased inundation depth. This neglects the fact that a faster velocity down the channel and over the floodplain will cause

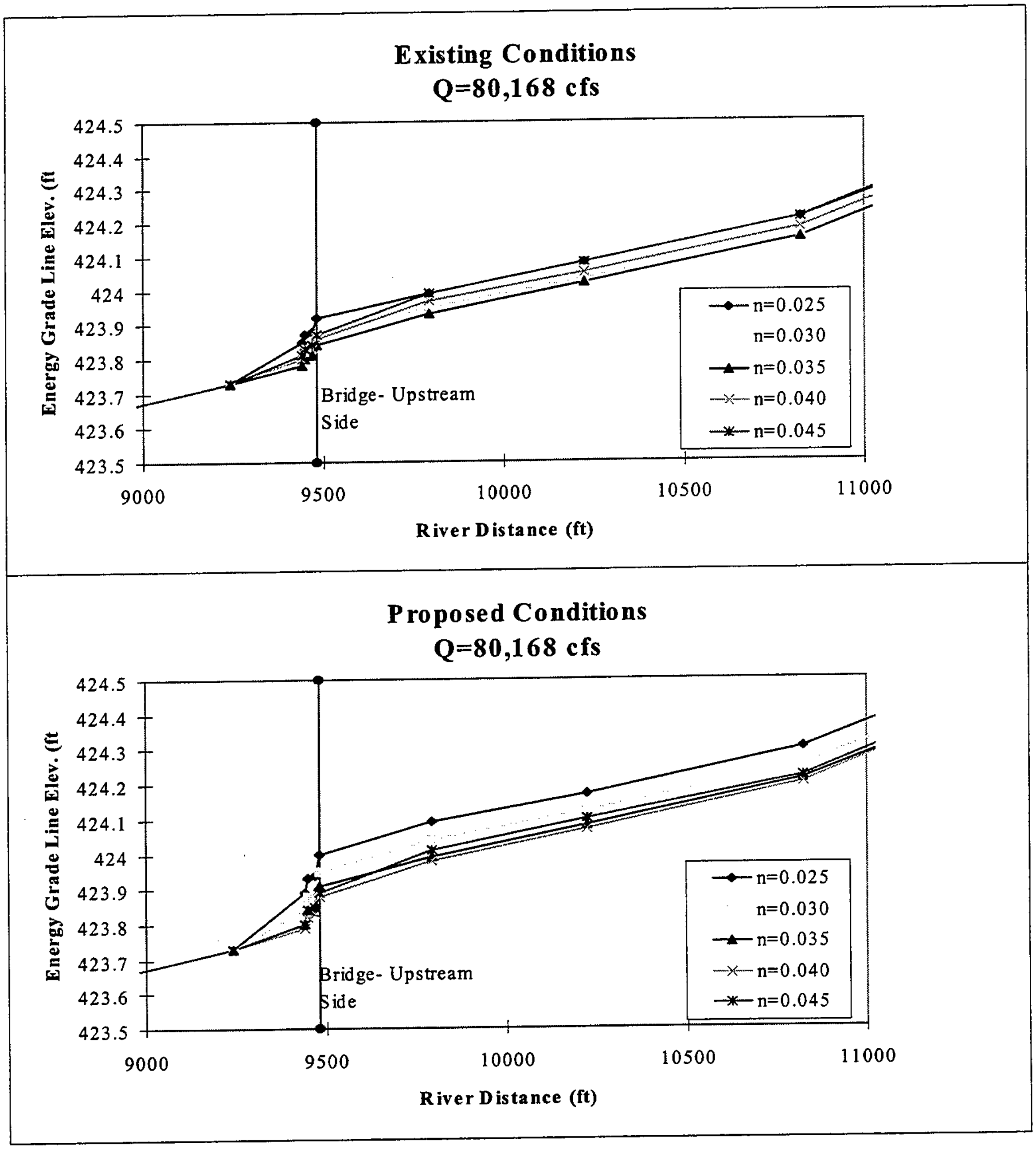


Figure 5.1. Energy Grade Line Elevation for Existing and Proposed Conditions of Meadowbrook Bridge.

more scouring, impose higher forces on structures, and cause higher damages. The energy grade line includes both flow depth and the velocity.

Figure 5.1 shows the energy grade line for Meadowbrook Bridge before and after the modifications to the bridge. The river is flowing from right to left on the graph, and the energy grade line is always decreasing in both cases.

The change in the energy grade line (EGL) is defined as the difference in energy grade line between the proposed and existing conditions of a bridge. Similarly to the calculation for change in water surface elevation calculation, the change in energy grade line was determined by holding Manning's n constant and varying the discharge for the lowest four predicted 100-year flow rates. Figure 5.2 shows the change in energy grade line based on changes in discharge.

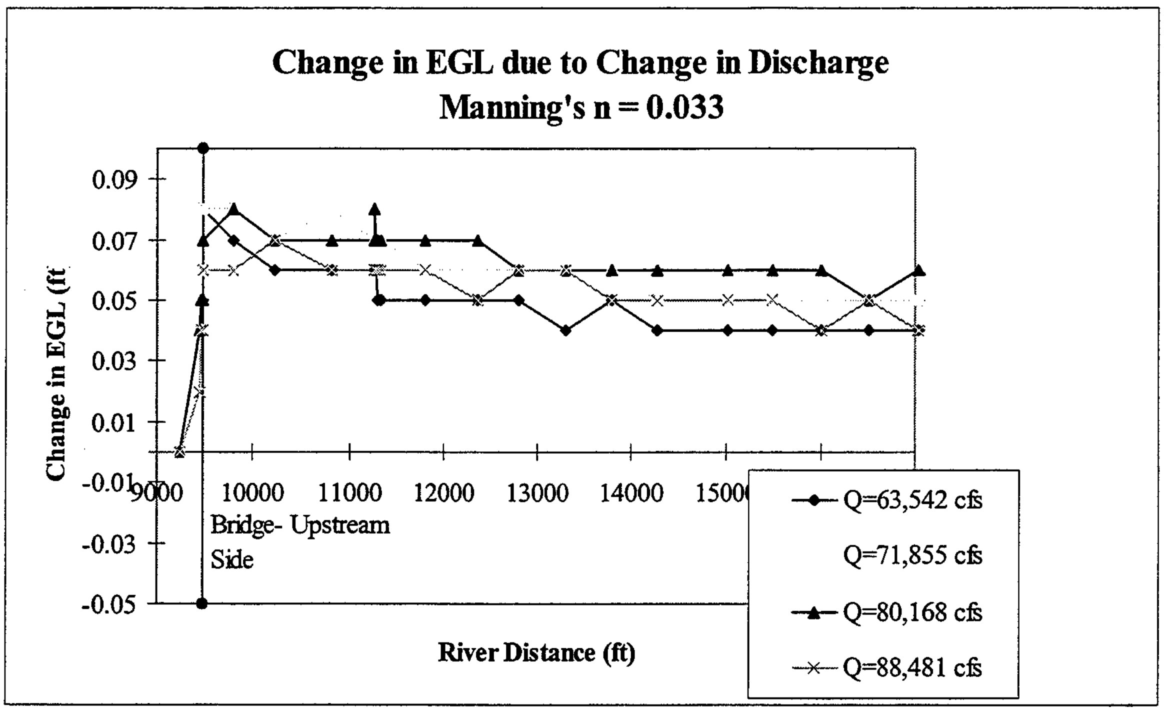


Figure 5.2. Change in Energy Grade Line (EGL) due to 100-year Peak Flow Variance.

Next, the change in EGL was calculated by holding the discharge constant at 80,168 cfs and varying Manning's n through the same five values used in the sensitivity analysis. Figure 5.3 shows the change in energy grade line based on changes in

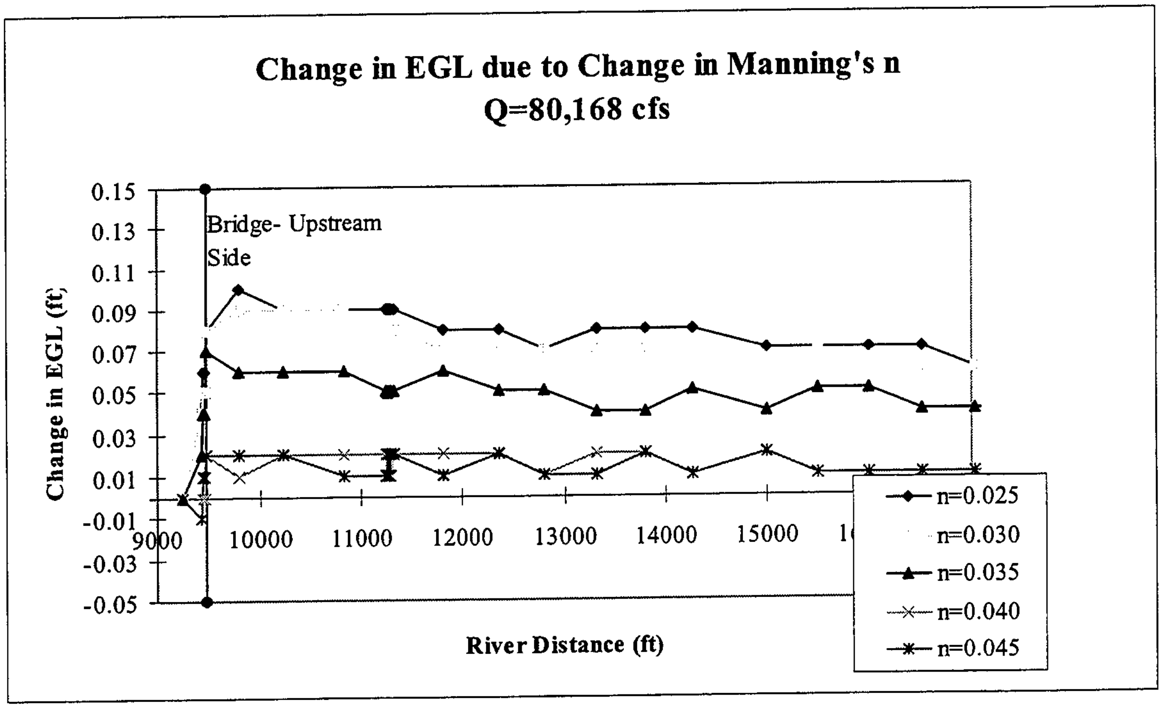


Figure 5.3. Change in Energy Grade Line (EGL) Due to Variance in Manning's n.

Manning's n. The maximum change in EGL due to variations in discharge and Manning's n at the critical section was 0.10 feet.

A law limiting the maximum allowable change in the energy grade line would be similar to a law limiting the maximum allowable change in water surface elevation. The limit would need to account for uncertainties inherent in the analysis. For the Meadowbrook Case study results, the maximum allowable change in energy grade line tolerance would need to be at least 0.10 feet. Although the change in energy grade line was not calculated for the two Cedar River case studies, the values would probably be greater because the larger change in water surface elevation. Therefore, a suitable tolerance for change in energy grade line would be on the order of 0.5 ft.

5.2 Options Based on Economic Factors

To examine the implications of the zero-rise ordinance on bridge modifications it is essential to look at both the beneficial and detrimental economic effects of the law. This section describes the economic analysis that was completed on Case Study 3, Meadowbrook Bridge, and describes alternatives that could be used to replace the zero-rise ordinance. The economic factors that were considered for the analysis included

- the cost of a bridge that does not meet the zero-rise requirement
- the cost of a bridge that does meet the zero-rise requirement
- the benefits of modifying the bridge
- the expected damages due to flooding inundation
- the change in damages due to backwater caused by a non-compliant bridge.

To compare these economic factors, the net present value for each of the five factors was calculated. The first two factors represented an initial construction cost that was assumed to be paid at the beginning of the first year and, as such, reflected present value. The other three factors represented annual costs that would occur each year throughout the lifespan of the bridge. These annual costs were converted to a present value with the following equation:

$$Present\ Value = Annual\ Value * \frac{(1+i)^n - 1}{i*(1+i)^n} \qquad \textbf{(5-1)}$$

The interest rate (i) was assumed to be 5 percent and n represented the bridge life, which was assumed to be 50 years.

The first three economic factors required rough estimates because the actual costs had not yet been determined. The costs of the originally planned modifications for the Meadowbrook Bridge were assumed to represent non-compliance with the zero-rise ordinance. These modifications included raising the lower bridge chord by 3 feet and decreasing the channel width by 5 percent to build stronger abutments. This was assumed to have an associated cost of $500,000. To meet the zero-rise requirement, Meadowbrook Bridge would have to be lengthened from 350 feet to 420 feet (Wood et al, 1997). At the time of this report, costs had not been estimated. So as an extremely rough estimate, the average percentage increase of 66 percent in cost for the seven King County bridges analyzed by Wood et al was used to predict the cost of building a bridge in compliance with the ordinance. Thus the cost of a Meadowbrook Bridge that would comply with the zero-rise ordinance was assumed to be $830,000. Because Meadowbrook Bridge would need to span a wide floodplain to meet the zero-rise criterion, $830,000 was probably a conservative estimate.

The cost of the third economic factor, the benefits of upgrading the bridge, would occur yearly throughout the life of the project. Meadowbrook Avenue was widened in 1993; however, the bridge was left intact and therefore remains narrow. Because the bridge is located on a collector arterial, widening the bridge would enhance safety for many people. Although it is difficult to place a monetary value on safety, for the purposes of this economic analysis the annual benefit of the upgraded bridge was assumed to be $10,000.

The expected damages of flooding and the change in damages caused by bridge backwater were determined by using historical streamflow data. The USGS Gauging Station #12144500 has recorded the maximum peak flows near the City of Snoqualmie for the last 36 years. These flow data were entered into the HEC-2 model for both the existing and the proposed conditions, assuming a Manning's n coefficient of 0.033. As mentioned previously, the HEC-2 output computes the water surface elevations for each section, as well as the limits of the channel width.

For each of the 36 flow rates, the expected damages due to flooding were calculated by the methodology described in Section 2.4. This represented the actual cost of flooding expected each year given the existing conditions of Meadowbrook Bridge. The costs were divided into land and structural damages. Previously, the costs of land damages were calculated on the basis of the difference in amount of land inundated before and after the bridge modifications. For this assessment, the land damages were calculated on the basis of the difference between the land inundated in the particular flood and the land inundated in a 2-year flood. The land inundated by the 2-year flood was assumed to have no associated value because it is inundated so frequently. Figure 5.4

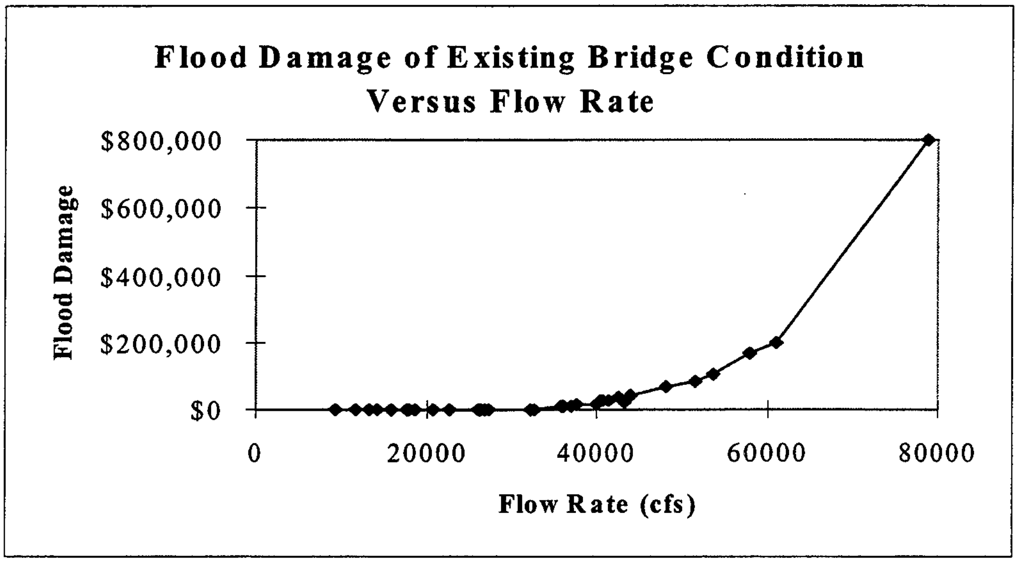


Figure 5.4 Expected Flood Damage Versus Flow Rate for the Existing Conditions of Meadowbrook Bridge (based on 36 years of peak flow data).

shows the expected flood damage caused by the 36 peak flow rates for the existing conditions of Meadowbrook Bridge. Logically, as the flow rate increases, the flood damage also increases.

To calculate the change in damages caused by the bridge modifications for the 36 flow rates, the cost of floods for the proposed conditions of Meadowbrook Bridge was calculated in the same manner as for the existing condition. The difference in these two values yielded the change in damages. Figure 5.5 displays the flow rate versus the change in damages due to bridge modifications. Between 0 and 60,000 cfs, an increase in flow rates corresponds to an increase in damages. However, as the flow increases beyond 60,000, the change in flood damages decreases. This decrease in flood damage was due to the same condition that produced almost no increase in water surface elevation with the largest modeled discharge rate in the sensitivity analysis of 96,794 cfs. As described in Section 3.3.2, for large discharge rates with the existing bridge, the water surface elevation would be greater than the low bridge chord elevation and should be modeled as pressure flow.

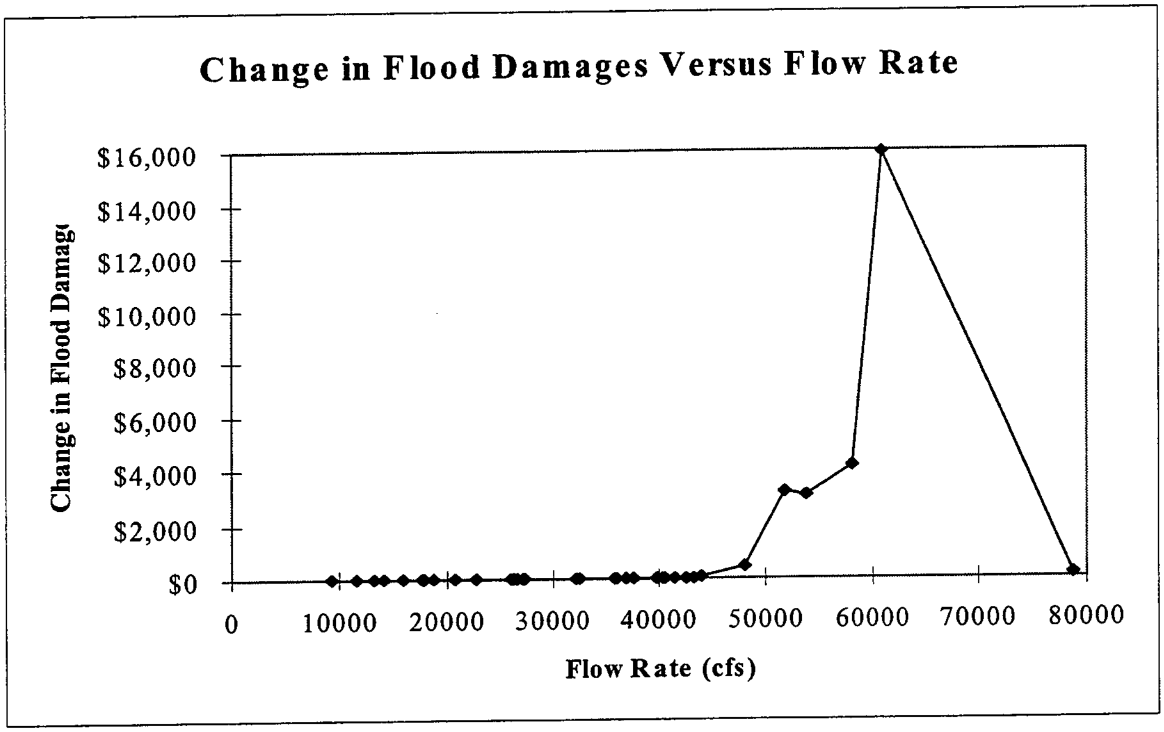


Figure 5.5 Change in Flood Damage Versus Flow Rate.

After the expected damages and the change in flood damage between the existing and proposed conditions had been calculated for the 36 peak flows, the next step was to calculate an annual total cost of flooding for each of the 36 years. The peak flow is an instantaneous maximum that does not reveal smaller floods that may have occurred during the year. Therefore, for the years with greater than 44,000 cfs of flow, the daily stream records were examined to determine how many additional floods had occurred. Below 44,000 cfs there are no calculated flood costs, as shown in Figure 5.4. The daily stream flow records only showed the flow averaged over one day, not the instantaneous peak flood that will cause the most damage. For each year in which flow was greater than 44,000 cfs, the maximum peak flood was compared to the daily mean record for the day of the flood. The average difference between these two values was then added to the averaged daily flow values and was considered to represent the maximum peak that occurred for the day. For Snoqualmie River, the value of this difference was 15,000 cfs.

For years that showed smaller, significant floods, the additional costs of damages were added onto the total cost of flooding for the year. The cost of smaller floods was determined from Figure 5.4. This same process was followed to calculate the total change in flooding due to bridge modifications for one year. The change in flood damage for a flow rate was determined from Figure 5.5. Using this information, the total cost of flooding and total change in flooding cost for each of the 36 years were determined.

For the economic analysis, the flooding costs had to be represented as an average cost occurring yearly. In reality, the actual costs vary each year. The annual flooding costs were determined as the mean of the 36 years of flood costs. Thus, the flood damage expected each year and the change in flood damage due to bridge modifications were annualized and then used for a 50-year bridge life span.

Table 5-1. Results of Economic Analysis for Meadowbrook Bridge.

Economic Factor	Initial Cost	Annual Cost	Present Value
Costs of Compliance			
• Construction Costs of Bridge	($830,000)	N/A	($830,000)
• Benefits of Improved Bridge	N/A	$10,000	$182,559
• Expected Flood Damages	N/A	$54,700	($998,599)
Total			($1,646,040)
Costs of Non-Compliance			
• Construction Cost of Bridge	($500,000)	N/A	($500,000)
• Benefits of Improved Bridge	N/A	$10,000	$182,559
• Expected Flood Damages with Compliant Bridge	N/A	($54,700)	($998,599)
• Increased Damages due to Non-Compliance	N/A	($1250)	($22,820)
Total			($1,338,860)

Table 5.1 displays the initial costs and the annual costs for the economic factors calculated for Meadowbrook Bridge. The table also shows the present value of each factor calculated by using a 50-year life span with a 5 percent interest rate.

5.2.1 Total Cost Analysis

The total cost was calculated for both a bridge in compliance and a bridge in non-compliance by using the present value of the economic factors listed in Table 5.1 for Meadowbrook Bridge. A bridge in compliance would create less than 0.01 foot of backwater, whereas a bridge in non-compliance would create more. The total cost was the combined costs minus the benefits. For a bridge in compliance with the zero-rise ordinance, the costs would include the compliant bridge construction cost of $830,000 and the expected flood costs of $998,599. This yields a cost of $1,828,599. When the benefits of an improved bridge of $182,559 was subtracted from this value, the net cost of a bridge in compliance with the zero-rise ordinance was determined to be $1,646,040.

Similarly, the cost for a bridge in non-compliance with the zero-rise ordinance was calculated. The cost would include the sum of the non-compliant bridge construction cost of $500,000, the expected flood damages of $998,599, and the change in damages caused by backwater rise of $22,820. This totals to $1,521,419. The total cost for a bridge in non-compliance was this value minus the benefits of $182,559, or $1,338,860.

5.2.2 Marginal Change in Expected Flood Damage.

The comparison of expected annual flood damages versus the increase in flood damages created by bridges with backwater shows the relative magnitude of increased flood costs. For instance, from Table 5.1, the expected cost of flood damages would be $998,599 for a 50-year period, whereas the change in flood damage would be $22,820 for Meadowbrook Bridge modifications. The backwater rise would increase the flood damages by 2.3 percent. This compares the impact of flooding caused by backwater rise to the total cost of flooding.

5.2.3 Additional Bridge Costs Versus Change in Flood Damages

Another economic evaluation compares the increased bridge costs with the increase in flood damages due to non-compliance. If a bridge is in compliance, there should be no increase in flood damages. Therefore, the difference in cost between compliance and non-compliance bridges should balance with the damages prevented--or the increase in flood damages. For Meadowbrook Bridge, as shown in Table 5.1, the cost of compliance would be $830,000, and the cost of non-compliance would be $500,000. The additional cost necessary to comply with the zero-rise ordinance would be $330,000. By paying this extra $330,000, an increase in flood damages of $22,820 would be avoided. The flood damage would be only 7 percent of the additional bridge costs.

Maw (1995) completed a similar analysis for two case studies. The increase in damages caused by backwater rise was only determined for the 100-year flood; however, this cost was not averaged over the expected bridge-life. The present worth of the average flood damages was determined to actually be less than the actual 100-year expected increase in flood damages. Nevertheless, this comparison is still useful for understanding the relative magnitude of the difference between the increased bridge costs and the amount of flood damages. For Maw's first case study, the cost of a bridge in compliance was approximately $1,660,000 greater than the cost of non-compliant bridge. The damages associated with backwater rise from the non-compliant bridge were determined to be $5,500, only 0.3 percent of the additional bridge costs. Likewise for Maw's second case study, the difference between a bridge in compliance and one in non-compliance was between $2 and $4 million. The damages associated with backwater rise was approximately $33,000, only 0.8 percent to 1.6 percent of the additional bridge costs.

5.3 Watershed Management Laws

Because the zero-rise ordinance restricts development in the floodplain, only undeveloped areas will receive substantial benefits from this law. Areas that have already been developed will still experience high flood damage costs, and the ordinance will not significantly reduce the amount of flood damages. This law is aimed at preventing future flood damages instead of reducing the damages that would occur during a flood today.

To reduce the amount of flood damages in developed areas, flood management laws may need to be re-examined to reduce the volume of runoff throughout the entire watershed. As shown in Figure 5.4, the amount of flood damages is highly dependent on the flow rate. In the determining the 100-year flood, the hydrology of the watershed was assumed to remain constant. However, this may not be a valid assumption. Land-use practices in the catchment area might greatly affect the magnitude of the 100-year flood. Practices such as forest harvesting and urbanization, which produce increased runoff, could greatly influence the amount of discharge to a river. The flood damages caused by increases in discharge could easily be greater than the damages produced by a small amount of backwater rise.

Before the influence of land-use practices on the magnitude of the 100-year peak flow can be determined, more research is needed. It is difficult to complete experiments that predict the influence on an entire basin from an event that only occurs once every 100 years. The change in water surface elevation cannot be readily assessed. Therefore although land-use practices may more heavily influence the amount of total damages than limiting future development, this type of law was not evaluated any further in this case study.

5.4 Summary and Recommendation

Table 5.2 summarizes the tradeoffs between limitation based laws and laws based on economic factors. The best approach is identified for each metric of performance. The laws based on economic factors rate more highly than the limitation based laws for four of the five metrics. The laws based on economic factors are superior for technical feasibility, cost effectiveness, environmental protection, and maintenance of public safety. However, laws based on economic factors are not as good as limitation based laws for reducing the amount of flooding. Limitation based laws attempt to reduce flood damages without considering the other factors involved.

Table 5.2 **Summary of Limitation Based Laws and Laws Based on Economic Factors.**

	Technically Feasible	Reduce Flooding	Cost Effectiveness	Environmental Protection	Maintenance of Public Safety
Limitation Based Laws	Can sometimes be feasible depending on the selected law	Good	Low	Design options could be selected which would meet law and yet be environmentally unsound	Low
Laws Based on Economic Factors	Yes	Balance between flooding and public safety	High	Better than some of the design options for limitation based laws	Balance between safety and flood reduction
Best Option	Economic	Limitation	Economic	Economic	Economic

A possible best solution for this complex issue would combine the two types of laws. A limitation based law should be set to encourage bridges and other structures to minimize the increase in flood damages. However, the limitation law needs to be at a tolerance that is technically feasible to implement. Additionally, exemptions based on economics should be made for public structures.

6.0 CONCLUSIONS

6.1 Economic Trade-off of Compliance and Non-Compliance

To quantify the impacts of non-compliance with the zero-rise ordinance, the cost of additional damages due to flooding was compared with the cost of constructing a single span bridge in compliance with the ordinance. The conclusions drawn from the three case studies used in this analysis are as follows:

¶ When the cost of not complying with the zero-rise ordinance was assessed as the additional structural and property damages incurred because of bridge modifications from a single 100-year flood event, the costs of compliance for Case Studies 1, 2, and 3 exceeded the costs associated with non-compliance.

- For the Cedar River Bridge, Case Study 1, an additional $1.66 million would be required for compliance with the zero-rise ordinance. The one-time 100-year flood damages associated with a bridge constructed in the floodplain would be approximately $5,500.
- For the Elliott Bridge, Case Study 2, an additional $2 to $4 million would be required to construct a single span compliant bridge. The one-time 100-year flood damages associated with a bridge constructed in the floodplain would be approximately $33,000.
- For the Meadowbrook Bridge, Case Study 3, it would not be practical to construct a single span bridge across the floodplain because of its great width near the town of Snoqualmie. The maximum damages associated with the 100-year flood event would be approximately $55,000.

¶ When the cost of not complying with the zero-rise ordinance was assessed as the additional structural and property damages incurred because of bridge modifications given an expected damage value calculated from 36 years of historical peak flows, the additional annual damage cost for the Meadowbrook Bridge, Case Study 3, was $1250. This represents a 2.3 percent increase from the annual damage cost of $54,700 without bridge modifications.

¶ The results of this preliminary impact analysis are only a marginal assessment of the damages associated with the construction of a bridge in the floodplain. This analysis provides a rough estimate of the tradeoffs associated with the zero-rise ordinance. To complete a more detailed investigation of the tradeoffs of this ordinance, the damage assessment should include the entire reach of river under study.

¶ The relative amount of land damages versus structural damages depends on the particular location of the case study. In the Case Studies 1 and 2, the estimated costs associated with land damages were very small in comparison to structural damages. Maximum land damages would be on the order of hundreds of dollars for the first case study and there are no land damages were associated with the second case study. In the third case study, on the other hand, the costs associated with land damages were very large in comparison to structural damages. The maximum land damages would be on the order of hundreds of thousand dollars because the relatively flat floodplain allows wide overflow to undeveloped areas.

¶ The results of the impact analysis showed that, for all three case studies, the potential benefits of maintaining the state's infrastructure by replacing, widening, or building new bridges and roads should be considered when development within the floodplains is restricted. The additional costs for compliance with the zero-rise ordinance were found to be uniformly much greater than the marginal cost of damages due to additional flooding. Figure 6.1 shows the relative costs of compliance with the zero-rise ordinance versus non-compliance for all three case studies. All three of the case studies are examples of why a zero tolerance regulation may not be cost effective in practice.

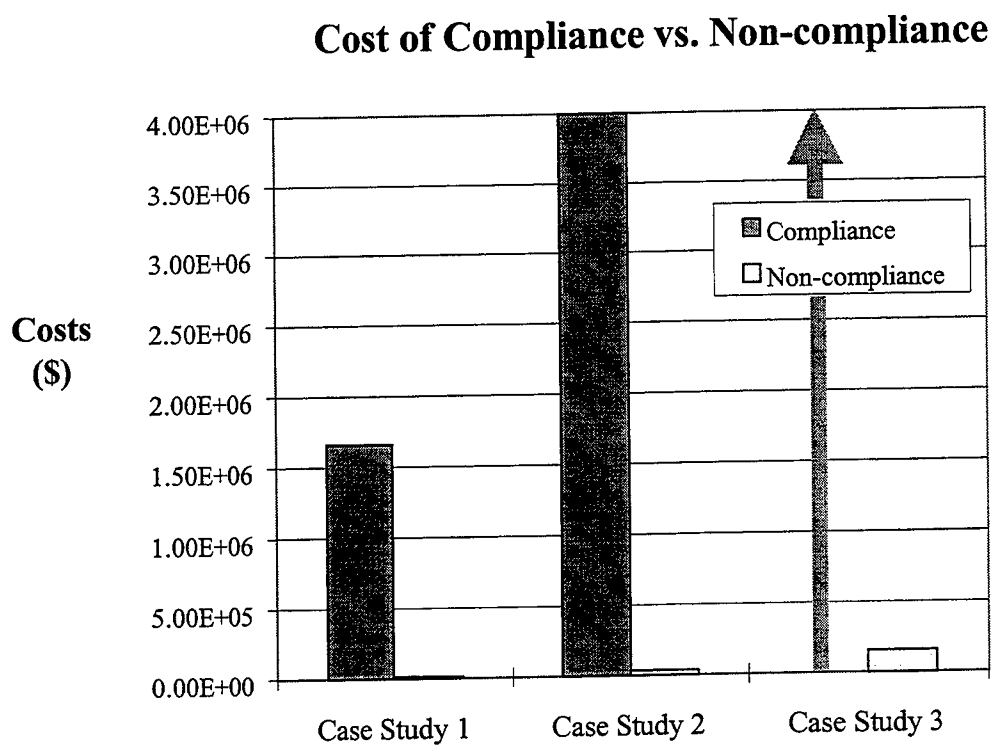


Figure 6.1. Costs of Compliance vs. Non-Compliance for Case Studies 1, 2, and 3. There is no cost of compliance available for Case Study 3 since the construction of a bridge across the entire floodplain is not feasible in this area.

6.2 Ability to Predict, Model, and Measure Changes

The Zero Floodplain Rise Ordinance requires a high degree of accuracy in modeling, predicting, and measuring changes in the water surface elevation caused by development in the floodplain. The conclusions drawn from the case studies are as follows.

<u>6.2.1 Model Response to Uncertainty in Input Variables</u>

Because of the uncertainties in input parameters, it is difficult for engineers to predict changes in the water surface elevations to within 0.01 ft.

- Extremely precise predictions of channel roughness are difficult to obtain because it is costly and time-consuming to verify roughness values with extensive field data and numerical simulations.

- Estimates of discharge in rivers rely on limited time series data for peak flows. As a result, the predicted values for low probability events such as the 100-year peak flow are subject to a significant amount of uncertainty.

The range of variation for predicted backwater elevations, given realistic levels of uncertainty in inputs, can be an order of magnitude larger than the 0.01-ft limit.

- Using a 95 percent confidence interval for a range of discharge values for the 100-year peak flow, maximum variations for predicted rise of 0.08 ft., 0.42 ft., 0.04 ft. and 3.05 ft. were found for case studies 1, 2, 3 and 4, respectively.
- The maximum water surface variations associated with varying the channel roughness for case studies 1 to 5 were 0.16 ft., 0.08 ft., 0.10 ft,. 1.9 ft., and 0.03 ft., respectively.
- The maximum variations in predicted rise associated with a combination of uncertainty in discharge and in channel roughness were 0.29 ft., 0.67 ft., 0.11 ft., 1.7 ft., and 0.03 ft. for case studies 1 to 5.

An additional point of note is that, for Case Study 4, the construction of a bridge that would improve the flow conveyance would cause local changes in the water surface that could inundate areas that were not previously in the 100-year floodplain. While the wider bridge opening would lower the entire level of the floodplain, the local topography and the existence of levees could change the flooding pattern. Also, it is conceivable that the removal of a floodplain obstruction such as a bridge may cause a rise in the floodplain downstream of the bridge. This was not observed in the five case studies in this report but is nonetheless a possibility.

The results of the sensitivity analyses for the five case studies are summarized in Table 6.1.

Table 6.1. Summary of Sensitivity Analysis Results.

	Case Study 1	Case Study 2	Case Study 3	Case Study 4	Case Study 5
Location	SR 169 at Cedar River	Elliot Ave at Cedar River	Meadow-brook Bridge at Snoq.	Bridge #234A at Raging River	SR-18 Bridge at Green River
Drainage Basin Area	104 sq. mi.	104 sq. miles	375 sq. miles	30.6 sq. miles	399 sq. miles
Slope	0.0006	0.0006	0.006	0.0004	0.004
100-year Peak Flow	11,632 cfs	11,632 cfs	80,168 cfs	7,413 cfs	12,000 cfs
Standard Deviation of Discharge	1198 cfs	1198 cfs	8313 cfs	1118 cfs	0 cfs
Critical Section	1404	2300	9792	0560	33.764
Maximum Change in Water Surface Elevation	0.29 ft	0.67 ft	0.11 ft	1.7 ft	0.03 ft
Maximum Range of Variation in Change in WSEL Prediction due to Q	0.08 ft	0.42 ft	0.04 ft	3.05 ft	N/A
Maximum Range of Variation in Change in WSEL Prediction due to Manning's n	0.16 ft	0.08 ft	0.10 ft	1.9 ft	0.03 ft
Maximum Range of Variation Due to Pier Size at Mid-Channel per ft. Change in Pier Size	0.13-0.16 ft	0.10-0.15 ft	N/A	N/A	N/A
Maximum Range of Variation Due to Pier Location for 6 ft. Pier	0.10-0.40 ft	0.60 ft	N/A	N/A	N/A
Hydraulic Input Variable of Most Sensitivity	Manning's n	100-year Discharge	Manning's n	100-year Discharge	Manning's n
Input Variable of Most Sensitivity	Pier Location	Pier Location	Manning's n	100-year Discharge	Manning's n

6.2.2 Modeling Limitations

The choice of a numerical model to predict water surface elevations limits the accuracy of the predictions. The topography of the river reach and the timing of the flood flow hydrograph must fall well within the assumptions of the numerical model for it to yield reasonable results.

¶ The U.S. Army Corps of Engineers, Hydraulic Engineering Center, the authors of HEC-2, have reported that this model is only accurate to within 0.5 ft. for the types of survey data and the characteristics of rivers typically found in King County. In Case Study 5, for which the Manning coefficients were estimated by engineering judgment, the level of the floodplain was found to be greater than 0.5 feet.

¶ The model used for these case studies, HEC-2, is a one-dimensional model for steady, gradually spatially varied flow. Because this type of flow is not fully representative of the flows found in the case studies, HEC-2 can not be expected to model changes in the water surface elevation to 0.01 ft.

¶ As demonstrated in the second and fourth case studies, some river reaches are either not well-modeled by HEC-2 or are extremely sensitive to small changes in input. These include areas with sharp bends or multiple channels, areas where the bridge design causes flow asymmetry in the channel cross-section, or reaches that contain abrupt transitions between leveed and unleveed cross-sections.

¶ The use of more complex two-dimensional models, while feasible, requires that more time and expense be devoted to the collection of field data and model calibration. While input data for one-dimensional models such as HEC-2 are usually available for a site, the data may not be sufficient to construct an accurate two-dimensional model.

6.3 Alternatives to the Zero Rise Ordinance

The alternative options to the zero-rise ordinance fall into one of three categories:

- limitation based laws

- laws based on economic factors
- broad-scale watershed management laws.

Although a specific evaluation of watershed management was not performed in this study, regulations based on economic analysis were compared to limitation based regulations for five metrics of performance. The laws based on economic factors rated more highly than the limitation based laws for four of the five metrics. The laws based on economic factors are superior for technical feasibility, cost effectiveness, environmental protection, and maintenance of public safety. Limitation based laws, however, perform better at reducing the amount of flooding. The analysis suggested that, given a limitation law that is set to a technically enforceable tolerance, exemptions based on economics should be made for public structures.

7.0 REFERENCES

Burges, S. J. (1979). "Analysis of Uncertainty in Flood Plain Mapping." *Water Resources Bulletin*. American Water Resources Association, 15(1), 227-243.

Department of the Army (1992a). *Guidelines for Risk and Uncertainty Analysis in Water Resources Planning, Volume I - Principles*. Institute of Water Resources, IWR Report 92-R-1.

Heijne, I. S. (1992). "Fluvial Flooding: Decision Making in Floodplain Management." *Fluid Mechanics & Its Applications*. Kluwer Academic Publishers, Vol. 15.

Henderson, F. M. (1966). *Open Channel Flow*. Macmillan Publishing Co., Inc., New York, New York.

Jarrett, R. D. (1984). "Hydraulics of High Gradient Streams." *Journal of Hydraulic Engineering*, ASCE, 110(11), 1519-1539.

King County (1990). *King County Sensitive Area Ordinance #9614*. Section 81-84.

Maidment, D.R. (1993). *Handbook of Hydrology*. McGraw-Hill Inc., New York, New York.

Maw, G., (1995). *An Analysis of the Implementation of the Zero Floodplain Rise Sensitive Area Ordinance*. Master's Thesis, University of Washington Dept. of Civil Engineering.

Owens, J. and Rudd, R. (1991). *Comprehensive Planning for Flood Hazard Management Guidebook*. W. S. Department of Ecology, Publication No. 91-44.

U.S. Army Corps of Engineers (1986). *Accuracy of Computed Water Surface Profiles*. Hydrologic Engineering Center, Technical Paper No. 114.

U.S. Army Corps of Engineers (1992). *Computing Water Surface Profiles with HEC-2 on a Personal Computer*. Hydrologic Engineering Center, Technical Paper No. 114.

U.S. Army Corps of Engineers (1995). Personal Communication. Hydrologic Engineering Center, Davis, California.

Woods A., Palmer R., Petroff C. (1997). "Comparison and Assessment of Zero-Rise Floodplain Ordinances." *Journal of Water Resources Planning and Management*. ASCE, 123(4), 239-245.

8.0 BIBLIOGRAPHY

Department of the Army (1992b). *Guidelines for Risk and Uncertainty Analysis in Water Resources Planning, Volume II - Examples*. Institute of Water Resources, IWR Report 92-R-2.

Department of the Army (1994). *Risk-Based Analysis for Evaluation of Hydrology/Hydraulics and Economics in Flood Damage Reduction Studies*. U. S. Army Corps of Engineers, Circular No. 1105-2-205.

Federal Insurance Administration (1987). *Depth/Damage Curves for One Story Wooden Homes without Basements,* U. S. Army Corps of Engineers.

King County Department of Public Works (1992). *Elliott Bridge Replacement Draft Environmental Impact Statement and Draft Section 4 (f) Evaluation.*

King County Department of Public Works (1994), *Raging River Bridge No. 234A, Bridge Type, Size and Location Study*, Prepared by Sverdrup Civil Inc. et al.

Platt, R. H. (1987). *Regional Management of Metropolitan Floodplains*. University of Colorado Institute of Behavioral Science Library of Congress.

Prasuhn, Alan (1987). *Fundamentals of Hydraulic Engineering*. Holt, Rinehart, and Winston.

Saul, A. J. (1992). *Floods and Flood Management*. Kluwer Academic Publishers, Dordrecht, Boston.

U.S. Army Corps of Engineers (1982). *Water Surface Profiles User's Manual.* Hydrologic Engineering Center, Technical Paper No. 114.

Washington State Department of Transportation (1993). *Design Report Supplement No. 1. SR 169, MP 19.21 to MP 23.90, 196th Avenue SE/Jones Road to Maplewood.* 0L7228, PIN-1169271, 0L8607, PIN-116923A.

Washington State Department of Transportation (1992). *Flood Plain Evaluation Report – Green River at SR 18.* Prepared by CH2MHILL.

Williams, J. R., Pearson, H. E., Wilson, J. D. (1985). *Streamflow statistics and drainage-basin characteristics for the Puget Sound Region, Washington.* U.S. Dept. of Interior, Geological Survey.