BACKGROUND

Snohomish County Surface Water Management (SWM) received an FCAAP grant for assessing the risk of a river channel avulsion in the vicinity of the Dike Road dike, which is on the left bank of the Stillaguamish River just downstream of the confluence of the two forks (Figure 1). SWM contracted with R2 Resource Consultants, Inc. (R2) to conduct this assessment.

The Dike Road dike was constructed between 1935 and 1937. The river flows through a confined bridge opening at the confluence and then westerly between two dikes. The upper left bank dike section is on City of Arlington land and is referred to as the Johnson levee. The right bank dike runs along Schloman Road and was constructed circa 1948 according to County records. Where Dike Road reaches the river the dike consists of a U.S. Army Corps of Engineers (Corps) revetment with the County berm and road on top. Dike Road Dike overtops during flood events that exceed approximately 19 feet at Arlington (flood stage is 14 feet at Arlington). In 2002, both the Dike Road dike and the Johnson levee overtopped. The County desires to evaluate alternatives to reduce the risk of avulsion during a flood event, and select a preferred alternative. This memorandum describes an analysis conducted to identify the risk at various locations.

EXECUTIVE SUMMARY

Hydraulic and geomorphic analyses were conducted to assess avulsion risk in the Stillaguamish River project area. A 2-dimensional finite element model was constructed of the river and floodplain below the SR 9 bridge; details of the modeling are provided as an appendix. As a result of the analyses, six (6) locations were identified along the Dike Road dike and Johnson levee as being associated with highest risk of erosion and subsequent avulsion. Twelve (12) swale segments were identified as primary floodplain drainage paths associated with varying potentials for future avulsion. The dike and swale locations are depicted in Figure 2, and were assigned relative risk ratings for a number of hydraulic and physiographic factors influencing avulsion risk in response to the 100-year flood event. Of these locations, the vicinity of the
junction of Dike Road and 59th Ave NE was determined to be associated with the greatest risk of dike failure and subsequent channel erosion downgradient. The overall risk of a single event avulsion occurring through any path was determined to be low. Proactive measures are suggested that may mitigate the risk at this and other locations, including protecting selected road prisms from scouring and undermining, and ensuring that adequate vegetative cover is maintained within the primary drainage swales during and one month prior to the flood season in order to prevent erosion.

![Dike Road dike and project area along the Stillaguamish River near Arlington, Washington.](image)

**Figure 1.** Location of Dike Road dike and project area along the Stillaguamish River near Arlington, Washington.

**PROBLEM DEFINITION**

The typical process for channel avulsion in a meandering river such as the Stillaguamish River involves first an increase in channel sinuosity, which results in lengthening the channel. As the meander amplitude increases, the water surface slope decreases, and the slope between successive crossover points increases. The risk of avulsion increases as cross-over points become more proximal and the shortest-distance water surface slope between the points is
greater than the channel slope. Overall, the potential for avulsion occurring during large floods depends on the following factors:

1. Depth and velocity of flow over bank protection works;
2. Width of flow over bank protection works and adjacent floodplain;
3. Material integrity of bank protection works, at toe (riverside) and backside;
4. Height of bank protection works above floodplain water surface elevation
5. Floodplain surface erodibility characteristics (soil and vegetation);
6. Slope, depth, and velocity over floodplain (as influenced by floodplain topography and roughness, flood magnitude and duration);
7. Presence of grade control (hard points) on floodplain; and
8. Increase in flood levels for a given recurrence interval due to hydrology or local sediment transport changes.

Avulsion risk can be evaluated in terms of each of these factors and the collective results compared to identify the most likely locations where either a complete or partial avulsion channel might develop. Methods are summarized below, followed by results for each factor. Based on the results, we identify locations more likely to require flood protection work.

**METHODS**

The analysis consisted of four primary components:

1. A 2-dimensional hydrodynamic model was constructed of the project area to simulate topography, depths, velocities, and shear stresses over the floodplain;
2. River and gravel bar margins were traced from aerial photographs in ARC-GIS, for five different years spanning the period 1933-2003;
3. Cross-sections were compared for five transects, using data collected on three different occasions; and
4. A field visit was conducted to assess floodplain and dike conditions and obtain anecdotal evidence of flood levels, flow directions, and flood damage.

**2-D Hydrodynamic Model**

The model reach extends from the SR 9 bridge to approximately RM 13 in the mainstem Stillaguamish River where the channel is nearest to SR 530. The model was developed using
SMS 8.0, which integrates RMA2 with a user-friendly interface for data input/output operations. RMA2 is a two-dimensional depth-averaged finite-element model supported by the U.S. Army Corps of Engineers. The model is an extension to the earlier effort by R2 to model flood hydraulics in the vicinity of the Hallar Trestle. The results of Hallar Trestle analysis are described in a separate Technical Memorandum dated 3/3/05 (Snohomish County Master Contract No. OC07-8-03, WA #2).

The development and analysis of the 2-D model is described in greater detail in the Appendix at the end of this memorandum. In summary, the 2-D finite-element mesh was constructed using bathymetric, LIDAR, RTK, total station and surveyed cross-section data. All data were converted to the same datums for input into RMA2. The vertical datum was NAVD 88, and horizontal datum NAD 83. A baseline model mesh was developed to simulate existing conditions, and two revised meshes were also constructed to simulate two dike-breaching alternatives:

1. Current conditions with a breach in the Dike Road dike just north of the Dike Road/59th Ave NE junction, at a location of recent road overtopping; and
2. Current conditions with a breach in the Dike Road dike near its upstream end, again at a location of recent road overtopping.

The locations of the breaches were identified after comparing the existing conditions model run with the geomorphic analysis results. After consultation with SWM staff, the breach width was chosen to match the resolution of the model mesh size, at approximately 150 feet wide each. The breaches were assumed to have eroded down to the level of the floodplain at the toe of the road prism.

Upstream and downstream model hydraulic boundary conditions were defined using a HEC-RAS model provided by SWM. The boundary conditions for the model runs were obtained from the HEC-RAS model. Flows for the 100-year and other recurrence interval event magnitudes were taken from Collins and Turner (2003).

The model output was used to classify floodplain surface areas in terms of mean column velocity and shear stress distributions. This information was compared with design criteria for vegetated channels and bare soil/newly seeded surfaces. Values presented in Chow (1959) were used as screening level criteria. For vegetated channels, a permissible velocity of 5 ft/s was used, which corresponds to a grass mixture growing on a low slope, relatively cohesive floodplain. For bare earth/newly seeded surfaces, a US Bureau of Reclamation shear stress design criterion of 0.05 lb/ft² was used.
Aerial Photograph Interpretation of Channel Planform

River channel and gravel bar margins were traced in ARC-GIS using orthocorrected aerial photographs for the years 1933, 1955, 1974, 1991, and 2003. These years provided a reasonable time span over which to evaluate channel migration. The traces were compared for successive years, and for all years combined to visually identify locations that were most and least active. The results were compared with plots of cross-section changes to identify morphologic changes that might influence spatial flooding patterns.

Cross-Section Profile Change

Cross-section profile data were compared from three periods. Data sources included:

- Corps flood maps from the 1970s;
- County survey data collected in 1997; and
- March 2005 LIDAR data used to develop the RMA2 model mesh.

The cross-section data were compared with analyses conducted as part of a gravel budget study of the Stillaguamish River (Collins 1992). That study was also reviewed for information pertinent to general deposition and erosion processes in the project reach.

Field Inspection

A field survey was conducted on 6/16/05. The survey included a visual inspection of the Dike Road dike prism at various locations to ground-truth surface erosion risk associated with overtopping. Side slopes, materials, and evidence of previous flood overtopping were noted. The survey also involved touring the project area, including along the Johnson and Schloman Road levees, and talking with locals about flood levels and flood damage during the 1990 and 2002 flood event. Other features were noted, such as presence of roads and channels over the floodplain, and vegetation characteristics.

RESULTS

LIDAR Data and Topography

The LIDAR data indicated topographic channels that are most likely to convey overbank flow. The locations of these channels and where they meet the river channel along the left bank dike are indicated in Figure 2. Longitudinal profiles along each channel are depicted in Figure 3.
Figure 2. Dominant floodplain swale flow paths for overbank flow. Dashed lines indicate secondary flow paths with lower likelihood of channel avulsion. Avulsion channel head locations are numbered for discussion in text. Circled numbers denote dike locations, numbers in squares denote swale segments discussed in text.

Six locations were identified along the Dike Road and Johnson Dikes where overbank flow would tend to concentrate, thereby increasing breaching and avulsion risk locally in the vicinity of the dike. Two other secondary channels were identified but were considered to be of low risk based on their greater distance from the river channel and presence of topographic divides restricting the amount of through-flow.

Of the six dike locations, the Johnson Dike location (number 1 in Figure 2) was considered to be of lowest relative risk of erosion leading to avulsion in the vicinity of a breach. There is an existing breach at Location 1 that is approximately 3 feet deep and 10 feet wide, but the invert of the breach appears to be around 2 feet or so higher than the adjacent floodplain, which is relatively level. In addition, Dike Road forms a downstream grade control (elevation ~61.1 ft, NAVD 88) that may restrict the flow rate through the existing breach which should prevent the breach from becoming significantly larger. The Dike Road surface is not substantially elevated above the floodplain and the downstream side slope of the road prism is gentle. These topographic factors contribute to a relatively low risk of avulsion in the vicinity of the dike.
because they limit the erosive forces locally in the vicinity of the dike and the road. There is an increased risk a short distance along swale segment 7 downstream of Dike Road where the longitudinal slope is approximately 0.4%, but the risk may be mitigated by the rise in elevation farther downstream again to the junction with swale segment 6 (Figure 3).

Dike location 2 in Figure 2 has a moderate risk of erosion leading to avulsion in the vicinity of a breach. There is also a low point in the road locally at this location. The floodplain slope of segments 5 and 6 is approximately 0.2% (Figure 3). Dike locations 3, 4 and 5 are associated with a high avulsion risk in the vicinity of the dike, on the basis that the floodplain swale gradients are generally around 0.5% and greater (Figure 3).

The risk of avulsion downgradient of dike location 6 in Figure 2 is also considered high because the slope in the upper half of the swale exceeds 1% (Figure 3), which is relatively steep. However, the lower half has a much gentler gradient and water could conceivably flow into the large gravel pit, further reducing flow strength in the lower gradient section. Hence, the overall risk is considered moderate.

Figure 3. Longitudinal profiles of swale segments identified in Figure 2. Numbers in legend correspond to consecutive segments in the upstream direction.
The topographic data indicate that avulsion risk may be relatively low for swale segments 1, 2, 3, 4, and 9 because their slopes are lowest there compared with farther upstream, around 0.1 percent. Conversations with local farm workers indicate that swale segment 3 did not erode during the 2002 or 1990 floods. Culverts placed in this swale segment reportedly have blown out during large flood events including the 2002 event, but erosion was said to have been restricted locally to the structure; the remainder of the swale vegetation and substrate remained intact.

A moderating factor for swale segments 5, 6, 7, and 8 is provided by the presence of 59th Ave NE, which would be expected to provide grade control for flood levels upstream, as well as a hard point against channel incision in the floodplain. There is no culvert at the location where segments 5 and 8 join, so flood water must flow over the road prism. However, the road is elevated by only a few feet at this location relative to the floodplain, so there is a reduced risk of the road prism failing due to overflow-induced erosion. The low point on the road is around elevation 54.5 ft according to the LIDAR data.

The Schloman Road dike appears to be higher in elevation than the Johnson Dike, and is of comparable elevation to the right bank floodplain. The avulsion risk is thus relatively low along the right bank upstream of RM 17.2. Flood protection work priority accordingly appears highest along the left bank of the Stillaguamish River.

2-D Modeling

The existing condition model run predicted that most of the Dike Road dike was dry during the 100-year flood event for the modeled boundary conditions, and that the dike was most likely to be overtopped at dike locations 1, 2, 5, 3, 4, and 6 depicted in Figure 2, in that approximate order (Figure 4). Predicted overflow depths were generally between about 0.5-1.0 ft, with wetted widths of 250-350 ft and flow velocities around 0.6-2.5 ft/s.
The water surface elevation slope on the north side of the floodplain was predicted to be mild in general, and this area appears to act effectively as storage during the 100-year flood. The water depth is generally between 1 to 3 feet over the floodplain. The water on the floodplain returns to the same general region of the main channel that it came from as the hydrograph subsides.

The flow characteristics on the floodplain south of Stillaguamish River are more complex. The water surface elevation slope is generally similar to the channel slope, although the velocity in the main channel is much greater. On the floodplain, the model predicted greater depths and faster velocities through the swale areas than on surrounding higher ground. The water depth was predicted to vary considerably, up to 13 feet deep in the swales. In general, the south side of the floodplain is well vegetated and the predicted velocity was less than the permissible velocity of 5 ft/s for grassy land, and predicted shear stresses were low. Therefore erosion or avulsion was not considered an immediate risk over vegetated surfaces. Shear stresses were high enough to be associated with higher erosion risk over bare and freshly seeded surfaces.
The predicted velocities over the floodplain are generally insufficient to initiate head-cutting in the swales. The field survey did not include walking the floodplain drainage network to identify the potential for head-cutting, but anecdotal accounts of extreme flood events by locals interviewed indicate that swale segments 1 and 3 do not experience erosion as the floodplain drains, except locally when a culvert blows out. If any head-cutting occurred, it would most likely be associated with a runoff event in the tributary draining to swale segments 2 and 1 when the mainstem is not flooding.

_**Modeling Caveats**_

It should be noted that there are several sources of uncertainty in the modeling that may influence avulsion risk ratings. One uncertainty stems from steady-state modeling of the 100-year flood level. Under steady-state modeling, the floodplain side of the dike was filled with water and the head difference across the dike had reached a numerical equilibrium. It is more likely during unsteady flow that the head difference across the dike would be greater than as modeled under steady flow, because it takes some time for the floodplain to fill while the flood hydrograph is rising. The steady-state approach allows the level on the floodplain side of the dike to rise higher than it might actually reach. Hence, the water depth may actually be shallower on the floodplain such that erosive energy of the overtopping flow may be greater than modeled. However, relative differences in predicted water surface elevations of the dike overflow and over the floodplain should be comparable between steady and unsteady flow simulations, such that the risk rankings of different dike locations would not be expected to be re-ordered dramatically depending on the simulation.

Another uncertainty stems from the mesh scale resolution and survey data accuracy. Using relatively coarse mesh elements may have eliminated smaller topographic low points in the dike road prism crest that would in actuality have some water flowing over them during the 100-year event. This uncertainty could be reduced by creating a finer mesh, especially in the transition zone from the main channel to the floodplain. The current transition elements in the model may not completely capture the variation of the velocity and water surface elevation change since slight instability was observed in this transition zone during the modeling. The reduction of element sizes should remove the instability and improve the transition and result in more accurate hydraulic predictions. Such modifications would require additional effort, however. The results based on the current mesh should nonetheless be suitable for purposes of a screening level analysis.

Finally, it should be noted that the modeling results reported here are for one set of boundary and initial conditions. Depending on flood duration, for example, the water surface elevation may vary at the downstream end of the model such that the solution results could change. There is some uncertainty in the predicted water surface elevations associated with the particular
boundary and initial conditions modeled, and some of the dike locations that were not predicted to top may under other circumstances. The results presented for the existing condition reflects the boundary conditions obtained from the previous HEC-RAS model. With different (i.e., higher) water surface elevations for boundary and initial conditions, the model could predict overtopping at more locations on the Dike Road dike. Additional analyses would be needed to determine if the conclusions of this analysis would change, if at all, based on other boundary conditions. Again, however, the relative differences in dike and predicted water surface elevations resulting from the present modeling should still be indicative of relative levels of risk of dike overtopping.

**Channel Migration**

Overlays of all five years of aerial photography indicates some segments of the Stillaguamish River have been active geomorphically, while other segments have been relatively quiescent since the Dike Road dike was constructed. Three primary areas of deposition were identified that may influence County actions and avulsion risk: between roughly RM 15.2-15.5, RM 17.0-17.8, and above the confluence of the north and south fork Stillaguamish rivers (Figure 5). Modification of deposition patterns in these segments could influence flooding and avulsion risk. For example, the Hallar Trestle piers and riprap appear to restrict flood flows sufficiently that deposition is favored at the confluence and upstream. The piers and riprap effectively meter the amount of coarse and fine sediments moving downstream. This deposition has resulted in extensive channel movement, particularly in the North Fork. If the piers and riprap were to be removed, it is probable that sediment transport rates to below the trestle location would increase considerably, leading potentially to increased deposition above RM 17.0. This could lead to an increase in flood water surface elevation and increased overtopping of the Johnson, Schloman, and upper portion of the Dike Road dikes.

Similarly, the constriction at approximately RM 15.2 has influenced deposition patterns immediately upstream, resulting in the channel moving about actively. Longer term aggradation between RM 15.2-15.5 could cause backwater effects upstream that promote overtopping of the Dike Road dike, particularly in the vicinity of the concrete plant and the junction with 59th Ave NE (dike locations 6 and 5, respectively in Figure 2). Moreover, aggradation near RM 15.2-15.5 could promote aggradation upstream of RM 17.0.
Greatest changes in channel planform in the vicinity of the Dike Road dike were evident in the comparison of the 1933 and 1955 channel locations (Figure 6). The exposed area of point bar on the right bank of the Dike Road bend shrank and filled in with sediment and vegetation. This may reflect the combined effects of gravel mining and dike construction in the reach. In addition, the location of the left bank just upstream of the concrete plant location changed between the 1933 and 1955 photographs, and then remained in the same location thereafter. This indicates the effect of bank hardening before the 1955 photographs were taken. There has been relatively little change since 1955 along the Dike Road dike as indicated by the subsequent photograph comparisons (Figures 7-9). The right bank scallop below the Schloman Road dike apparently filled in and became vegetated some time between 1955 and 1974 (Figure 7). The right bank river edge has not moved much at that location since 1974, suggesting that avulsion at that location may be unlikely in the immediate future because of the presence of the Schloman Road dike.
Figure 6. Comparison of river margin and gravel bar edge locations between 1933 and 1955 in the Stillaguamish River project area. Transect locations where cross-section profiles were compared are indicated also.

Figure 7. Comparison of river margin and gravel bar edge locations between 1955 and 1974 in the Stillaguamish River project area. Transect locations where cross-section profiles were compared are indicated also.
Figure 8. Comparison of river margin and gravel bar edge locations between 1974 and 1991 in the Stillaguamish River project area. Transect locations where cross-section profiles were compared are indicated also.

Figure 9. Comparison of river margin and gravel bar edge locations between 1991 and 2003 in the Stillaguamish River project area. Transect locations where cross-section profiles were compared are indicated also.
Cross-Section Data

Cross-section profiles were generally consistent between the Corps’ survey conducted in the mid-1970s, the County 1997 survey, and the LIDAR data (Figures 10-12). Differences in floodplain elevation typically could be explained by the presence of vegetation: the LIDAR elevations were generally higher in areas where vegetation was dense (e.g., right bank of transect ST9; Figure 10). The greatest difference in channel elevations was noted for ST 11, but this could potentially reflect the 1997 data having been collected a short distance upstream of the Corps transect where the channel widens (Figure 11). The 1997 data generally recorded lower bed elevations in the main channel and bars than the 1970s and the recent LIDAR data. This may have reflected a net loss during the 1990 flood and gradual gain of coarse bed material thereafter. Collins (1992) calculated a net loss in gravel storage in the project reach between 1972 and 1991, especially in the vicinity of the left bank point bar at the concrete plant location. The changes may also have reflected gravel mining, which was occurring in the early 1990s but stopped circa mid 1990s.
Figure 10. Changes in cross-sections ST9 and ST10 profiles, Stillaguamish River (locations depicted in Figures 6-9).
STILLAGUAMISH RIVER CROSS-SECTION COMPARISON
SECTION ST11

STILLAGUAMISH RIVER CROSS-SECTION COMPARISON
SECTION ST12

Figure 11. Changes in cross-sections ST11 and ST12 profiles, Stillaguamish River (locations depicted in Figures 6-9).
Figure 12. Changes in cross-section ST13 profile, Stillaguamish River (location depicted in Figures 6-9).
SYNTHESIS AND CONCLUSIONS

Tables 1 and 2 summarize the results of the analysis in terms of the various factors influencing avulsion risk. Table 1 presents risk ratings for each floodplain swale segment depicted in Figure 2; Table 2 similarly presents risk ratings at each of the six dike locations depicted in Figure 2. Each factor is classified according to low, medium, or high avulsion risk across sites. However, a given level of risk (e.g., “High”) does not equate in the tables to a similar degree of risk across factors. For example, a high ranking for the “head difference across dike” factor does not have the same weight as a high ranking for the “floodplain topography near dike” factor in Table 2. The overall ranking of avulsion risk for a given dike location or swale segment will depend on how each factor is weighted, which in turn reflects some subjectivity. In addition, it is important to note that a “High” ranking does not necessarily imply that avulsion will happen in the near term, for example in the next 10-20 years. All measures should be considered strictly relative.

Table 1. Relative ratings of avulsion risk in swale segments identified in Figure 2, Dike Road dike analysis, Stillaguamish River near Arlington. Note that rating weights may vary across factors.

<table>
<thead>
<tr>
<th>Avulsion Risk Factor</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
<th>12</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal Slope</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>M</td>
<td>M</td>
<td>L</td>
<td>H</td>
<td>M</td>
<td>H</td>
<td>H</td>
<td>M</td>
</tr>
<tr>
<td>Hard Grade Control</td>
<td>H</td>
<td>H</td>
<td>H</td>
<td>H</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>H</td>
<td>M</td>
<td>H</td>
<td>H</td>
</tr>
<tr>
<td>Downstream</td>
<td>H</td>
<td>H</td>
<td>H</td>
<td>H</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>H</td>
<td>M</td>
<td>H</td>
<td>H</td>
</tr>
<tr>
<td>Velocity at 100-yr Flood Level: Vegetated Floodplain</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
</tr>
<tr>
<td>Velocity at 10-yr Flood Level: Vegetated Floodplain</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
<td>L</td>
</tr>
<tr>
<td>Shear Stress at 100-yr Flood Level: Bare/Freshly Seeded Floodplain</td>
<td>H</td>
<td>L</td>
<td>H</td>
<td>H</td>
<td>H</td>
<td>M</td>
<td>H</td>
<td>H</td>
<td>H</td>
<td>H</td>
<td>H</td>
<td>H</td>
</tr>
<tr>
<td>Shear Stress at 10-yr Flood Level: Bare/Freshly Seeded Floodplain</td>
<td>H</td>
<td>L</td>
<td>H</td>
<td>H</td>
<td>H</td>
<td>M</td>
<td>H</td>
<td>H</td>
<td>H</td>
<td>H</td>
<td>H</td>
<td>H</td>
</tr>
</tbody>
</table>

Key to Velocity and Shear Stress Ratings (evaluated at 100-yr flood level)

Velocity:  
L: \( V < 5 \text{ ft/s} \)

Shear Stress:  
L: \( < 0.025 \text{ lb/ft}^2 \)
M: \( 0.025 \leq \text{Shear Stress} < 0.05 \text{ lb/ft}^2 \)
H: \( \geq 0.050 \text{ lb/ft}^2 \)

(Shear stress = 62.4 \times \text{approximate depth (ft)} \times \text{swale width (ft)} \times \text{swale surface slope})
Table 2. Relative ratings of avulsion risk associated with six hypothetical dike breach locations identified in Figure 2, Dike Road dike analysis, Stillaguamish River near Arlington. Note that rating weights may vary across factors.

<table>
<thead>
<tr>
<th>Avulsion Risk Factor</th>
<th>Breach/Local Avulsion Risk, by Dike Road/Johnson Dike Location Number</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td>Flow Depth Overtopping Dike at 100-yr Flood Level</td>
<td>H</td>
</tr>
<tr>
<td>Flow Depth Overtopping Dike at 10-yr Flood Level</td>
<td>M</td>
</tr>
<tr>
<td>Flow Velocity Overtopping Dike at 100-yr Flood Level</td>
<td>M</td>
</tr>
<tr>
<td>Flow Velocity Overtopping Dike at 10-yr Flood Level</td>
<td>L</td>
</tr>
<tr>
<td>Floodplain Topography Near Dike</td>
<td>L</td>
</tr>
<tr>
<td>Head Difference Across Dike at 100-yr Flood Level (ft)</td>
<td>1.8</td>
</tr>
<tr>
<td>Future In-River Aggradation Leading to Overtopping</td>
<td>H</td>
</tr>
<tr>
<td>Slope of Lee Side of Dike</td>
<td>L</td>
</tr>
<tr>
<td>Apparent Integrity of Dike on River Side</td>
<td>L</td>
</tr>
<tr>
<td>Downstream Flow Path Swale Segments (see Figure 2)</td>
<td>1-3-4-5-7</td>
</tr>
<tr>
<td>Length of Flow Path Downstream Through Swales</td>
<td>L</td>
</tr>
<tr>
<td>Downstream Flow Path Avulsion Risk (cf. Table 1)</td>
<td>L</td>
</tr>
</tbody>
</table>

**Key to Velocity and Depth Ratings**

- **Velocity:**
  - L: < 0.5 ft/s
  - M: 0.5 ≤ Velocity < 1.0 ft/s
  - H: ≥ 1.0 ft/s

- **Depth:**
  - L: Dry
  - M: < 0.5 ft
  - H: ≥ 0.5 ft

All flow paths below the six dike locations depicted in Figure 2 have at least part of their length associated with a high risk of erosion when the floodplain surface is bare or has newly sprouting vegetation (e.g., for approximately 1 month after tilling and seeding; Table 1). Anecdotal accounts made by local farm workers indicate that extensive floodplain erosion has occurred in previous large floods only in areas with this surface condition. There appears to be little risk of
erosion and gully formation when the floodplain vegetation is well established, based on both the 2D modeling of floodplain velocities and anecdotal information. Hence, of the risk factors listed in Table 1, the longitudinal slope and presence/absence of a hard grade control along a portion of the route should receive higher weighting for determining relative avulsion risk than the other factors. Presence of a hard grade control is an important factor reducing risk. For example, the 59th Ave NE prism and surface should act as a hard point reducing avulsion risk in upstream swale segments 5, 6, 7, and 8. The resulting relative risk rating for each complete flow path is given in the last row of Table 2, and reflects higher weighting of these two risk factors.

Table 3 presents suggested relative rankings of avulsion risk, from highest (rank = 1) to lowest (rank = 6) for each dike location depicted in Figure 2, based on consideration of all factors and their respective risk ratings in Table 2. Highest risk, and therefore priority for implementing protective actions, is associated with dike locations 5 and 4 and their floodplain flow paths.

<table>
<thead>
<tr>
<th>Dike Location No.</th>
<th>Avulsion Rank</th>
<th>Comments</th>
<th>Avulsion Risk Associated Mostly With Potential For:</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6</td>
<td>Smallest risk of downstream swale erosion; most problems expected in vicinity of dike</td>
<td>Dike breach</td>
</tr>
<tr>
<td>2</td>
<td>5</td>
<td>Similar to Location 1, but greater head differential across dike during 100-yr event, most problems expected in vicinity of dike</td>
<td>Dike breach</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>Deep flow and fast velocity over road during 100-yr event, intermediate head difference across dike, steep swale segment near potential breach location, most problems expected in vicinity of dike</td>
<td>Dike breach</td>
</tr>
<tr>
<td>4</td>
<td>2</td>
<td>Similar to Location 5, but mitigated slightly by 59th Ave NE grade control, however, some evidence seen of dike toe erosion on river side</td>
<td>Dike breach leading to swale erosion</td>
</tr>
<tr>
<td>5</td>
<td>1</td>
<td>High head differential across dike during 100-yr event, no grade control downstream, steep swale segment near potential breach location</td>
<td>Dike breach leading to swale erosion</td>
</tr>
<tr>
<td>6</td>
<td>4</td>
<td>Steep swale segment near potential breach location, shortest distance for floodplain erosion, potentially mitigated by heavy bank protection on dike near concrete plant</td>
<td>Swale erosion leading to dike breach</td>
</tr>
</tbody>
</table>
POSSIBLE PROTECTIVE ACTIONS

The following proactive actions would reduce the risk of avulsion during an extreme flood event:

• Work with landowners to avoid tilling and seeding in the vicinity of each swale segment identified in Figure 2, during the flood season and one month prior to ensure that adequate vegetative cover is maintained to prevent erosion. Special emphasis should be placed on not disturbing groundcover along segments 1, 3, 4, 9, and 11. Tilling should be minimized elsewhere to the extent possible during that period.

• Protect the floodplain side of the Dike Road prism at dike locations 2, 3, 4, and 5 in Figure 2, using rock or other suitable erosion resistant structures. If possible, the side slope should be reduced. The toe should be protected with an erosion-resistant rock blanket to prevent scour and undermining of the protective surface.

• Protect the downstream side of the 59th Ave NE road prism at the junction of swale segments 4, 5, and 8 in Figure 2, using rock or other suitable erosion resistant structures. The toe should be protected with an erosion-resistant rock blanket to prevent scour and undermining of the protective surface.

• Monitor the river-side toe of the Dike Road dike at dike location 4 in Figure 2 after each major flood to determine if additional erosion has occurred, and formulate corrective measures as necessary.

• If the Johnson levee is removed in part or whole, the downgradient side of the Dike Road prism should be protected at its low point near the junction with SR 530 using rock or other suitable erosion resistant structures.
APPENDIX A

Two-Dimensional Hydraulic Model Simulation Results:
Dike Road Dike Avulsion Risk Assessment, Stillaguamish River

This Appendix summarizes the methods and results of 2-dimensional (2-D) hydraulic modeling performed to assess the risk of river channel avulsion in the vicinity of Dike Road dike, which is located along the Stillaguamish River. The simulation reach extends upstream from just below the SR 9 Bridge near Arlington, WA to about 4.5 river miles downstream. Plate 1a depicts the project simulation area. Just below the SR 9 Bridge, the river is confined by the Schloman Road dike on the north, and the Johnson levee and Dike Road dike on the south. The downstream boundary of the simulation area is at the location where the stream channel is nearest to SR 530. The floodplain simulation area is bounded by high terrain on the north side and by SR 530 on the south. The floodplain vegetation is mainly agricultural (Plate 2), and only a small portion along the right side (looking downstream) of the stream channel is forested.

Historically, the Stillaguamish River meandered extensively over its floodplain, as evidenced by remnant swales on the north and south sides of the floodplain. One of the purposes of the 2-D modeling is to understand the flow behavior in these swales which might be associated with channel avulsion and migration in the future.

I. MODEL DEVELOPMENT

Plate 1b shows river stations defined by Snohomish County Surface Water Management (SWM). The model reach extends about 4.5 river miles, from upstream at River Station 600 ft (RS 600, just below SR 9 Bridge) to downstream at RS 24,600 ft. The model was developed using SMS 8.0, which integrates RMA2 with a user-friendly interface for data input/output operations. RMA2 is a two-dimensional depth-averaged finite-element model supported by the U.S. Army Corps of Engineers (USACE).

The data required to develop the 2-D hydraulic model included topographic, hydraulic, hydrologic, ground vegetation, and bed substrate data. The topographic data were used for the finite-element mesh generation and elevation interpolation. The hydraulics and hydrology data were used to establish boundary conditions in the model calibration and simulation. Vegetation type and bed substrate materials were used for estimating channel roughness and the effects of flow turbulence.

I.1 Topographic Data

Four types of elevation data were used to determine ground elevations, including LIDAR, bathymetric, real time kinematic (RTK) GPS, and previous HEC-RAS model data. Each type of data was applied to specific portions of the simulation area to achieve the best possible accuracy for the model mesh geometry. The vertical and horizontal datums used in developing the model were NAVD88 and NAD83, respectively.
LIDAR Data: Bare earth model LIDAR data collected in March 2005 were received from SWM for the project area. The original LIDAR data file contained more than 5 million data points with approximately 6 ft spacing between points. The density of points was greater than needed because finite-element mesh nodes are much coarser, usually more than 100 feet apart on the floodplain. Thus such fine resolution (6 ft spacing) does not improve nodal elevation accuracy. Consequently, the LIDAR data set was reduced using a Fortran program by systematically selecting the first of every 15 points and dumping the next 14 points in the input file. Also, points with elevations greater than 70 ft were removed because the highest inundation level for 100-year flood was expected to be below that level. Points south of SR 530 and in the wetted channel were also eliminated. This reduced the total number of LIDAR data to around 136,600 points, which was more manageable. The LIDAR data were used to interpolate elevations for mesh elevations located on exposed gravel bars and floodplain areas.

Bathymetric Data: Channel bathymetry was surveyed in the Stillaguamish River by both SWM and R2 staff using GPS/ADCP/depth finder equipment. Surveys were conducted on a number of occasions in 2005, and data processed by SWM and provided to R2. There were a total of 6,519 bathymetric data points surveyed within the 4.5-mile stream reach. The channel bathymetric data were used to compute the elevations for the mesh points located in the wetted portion of the channel.

RTK Data: An RTK instrument operated by SWM staff was applied to survey dikes, levees, and the longitudinal water surface elevation along the Stillaguamish River. There were a total of 61 points surveyed for the Schloman Road dike, 62 points for the Johnson levee, and 111 points for the Dike Road dike. The RTK data were used exclusively to represent the nodal elevations on the dikes/levees. Longitudinal water surface elevations were measured four times in April and May of 2005, and were used to determine channel roughness (Manning’s n) for the model. The LIDAR and RTK data elevations were compared as a QA/QC measure. A total of 20 RTK points selected randomly from all three levees/dikes were compared to proximal LIDAR points. It was determined that differences in elevation varied. For the Dike Road dike and Johnson levee, the differences were between +/-0.3 ft, which was considered acceptable for modeling large floods. For Schloman Road, the differences were greater. However, such greater differences may be due to the irregular configuration of riprap surveyed. Therefore, in general, the elevation differences between RTK and LIDAR data was not considered to be significant.

HEC-RAS data: The bathymetric data were insufficient to determine all nodal elevations in the wetted portion of the channel. Therefore, cross-sections in a HEC-RAS model developed by the Snohomish County Surface Water Management (SWM) and Pacific International Engineering (PIE) were used together with the bathymetric data to obtain complete transect profiles near the upstream boundary.

I.2 Finite-Element Mesh Development

The simulation area boundary shown in Plate 1 generally reflects the inundated area under the 100-year flood. Although the floodplain south of SR 530 is also flooded during the 100-yr flood
event due to flow overtopping the road prism at selected locations, this area was not included in
the modeling. This is because the water overtopping SR 530 flows farther downstream, out of
the domain area, and thus could be considered lost in the model. Also, the water level for
smaller flood sizes, such as 10-year return interval event, does not overtop the road.

The mesh elements in the main channel of the Stillaguamish River required careful specification
for numerical stability. Predicted velocities in the main channel generally accelerated near the
Dike Road dike bend and the simulated flow direction changed rapidly. The predicted maximum
velocity was as much as 11.5 ft/s for the 100-year flood. Rapidly changing velocity gradients
and directions are typically associated with numerical instability and longer computation time in
2-D models. Hence, the elements in the main channel were reduced in size and oriented in the
directions of the streamlines to facilitate a stable numerical solution (Plate 3).

On the floodplain, the predicted velocities were slower and the change of flow direction more
gradual. Therefore, the elements were allowed to be larger than those in the main channel. Such
element size relaxation reduced the computational time significantly (computational time is
proportional to the square of the band width of the global finite-element matrix; Steffler, 2002).
Also, in order to better capture the flow behavior in the swales on the floodplain, the elements
near the swales were reduced in size and oriented in generally expected flow directions.

The mesh for the wetted channel was constructed initially using the LIDAR data. Each nodal
elevation in the wetted channel was then adjusted manually to reflect the channel bathymetry
data. The nodal elevations representing the dike and levees were also adjusted to reflect the
survey data. The resulting finite-element mesh was composed of 6-point and 8-point quadratic
elements, with a total of 1,880 elements and 5,455 nodal points, as shown in Plate 3.

II. HYDRAULICS AND HYDROLOGY

II.1 Hydraulics

A field survey was conducted on April 26, 2005 by R2 with the assistance of SWM staff to
measure velocities and flow depths at a transect in the vicinity of the Dike Road river bend (at
~RS 5300 ft in Plate 1b) during a moderate flow level. The measured discharge was 3,516 cfs.
The result was used to estimate a channel roughness Manning’s n equal to 0.0349. The
calculation is summarized in Table A-1.

The following equivalent roughness equation was employed to estimate Manning’s n values for
higher flows (USACE, 2003):

\[
n_{\text{estimated}} = \frac{1.486 R^{1/6}}{32.6(\log_{10} \frac{R}{k})},
\]

where R is the hydraulic radius, equal to flow conveyance area A divided by the wetted
perimeter P, k is the equivalent roughness height, which remains constant for all flows, and
n_{\text{estimated}} is the calculated Manning’s n value. The k value was calculated to be 0.095 ft using the
measured flow and the corresponding Manning’s n value. Using this k value and varying WSE to obtain R and \( n_{\text{estimated}} \), the discharge was calculated iteratively from Manning’s equation:

\[
Q = \frac{1.486}{n_{\text{estimated}}} R^{2/3} S^{1/2} A
\]

to arrive at the proper values of \( n_{\text{estimated}} \). The resulting \( n_{\text{estimated}} \) values for greater flows are listed in Table A-2; \( n_{\text{estimated}} = 0.0324 \) for 2-year flood, \( n_{\text{estimated}} = 0.0323 \) for 5-year flood, \( n_{\text{estimated}} = 0.0323 \) for 10-year flood, and \( n_{\text{estimated}} = 0.0322 \) for 100-year flood.

Longitudinal water surface elevation (WSE) profiles were surveyed four times in April and May 2005. Water surface elevation data were surveyed at the waters edge. The calculated longitudinal water surface elevation slope near the surveyed cross-section (RS 5,300) was 0.00119. Figure A-1 depicts the longitudinal water surface elevation surveys.

![Figure A-1. Longitudinal water surface elevation (WSE) profile of the Stillaguamish River (RS = 0 corresponds to the Hallar Trestle).](image-url)
Table A-1  Measured velocity distribution and transect profiles data, and calculated hydraulic parameters for cross-section measured at ~RS 5300 ft of Stillaguamish River.

<table>
<thead>
<tr>
<th>Station</th>
<th>WSE (ft)</th>
<th>Depth (ft)</th>
<th>V_{0.6/0.2} (ft/s)</th>
<th>V_{0.8} (ft/s)</th>
<th>V_{ave} (ft/s)</th>
<th>Angle (degrees)</th>
<th>Wetted area (ft^2)</th>
<th>Q (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WE</td>
<td>0</td>
<td>100.0</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>6.9</td>
<td>11.8</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>95.3</td>
<td>4.7</td>
<td>2.81</td>
<td>0.98</td>
<td>1.90</td>
<td>0.0</td>
<td>5.4</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>93.3</td>
<td>6.7</td>
<td>3.54</td>
<td>2.56</td>
<td>3.05</td>
<td>0.0</td>
<td>5.2</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>91.8</td>
<td>8.2</td>
<td>4.50</td>
<td>4.13</td>
<td>4.32</td>
<td>0.0</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>91.3</td>
<td>8.7</td>
<td>5.41</td>
<td>4.93</td>
<td>5.17</td>
<td>20.0</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>91.3</td>
<td>8.7</td>
<td>6.02</td>
<td>5.69</td>
<td>5.86</td>
<td>20.0</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>91.8</td>
<td>8.2</td>
<td>6.21</td>
<td>5.27</td>
<td>5.74</td>
<td>15.0</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>35</td>
<td>91.8</td>
<td>8.2</td>
<td>6.22</td>
<td>5.29</td>
<td>5.76</td>
<td>15.0</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>91.6</td>
<td>8.4</td>
<td>6.15</td>
<td>5.38</td>
<td>5.77</td>
<td>15.0</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>45</td>
<td>91.6</td>
<td>8.4</td>
<td>6.09</td>
<td>5.30</td>
<td>5.70</td>
<td>15.0</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>91.4</td>
<td>8.6</td>
<td>5.73</td>
<td>5.40</td>
<td>5.57</td>
<td>10.0</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>55</td>
<td>91.8</td>
<td>8.2</td>
<td>5.47</td>
<td>4.72</td>
<td>5.10</td>
<td>10.0</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>91.8</td>
<td>8.2</td>
<td>5.27</td>
<td>4.66</td>
<td>4.97</td>
<td>5.0</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>65</td>
<td>92.2</td>
<td>7.8</td>
<td>5.09</td>
<td>4.46</td>
<td>4.78</td>
<td>5.0</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>70</td>
<td>92.2</td>
<td>7.8</td>
<td>4.68</td>
<td>4.37</td>
<td>4.53</td>
<td>5.0</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>75</td>
<td>92.6</td>
<td>7.4</td>
<td>4.21</td>
<td>4.31</td>
<td>4.26</td>
<td>5.0</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>80</td>
<td>92.6</td>
<td>7.4</td>
<td>3.82</td>
<td>4.09</td>
<td>3.96</td>
<td>0.0</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>85</td>
<td>93.2</td>
<td>6.8</td>
<td>3.52</td>
<td>3.85</td>
<td>3.69</td>
<td>0.0</td>
<td>5.2</td>
</tr>
<tr>
<td></td>
<td>90</td>
<td>94.8</td>
<td>5.2</td>
<td>3.34</td>
<td>3.43</td>
<td>3.39</td>
<td>0.0</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>95</td>
<td>94.8</td>
<td>5.2</td>
<td>3.10</td>
<td>2.21</td>
<td>2.66</td>
<td>0.0</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>95.2</td>
<td>4.8</td>
<td>2.72</td>
<td>2.23</td>
<td>2.48</td>
<td>0.0</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>105</td>
<td>95.5</td>
<td>4.5</td>
<td>2.40</td>
<td>2.04</td>
<td>2.22</td>
<td>0.0</td>
<td>10.1</td>
</tr>
<tr>
<td></td>
<td>115</td>
<td>96.8</td>
<td>3.2</td>
<td>1.98</td>
<td>1.66</td>
<td>1.82</td>
<td>0.0</td>
<td>10.0</td>
</tr>
<tr>
<td></td>
<td>125</td>
<td>97.6</td>
<td>2.4</td>
<td>1.16</td>
<td>1.16</td>
<td>1.16</td>
<td>0.0</td>
<td>10.0</td>
</tr>
<tr>
<td></td>
<td>135</td>
<td>98.5</td>
<td>1.5</td>
<td>0.60</td>
<td>0.60</td>
<td>0.60</td>
<td>0.0</td>
<td>10.0</td>
</tr>
<tr>
<td></td>
<td>145</td>
<td>99.1</td>
<td>0.9</td>
<td>0.05</td>
<td>0.05</td>
<td>0.05</td>
<td>0.0</td>
<td>10.0</td>
</tr>
<tr>
<td></td>
<td>155</td>
<td>99.4</td>
<td>0.6</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0</td>
<td>10.0</td>
</tr>
<tr>
<td></td>
<td>165</td>
<td>99.5</td>
<td>0.5</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0</td>
<td>10.0</td>
</tr>
<tr>
<td></td>
<td>175</td>
<td>99.8</td>
<td>0.2</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0</td>
<td>10.0</td>
</tr>
<tr>
<td>WE</td>
<td>185</td>
<td>100.0</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Wetted Perimeter (P)= 188.1

Flow Area (ft^2)= 864.8

Q(cfs)= 3516.1

Calculated WSE Slope(cfs)= 0.00119

Equivalent Roughness k (ft)= 0.095

Estimated Manning's n= 0.0349
II.2 Flood Hydrology

Flood magnitudes were obtained for different return intervals from SWM (2000) and Collins and Turner (2003). Table A-2 lists the flood magnitudes for various return intervals, and the corresponding estimates of Manning’s n adjusted for relative roughness.

Table A-2. Estimated channel roughness and flood magnitudes for different return intervals, Stillaguamish River at the Dike Road dike.

<table>
<thead>
<tr>
<th>Return Interval (years)</th>
<th>Flow (cfs)</th>
<th>$n_{estimated}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>45300</td>
<td>0.0324</td>
</tr>
<tr>
<td>5</td>
<td>58000</td>
<td>0.0323</td>
</tr>
<tr>
<td>10</td>
<td>65400</td>
<td>0.0323</td>
</tr>
<tr>
<td>25</td>
<td>73100</td>
<td>0.0323</td>
</tr>
<tr>
<td>50</td>
<td>79800</td>
<td>0.0322</td>
</tr>
<tr>
<td>100</td>
<td>85400</td>
<td>0.0322</td>
</tr>
</tbody>
</table>

II.3 Hydraulic Boundary Conditions

The WSE at the downstream boundary was determined using the 1-dimensional HEC-RAS model developed by SWM and PIE. The HEC-RAS model was originally developed for simulating unsteady flow in the Stillaguamish River, from the Hallar Trestle to Possession Sound. A hydrograph of the 100-year event and its simulation results (resulting hydrograph) for existing channel geometry condition were used to determine the WSEs at the downstream and upstream boundaries for various flood flows used in our 2-D modeling (it should be noted that the predicted hydraulics would change with different initial and boundary conditions specified).

The WSEs and flow magnitudes for various flood events from the HEC-RAS model are summarized in Table A-3. The 100-year flood discharge at the downstream boundary was 11,800 cfs less than the inflow at the upstream boundary due to the flow overtopping the SR 530. According to conversations with SWM staff, there is no overtopping occurring at 10-year flood event. The 25-year and 50-year flood events were not modeled.

Table A-3. Flow magnitudes and WSEs at the boundaries of 2-D simulation area.

<table>
<thead>
<tr>
<th>Flood frequency</th>
<th>Flow (cfs)</th>
<th>WSE (NAVD88, ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>u/s boundary</td>
<td>d/s boundary</td>
</tr>
<tr>
<td>2-year</td>
<td>45300</td>
<td>45300</td>
</tr>
<tr>
<td>5-year</td>
<td>58000</td>
<td>58000</td>
</tr>
<tr>
<td>10-year</td>
<td>65400</td>
<td>65400</td>
</tr>
<tr>
<td>25-year</td>
<td>73100</td>
<td>-</td>
</tr>
<tr>
<td>50-year</td>
<td>79800</td>
<td>-</td>
</tr>
<tr>
<td>100-year</td>
<td>85400</td>
<td>73600</td>
</tr>
</tbody>
</table>
III. CHANNEL ROUGHNESS AND BED MATERIAL ASSIGNMENT

The project simulation area was divided into three zones to characterize the ground roughness for different types of land cover. The three zones include the channel of the mainstem Stillaguamish River, the floodplain/pasture, and forest. Plate 4 shows the areal distribution of each land cover designation. Each zone was assigned different Manning’s $n$ and eddy viscosity values to reflect the corresponding resistance to water movement and the mixing scale of the flow, respectively, of the particular type of land cover. Table A-4 summarizes the Manning’s $n$ and eddy viscosity values.

<table>
<thead>
<tr>
<th>Land Cover Type</th>
<th>Manning’s $n$</th>
<th>Eddy Viscosity (lb-sec/ft$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mainstem Channel</td>
<td>0.0320</td>
<td>50</td>
</tr>
<tr>
<td>Forest</td>
<td>0.1000</td>
<td>50</td>
</tr>
<tr>
<td>Floodplain/Pasture</td>
<td>0.0650</td>
<td>50</td>
</tr>
</tbody>
</table>

IV. MODEL SIMULATION

**Existing Condition:** The model was first run for existing conditions and the 100-year flood event. The flow magnitude at the upstream boundary was 85,400 cfs and the WSE boundary condition at the downstream was 55.40 ft, as shown in Table A-3. The outflow at the downstream boundary was 73,600 cfs and thus there was a simulated 11,800 cfs loss associated with overtopping of SR 530. According to the HEC-RAS model, the WSE for the 100-year flood at the upstream boundary was 70.33 ft. The results of the initial model run indicated the water surface elevation (WSE) at the upstream boundary varied between 69.85-70.82 ft. The transect WSE average was 70.35 ft, which corresponded to a prediction error of 0.02 ft.

The simulation results are depicted in Plates 5-8. Simulated water depth is shown in Plate 5, in which the area not colored is dry land. Most of the Dike Road dike pavement was predicted to be above water and only a few areas were predicted to be inundated. Plate 6 is the velocity vector plot, and Plate 7 is the contour for the velocity magnitude. From the velocity vector plot, it is seen that predicted velocities in the swales were greater than over higher ground. The difference was generally about 1 ft/s or less, however. Plate 8 shows the predicted WSE contour (it should be noted that RMA2 treats the flow as groundwater if the WSE goes below the ground elevation, and thus allowing WSE to be continuous even the land becomes “dry.” mass conservation is checked after each simulation is finished to ensure the continuity law is not violated during the calculation).

After the model was set up to model the 100-year event, it was used to simulate smaller flood events with the same channel roughness. Plates 9, 10, and 11 show the simulated water depths for the 10-year, 5-year and 2-year flood events, respectively. The flow magnitudes and downstream WSE boundary conditions are listed in Table A-3.
**Dike Breaching Simulations:** Two hypothesized dike breaching alternatives were selected to reflect recent road overtopping. The first alternative involved a breach in the Dike Road dike just north of the Dike Road/59th Ave NE junction, and the second alternative a breach in the Dike Road dike near its upstream end, both of which are shown in Plate 12. The breaching width for each alternative was specified as 150 ft and the bottom of the breach was assumed equal to the floodplain level at the toe of the dike embankment. For Alternative 1, the invert elevation of the breach was set at 59.4 ft, and for Alternative 2 at 62.3 ft.

**Simulation Results for Alternative 1:** Plate 13 shows the mesh in the vicinity of the hypothesized Alternative 1 breaching. The mesh elements are oriented in the direction of streamlines of the flow coming out of the breach (such orientation can increase the numerical stability near the breaching where transition is drastic by maximizing the diagonal values of the local element matrix, and at the same time lower the number of elements to reduce the computational time). For the 100-year flood event simulation, the model predicted a total discharge through the breaching opening of about 4,000 cfs. The predicted water surface elevation at the opening was 64.65 ft and the maximum predicted mean column velocity was 6.8 ft/s. Plate 14 shows the water surface elevation contour and Plate 15 the corresponding velocity vector plot. Comparing Plates 8 and 6 (without breaching) to Plates 14 and 15 (with breaching) visually, we find only subtle differences occurring between these plots. This may be because the 4,000 cfs is less than 5% of the total inflow (85,400 cfs). Increased avulsion potential would thus be expected to occur only in the vicinity of the breach, but not downstream in response to the breach.

**Simulation Results for Alternative 2:** Plate 16 shows the mesh near the breaching for Alternative 2. The mesh elements are constructed similar to the elements for Alternative 1 that follows the streamlines of the flow coming out of the breach. For the simulation of the 100-year flood event, the model predicted a total discharge flowing through the breaching opening of about 2,400 cfs. The predicted water surface elevation at the opening was 67.65 ft and the predicted maximum velocity was about 5.8 ft/s. Plate 17 shows the water surface elevation contour and Plate 18 the corresponding velocity vector plot. Comparing Plates 8 and 6 (without breaching) to Plates 17 and 18 (with breaching), the differences in floodplain hydraulics again appear minor. The 2,400 cfs through the opening is a negligible amount (2.8%) of the total inflow flow (85,400 cfs). Again, increased avulsion potential would thus be expected to occur only in the vicinity of the breach, but not downstream in response to the breach.

**References:**


Plate 1a. Stillaguamish River flood simulation area.

Plate 1b. River Stations in Stillaguamish River.
Plate 2. View of floodplain vegetation south of the Dike Road dike.

Plate 3. Finite-element mesh developed to model existing conditions (without breaching).
Plate 4. Land-cover types assigned in 2-D simulation domain.

Plate 5. Simulated water depth contours for the 100-year flood event.
Plate 6. Simulated velocity vectors for the 100-year flood event.

Plate 7. Simulated velocity contours for the 100-year flood event.
Plate 8. Simulated water surface elevation contours for the 100-year flood event.

Plate 9. Simulated water depth contours for the 10-year flood event.
Plate 10. Simulated water depth contours for the 5-year flood event.

Plate 11. Simulated water depth contours for the 2-year flood event.
Plate 12. Two potential high-risk areas selected for simulated dike breaches, superimposed over simulated water surface elevation contours.

Plate 13. Finite-element mesh near the hypothesized breaching location 1.
Plate 14. Simulated water surface elevation contours for hypothesized breaching location 1, 100-year flood event.

Plate 15. Simulated velocity vectors for hypothesized breaching location 1, 100-year flood event.
Plate 16. Finite-element mesh near the hypothesized breaching location 2.

Plate 17. Simulated water surface elevation contours for hypothesized breaching location 2, 100-year flood event.
Plate 18. Simulated velocity vector for hypothesized breaching location 2, 100-year flood event.

Plate 19. Simulated velocity and shear stress distribution contours near the hypothesized breach location 1 for 100-year flood event.
Plate 20. Simulated velocity and shear stress distribution contours near the hypothesized breach location 2 for 100-year flood event.