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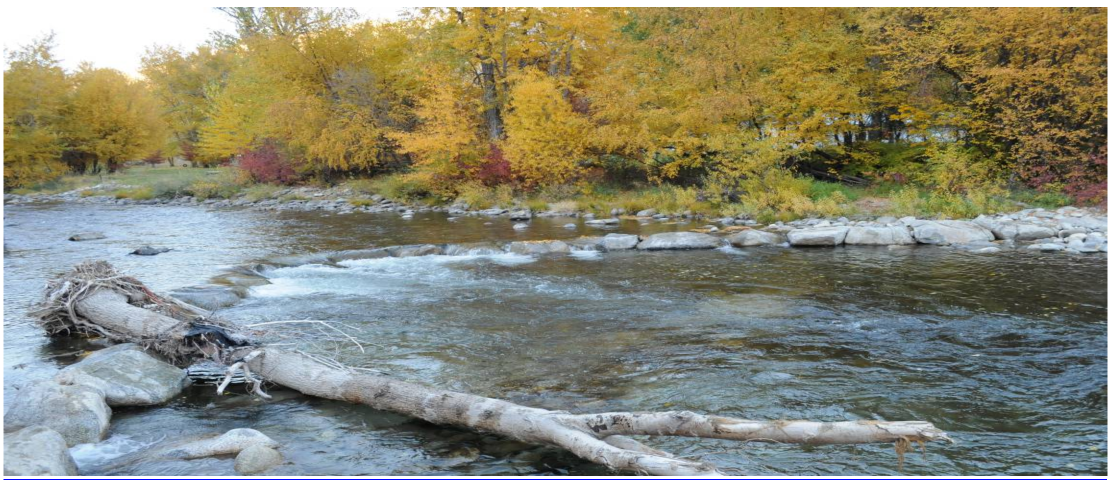
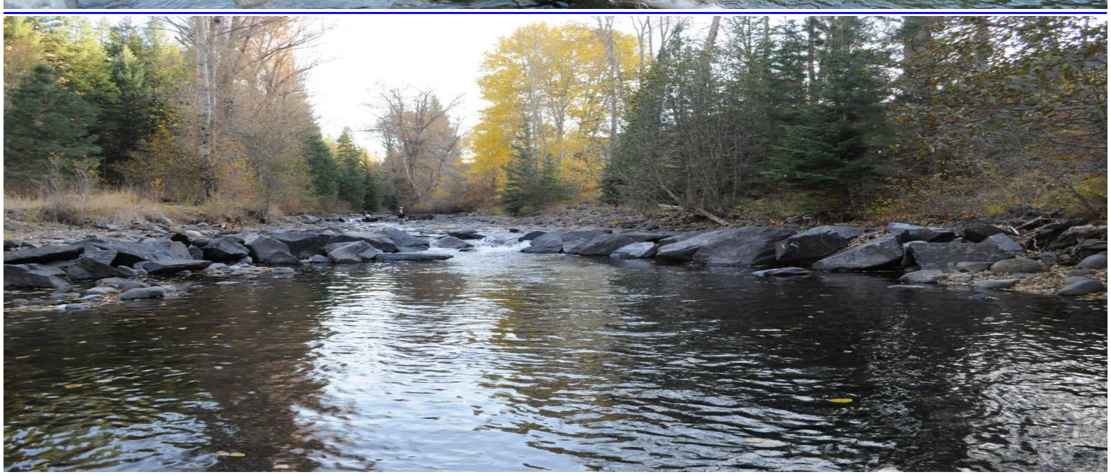
SRH-2009-46

Quantitative Investigation of the Field Performance of Rock Weirs





U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
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Mission Statements

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The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

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Technical Service Center, Denver, Colorado
Sedimentation and River Hydraulics Group, 86-68240

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Executive Summary

This report is the second in a series relating to field analyses of rock weir performance in support of a broader research effort on river spanning rock structures. The first report (Mooney et al, 2007) included a brief literature review of existing rock weir guidelines and documented qualitative evaluations of rock weir field performance, including identification of common failure mechanisms. Of the 127 structures evaluated, over 70 percent were determined to have at least partially failed based on their definition of failure. The most common failure mechanism was the growth of the scour pool and subsequent slumping of the footer rocks. The purpose of this report is to utilize field measurements and topographic surveys collected during field investigations to more quantitatively capture ranges in specific design parameters and to link the measured parameters to possible failure mechanisms. Results from the quantitative evaluation of field performance will inform other aspects of the research effort and assist engineers, planners, and managers in improving rock weir designs through increased focus on critical design parameters.

Of the 127 structures evaluated, topographic surveys were performed on 76 river-spanning loose rock structures between June 2005, and October 2008. Sixty-nine of these structures were included in the quantitative analysis. Structures surveyed include A-weirs, U-weirs, Asymmetrical U-weirs, W-weirs, and VW-weirs. Structure site characteristics and design variables (lengths and angles) were measured, and a comparison of parameters by degree of failure for each structure type was performed. Several discernible relationships were identified between structure parameters and degree of failure. The most notable include the relationships with recurrence interval of high flows, throat width, planform angles, and scour offset from structure. In addition, structure spacing, planform location and scour depth are important variables that relate to structure performance, but clear relationships were difficult to discern in this investigation.

The recurrence interval of the highest flows that each structure has been subject to since construction is a good indicator of the degree of structure failure. Based on the hydrologic analysis, structures having experienced fairly high flows (greater than 3-year discharge) had a high likelihood of mobilization of constituent rocks of the structure.

In general, structures with wider throats relative to the channel width failed less frequently than those with narrower or no throats (V-weirs). This relationship was identified for all types of structures. From a physical processes standpoint, this relationship is likely due to less flow constriction to the center of the channel.

Investigation of the planform angles suggests that the greater the open angle of the structure (more perpendicular the structure arms are to flow), the more likely the structure is to experience some degree of failure.

Based on the observations of this study, the closer the maximum scour depth is to the structure crest, the more likely the structure is to have experienced failure. The maximum scour for many of the structures visited was measured along the structure arms,

where the head drop over the structure is greatest, the footer protection is least, and high velocities have the greatest potential to scour. In most of the designs, as the crest elevation increases from the throat of the structure to the top of bank, the footer elevations also increase. Therefore, the depth of the footer rocks below the bed is least where the greatest head loss over the structure occurs.

Structure spacing is an important parameter to consider for structure design. Structures that were more closely spaced tended to have greater success rates than those that were spaced farther apart. However, structures spaced too closely may result in increased potential for failure. Multiple structures in sequence tended to outperform individual structures possibly because they offer a more stepped approach to energy dissipation. Asymmetrical U-weirs on the three forks of the Little Snake River failed more frequently as the structure spacing decreased, which may result from the development of a scour pool hindering the stability of the subsequent downstream structure.

Despite current knowledge of the importance of scour depth in predicting failure, this study was unable to capture a strong relationship between scour depth and failure. This is likely due to the inability to accurately capture the scour depth that caused the failure and the inability to obtain the foundation depth of the structure. These results support the need for a foundation of sufficient depth around the entire structure to prevent undermining of the footer rocks.

Structure planform location relative to meander bends is another key variable to consider in design. All structures failing as a result of general bank migration/flanking were located on bends. This result supports the need for a preliminary analysis of historical channel position prior to determining the best location for the structure and also points toward the importance of structure tie-ins to the bank.

The parameters included in this analysis did not address system wide processes that would need to be evaluated on a larger scale to evaluate potential impacts to structure stability**.** Lateral channel migration, sediment transport throughout the system, changes in slope through the system, and root strength present along the channel banks, among many other system-wide processes, may all play a role in how well a structure performs in a given location. One substantial finding of this analysis is that an understanding of the fluvial geomorphic processes must be gained prior to installing these structures to best recognize their potential for success in reaching a specific project objective and to realize the level of maintenance required at each site.

The quantitative analysis performed in this study must be coupled with the results from the laboratory and numerical modeling components to best understand the relationships between geometric variables of the structure and structure stability. The relationships identified through the field investigation help inform inputs to the numerical and physical modeling and can be used to assess ranges of specific parameters that drive structure stability. Furthermore, the information from the field investigations can be used to determine how well relationships developed from the physical and numerical modeling translate to the field. As a first step in synthesizing field findings with other aspects of the research, scour relationships developed through the physical model were applied to the field data.

1. Project Background

River spanning loose-rock structures are used in channels for a variety of purposes ranging from grade control to habitat complexity. The most common objectives in implementing these structures in rivers are to provide sufficient head for irrigation diversion without creating migration barriers for fish, to increase bed and bank stability, and to improve habitat features for endangered fish species. Due to the wide range in their application, designs of the structures and determinations of structure functionality are highly variable. However, common performance objectives of most river spanning rock structures include the ability to withstand high flow events and preserve their intended functions over a range of flow conditions.

Despite the use of in-channel rock weirs for a myriad of purposes over the last 50 to 100 years, a literature review of available information on river spanning rock structures identified a paucity of widely applicable guidelines for the design of the structures (Mooney et al, 2007). The review found that methods and standards for designing rock weirs based upon predictable engineering and hydraulic performance criteria currently do not exist. As such, Reclamation initiated a multi-faceted research effort to develop design criteria for river spanning rock structures. Ongoing research consists of three primary components:

1. Field investigations of river spanning rock structure performance,
2. Physical modeling in a laboratory setting, and
3. Numerical modeling of hydraulics resulting from the presence of river spanning rock structures.

Integration of field, laboratory, and numerical data sets will provide a scientific basis for predicting structure performance under various river conditions and for developing the most-effective design criteria.

This report is the second in a series relating to field analyses of rock weir performance. The first report (Mooney et al, 2007) included a brief literature review of existing rock weir guidelines and documented qualitative evaluations of rock weir field performance, including identification of common failure mechanisms. Within the report, field performance was determined by each structure's ability to maintain upstream water surface elevation and/or downstream pool depths, and degrees of failure were defined by departures from original designs and shifting of rocks within the structure. Of the 127 structures evaluated, over 70 percent were determined to have at least partially failed based on that definition of failure. The most common failure mechanism was the growth of the scour pool and subsequent slumping of the footer rocks. Field observations suggest that most structures were comprised of rocks adequately sized to prevent failure by sliding and rolling through incipient motion for flows they have experienced to date. The qualitative investigations also identified a need for a greater understanding of system processes (e.g., sediment loads and channel migration rates) and recognized structure tie-in to the bank and depth of foundation as critical components of structure designs.

The purpose of this report is to utilize field measurements and topographic surveys collected during field investigations to quantitatively capture ranges in specific design parameters and to link the measured parameters to possible failure mechanisms. River spanning rock structures evaluated for this report include rock weirs comprised of loose rocks that extend across the entire width of the channel. Results from the quantitative evaluation of field performance will inform other aspects of the research effort and assist engineers, planners, and managers in improving rock weir designs through increased focus on critical design parameters.

2. Methods

2.1. Delineation of Structure Success or Failure

In a report documenting qualitative evaluations of rock weir field performance, Mooney et al. (2007) identified the structural integrity of each rock weir visited and common failure mechanisms. Determination of structure success or failure is complicated by the definition of success, whether it was sufficient fish passage, adequate head for irrigation diversion, habitat complexity, or other project goals. For the purpose of the present research, failures were categorized as either partial or full failures. Partial failures were those that may have undergone some minor shifting of the rocks from the original placement, but the structures were still meeting intended purposes to some extent. Full failures were characterized as those structures that required significant design modifications post-construction, those that have substantially departed from the original design, or those that were no longer serving their functional role. Mobility of the constituent rocks occurs when one or more piece of the structure moves out of the original alignment. Structures may continue to at least partially perform their intended function despite experiencing some degree of motion.

Potential failure mechanisms outlined in the qualitative investigation are described in Table 1. These were the mechanisms identified through the field investigations, but the table does not represent an exhaustive list of all potential failure modes. Other potential causes of failure for river spanning rock structure noted by design engineers include geological properties of poorly suited rock material and poor selection of structure location.

Table 1. Descriptions of each hypothesized failure mechanism

Growth of Scour Pool	Geotechnical failure due to an increase in the depth of the scour pool. The failure commonly resulted in shifting of the footer rock followed by tilting of the header, often into the downstream scour pool.
Sliding or Rolling	Movement of the rock material due to physical forces of incipient motion.
Filling and Burying	Substantial filling both upstream and downstream of the scour pool resulting in no defined scour pool downstream of the structure
General Bank Migration/ Flanking	Migration around the structure or flanking of the bank due to lack of a sufficient tie-in or lateral channel migration processes (e.g., around the outside of a structure bend).
Piping through arm resulting in flanking	Substantial water flowing between the crest rocks comprising the arm or localized scour between the arm and the bank.
Piping underneath header rocks	Substantial water flowing between the header and the footer rocks, resulting in a reduction in the upstream and downstream water surface elevation difference.

2.2. Surveys

Of the 127 structures evaluated, topographic surveys were performed on 76 river-spanning loose rock structures between June 2005, and November 2007. As part of these surveys, some of the structures with limited data in Mooney et al., 2007 were revisited in 2008, to complete the data set for these structures. Additional sites were visited as well in 2008 to extend the size of the data set. Sixty-nine of these structures were included in the quantitative analysis. The remainder of the original 127 structures either were non-river spanning, not rock structures, too newly constructed (minimal flows experienced), or were not sufficiently surveyed to appropriately quantify structure dimensions. Surveyed data points were classified by common definitions (structure header, structure footer, scour pool, thalweg, etc.). Structures surveyed include A-weirs, U-weirs, Asymmetrical U-weirs, W-weirs, and VW-weirs (Figure 1). A consistent method of identifying arms and angles was decided upon for consistency. The most commonly surveyed structures were U-weirs and A-weirs. Examples of both are included in this section.

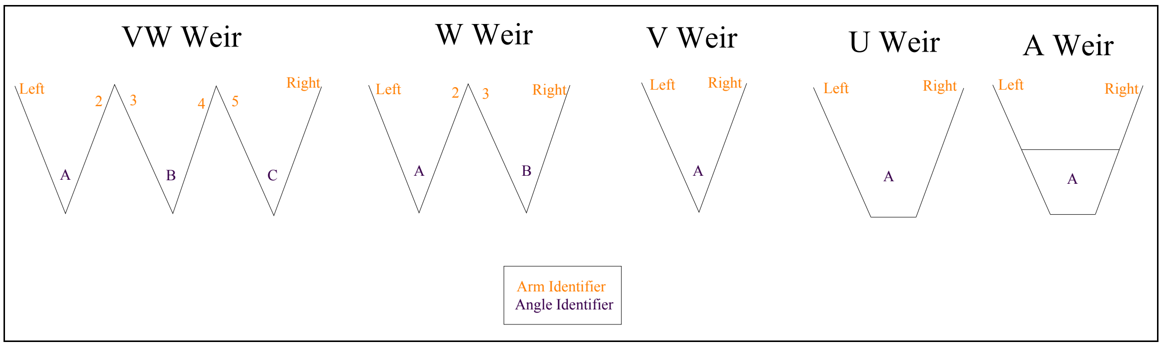

Figure 1. Arm and Angle Identifiers for surveyed structure types.

2.3. Measurements of Structure Parameters

Following collection of the surveys, the data were imported into ArcMap (ESRI Version 9.2). Structure planforms were digitized within ArcMap, along with other site characteristics such as bank line locations, structure opening, location of maximum scour depth, etc. Figure 2 shows an example of two U-weir structures with the digitized lines overlaying the topographic survey. The digitized lines were used to calculate parameters such as arm lengths, throat widths, cross arm lengths, bankline and structure angles, and scour distances to points on the structure.

All structure parameters were gathered into a single database to determine the ranges of different elements of constructed rock structures and to develop any statistical correlations that could be used to direct the design of future structures to prevent or reduce the occurrence of the observed failure mechanisms. A brief description of each parameter and the method used in the calculation of their value is provided below. An idealized A-weir structure is shown in the following section to present the parameters visually. All parameters are listed and described in Appendix A.

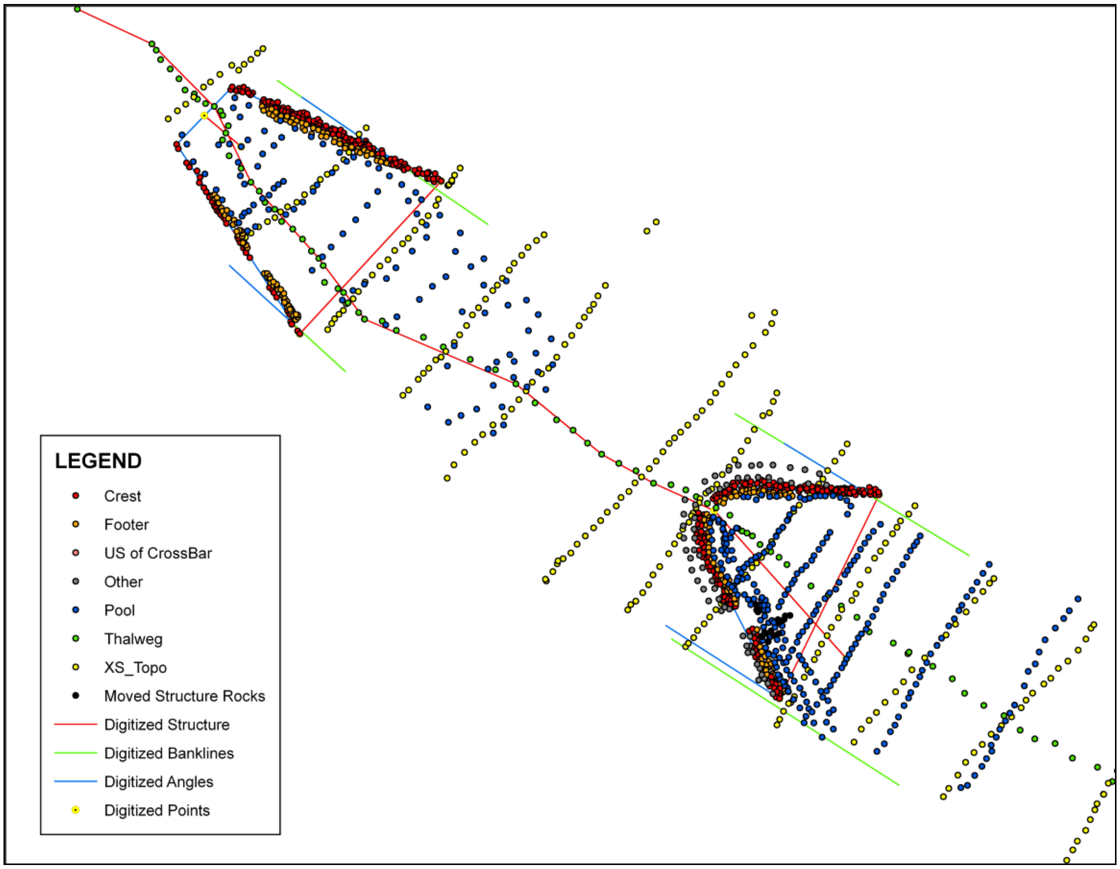

Figure 2. U-weir structures on Entiat River with digitized lines used to collect the parameters.

2.3.1. Lengths

Figure 3 shows a plan view of a typical A-weir structure and the length parameters measured from the digitized structures.

Throat Width, Cross-Bar Width

The throat is defined as the portion of the structure that is perpendicular to the flow of water and provides a drop in water surface elevation. In A-weirs, the cross-bar is also perpendicular to the flow, provides another drop in water surface elevation, and is typically positioned about halfway down the structure arm. These parameters were calculated by measuring the length between the end points along the feature.

Left Arm Length, Right Arm Length, Other Arm Length, Left Arm Tie-In, Right Arm Tie-In

The arm length is the portion of the structure from the end of the throat to the downstream extent of the arm. The arm tie-in is the part of the structure from the downstream extent of the arm to the point at which the structure is keyed into the bank. These parameters were calculated by measuring the length between the end points along the feature.

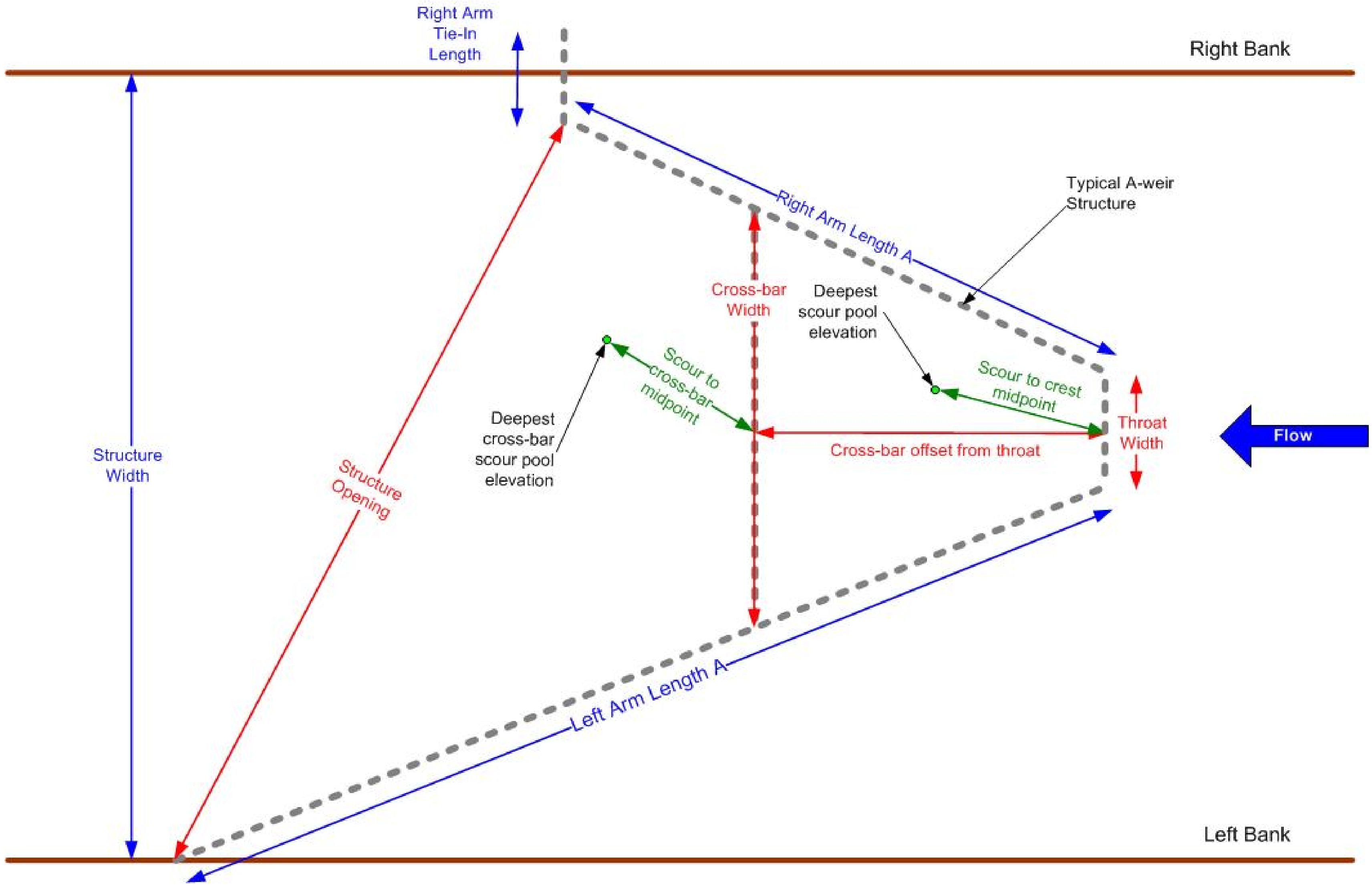

Figure 3. Schematic of A-weir structure showing length of parameters measured.

Structure Opening
The structure opening is defined as the distance between the downstream extent of the arms. This parameter was calculated by measuring the length between the downstream endpoint of the left arm and right arm.

Structure Width
The structure width is defined as the distance between the left top of bank and the right top of bank. This parameter was calculated using the perpendicular distance between the bank lines at the downstream end of the structure.

Cross-bar offset from throat
The cross-bar offset is defined as the distance along the river between the cross-bar midpoint and the throat midpoint. The length between the midpoint of the cross-bar and the midpoint of the throat was calculated for this parameter.

Scour to crest midpoint, scour to cross-bar midpoint
The length between the deepest point in the scour pool (throat scour or cross-bar scour) and the midpoint of the throat or cross-bar was calculated for this parameter. The deepest point in the scour pool was found by selecting all surveyed points labeled scour and sorting to find the lowest elevation. If scour points were not delineated in the survey, all points within the structure extents were selected to determine the deepest point.

Left Lateral Constriction, Right Lateral Constriction
This parameter was used to determine whether or not the structure was actually river spanning (extended beyond the channel width). The lateral constriction is the distance, if any, between the extent of the arm and the bankline. This value is equal to zero if the arm's downstream extent extends beyond the bank line. If the arm's downstream extent is not beyond the bankline, this value uses the ArcGIS near function to find the closest distance between the arm's downstream extent and the digitized bankline. In Figure 4, the right arm is tied into the bank because the arm extent extends beyond the bankline. If the left tie-in is ignored, the left arm has a lateral constriction greater than zero since it does not extend past the bankline. However, since the structure in Figure 4 does have a tie-in that extends beyond the left bankline, the left lateral constriction is zero.

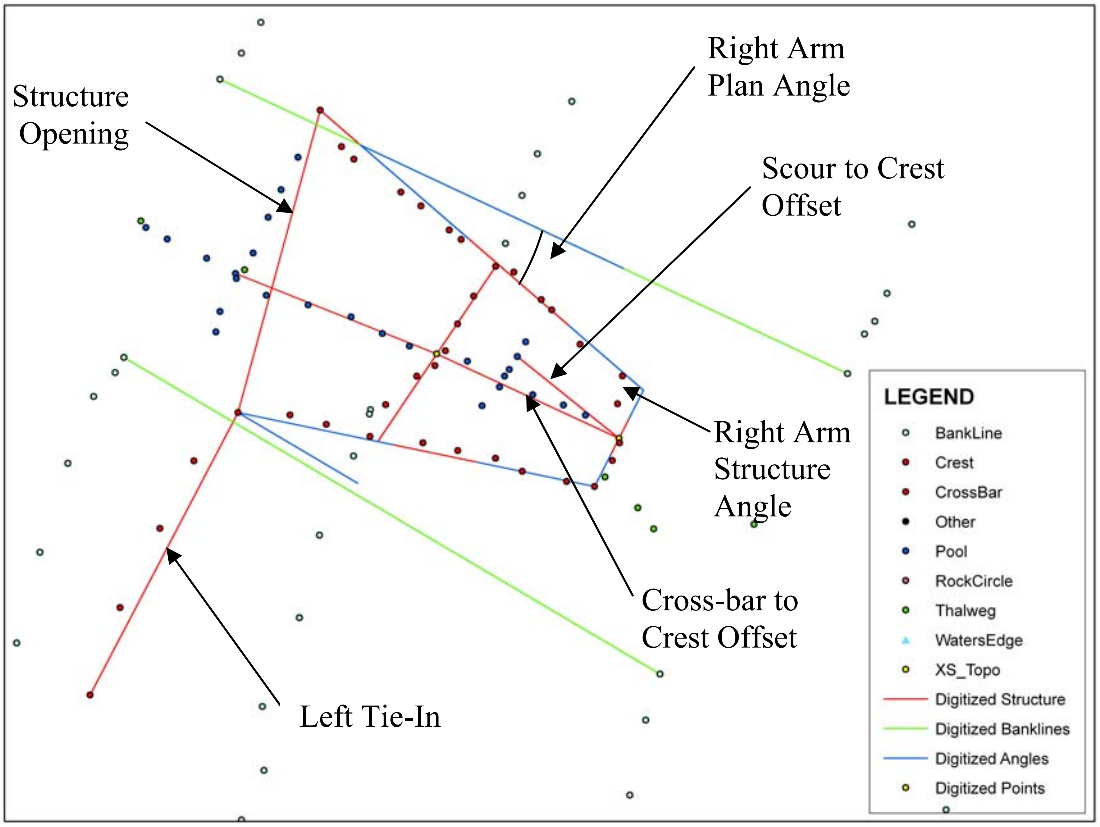

Figure 4. A-weir structure K at Rio Blanco site.

Upstream Structure, Downstream Structure

This parameter is defined as the distance from the structure being analyzed to the nearest upstream (or downstream) structure having a hydraulic influence on the river. Distances between structures were measured along the channel thalweg. When measuring between two river spanning rock structures, this parameter represents the distance from the midpoint of the structure throat to the midpoint of the next structure's throat. If the next structure being measured to was not a river spanning rock structure (e.g., the next upstream/downstream structure was J-hook), then this parameter represents the distance from the midpoint of the structure throat to the most upstream extent of the next structure. Thalweg survey points were used to determine the channel path along which to measure distances if they were collected in the field. If not, National Agriculture Imagery Program (NAIP) (USDA, 2009) or other aerial photography was used to identify the channel path. The distances were converted into channel widths, assuming the structure opening was equal to one channel width.

2.3.2. Angles

Figure 5 shows a plan view of a typical A-weir structure and the angle and offset parameters measured from the digitized structures.

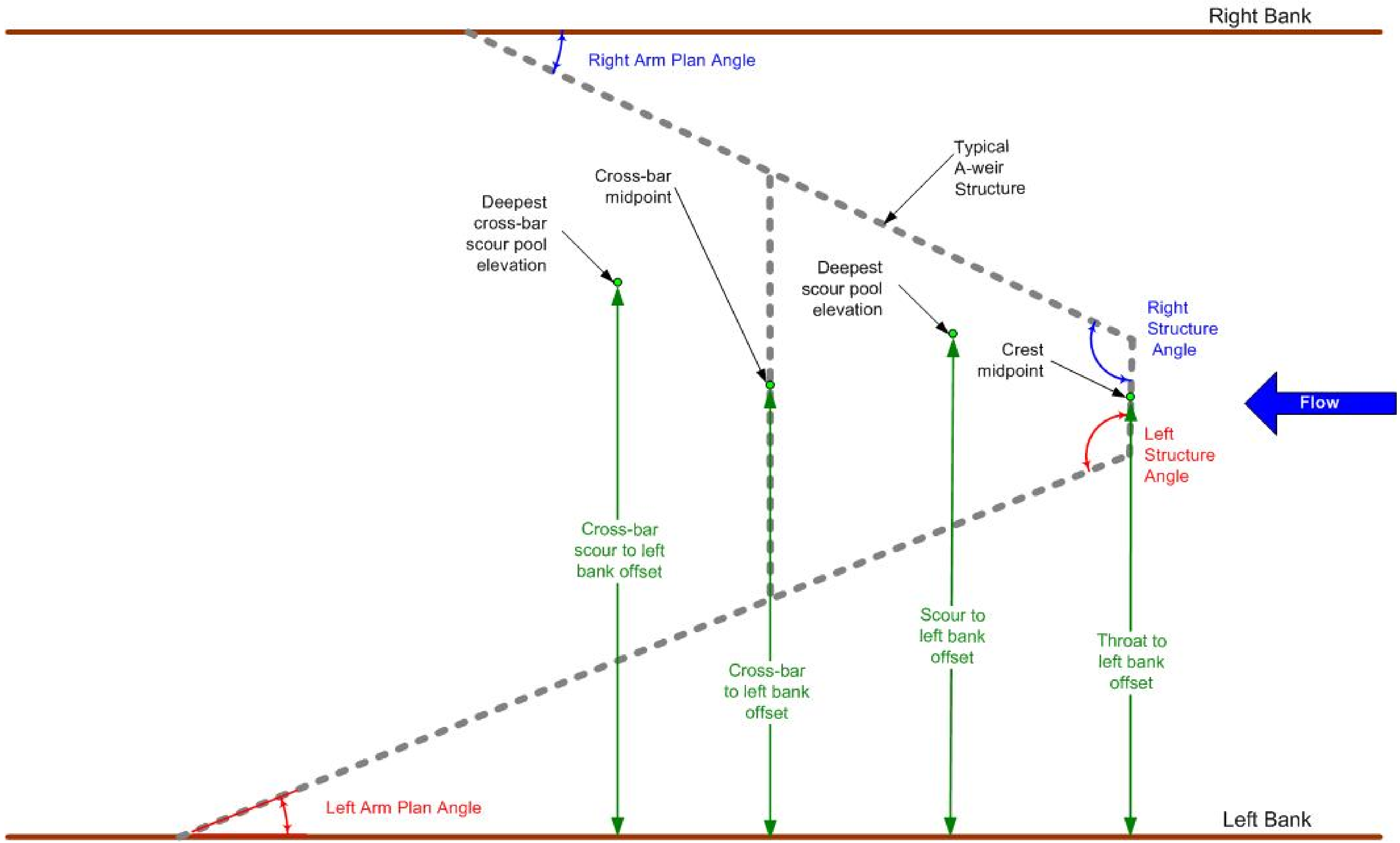

Figure 5. Schematic of A-weir structure showing angle and offset parameters measured.

Left Arm Angle, Right Arm Angle
The arm angles are defined as the minor angle between the structure arm and the bankline. These parameters were calculated by measuring the minor angle between the selected arm and the corresponding bankline (right arm with right bankline). The calculation was performed by utilizing a script that was written to calculate angles between two polylines within ArcMap. If the lateral constriction was greater than zero, the corresponding bankline was copied and placed at the downstream extent of the arm to calculate the angle. See Figure 5 for an example.

Left Arm Structure Angle, Right Arm Structure Angle
The arm structure angles are defined as the interior angle between the structure arm and the structure throat. These parameters were calculated by measuring the angle between the selected arm and the throat utilizing the same script described above. See Figure 5 for an example.

Open Angle
The open angle is the angle that structure arms make relative to each other. For V weirs and W weirs, it is the angle two arms make relative to the upstream point. The same script used within ArcMap to calculate left and right arm angles was used to calculate the open angle for V weirs and W weirs. For A weirs and U weirs, the open angle is the sum of the left and right arm structure angles minus 180 degrees.

2.3.3. Offsets

Throat(s) offset, Cross-bar(s) offset
These offsets are defined as the shortest distance between the specified point and the left bankline. This parameter was calculated by using the ArcGIS near function to find the closest distance between the midpoint of the throat (or cross-bar) and the left bankline.

Scour to left bank offset, Cross-bar scour to left bank offset
These offsets are defined as the shortest distance between the specified point and the left bankline. This parameter was calculated by using the ArcGIS near function to find the closest distance between the deepest point of the throat (or cross-bar) scour pool and the left bankline.

2.3.4. Elevations

Scour Pool Elevation, Cross-bar Pool Elevation
This parameter is the elevation surveyed at the deepest point in each scour pool.

Crest Elevation, Cross-bar Elevation, Left Tie-in Elevation, Right Tie-in Elevation
The crest (or cross-bar) elevation was calculated by averaging the elevations of all the surveyed points recognized to be applicable to the digitized throat (or cross-bar) line. Similar averaging was used to identify left and right tie-in elevations.

2.3.5. Profile

Figure 6 shows a profile view of a typical A-weir structure and the elevation and profile parameters collected from the digitized structures.

Scour Depth

The scour depth is defined as the difference in elevation between the crest at the structure throat and the deepest point of the scour pool. The digitized site characteristics were used to identify structure points with which profile characteristics (depths and heights) could be dimensioned within Microsoft Excel. This parameter was calculated by taking the difference between the average crest elevation at the throat and the scour pool elevation at its deepest point.

Cross-bar Scour Depth

The cross-bar scour depth is defined as the elevation drop between the average elevation along the cross-bar and the deepest point in the cross-bar scour pool. The cross-bar scour depth was calculated by measuring the difference between these two parameters.

Left Tie-in Height, Right Tie-in Height

The tie-in heights are defined as the elevation difference between the crest and the tie-in endpoint. The relative heights of the left tie-in and right tie-in were calculated by taking the difference of the appropriate elevation and the crest elevation.

Left Arm Profile Angle, Right Arm Profile Angle, Other Arm Profile Angle

The arm profile angles are between a level horizon and the arm elevation change. See Figure 6 for an example. Arm profile angles were calculated by first arranging the structure arm points (as identified using engineering judgment) in order by distance from the structure throat and calculating the distance of those points to the appropriate (left or right) extent of the arm. The slope function in Excel was utilized to regress a trend through the arm profile points to produce an arm slope for the structure.

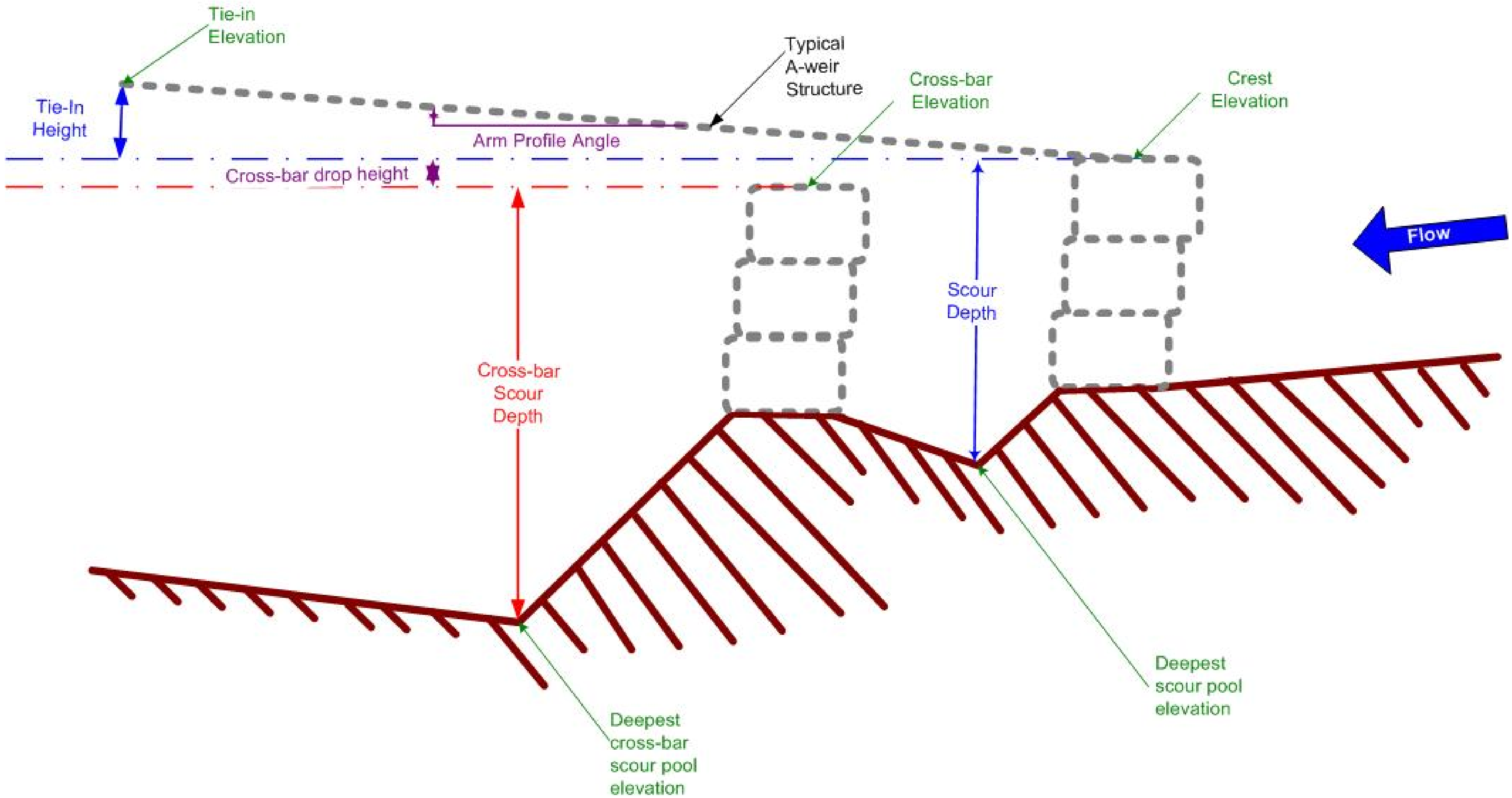

Figure 6. Schematic of A-weir structure profile showing elevation parameters measured.

2.3.6. Additional Parameters

Structure Planform Location

This parameter was determined by using the NAIP photographs or other available aerial photographs to look at the channel morphology of the river where the structure was constructed. The options included a straight section, a section with the river bending to the left (looking downstream), a section with river bending to the right, or a crossing (or transitional section) of the river between a left and right bend. Figure 7 illustrates the location descriptions.

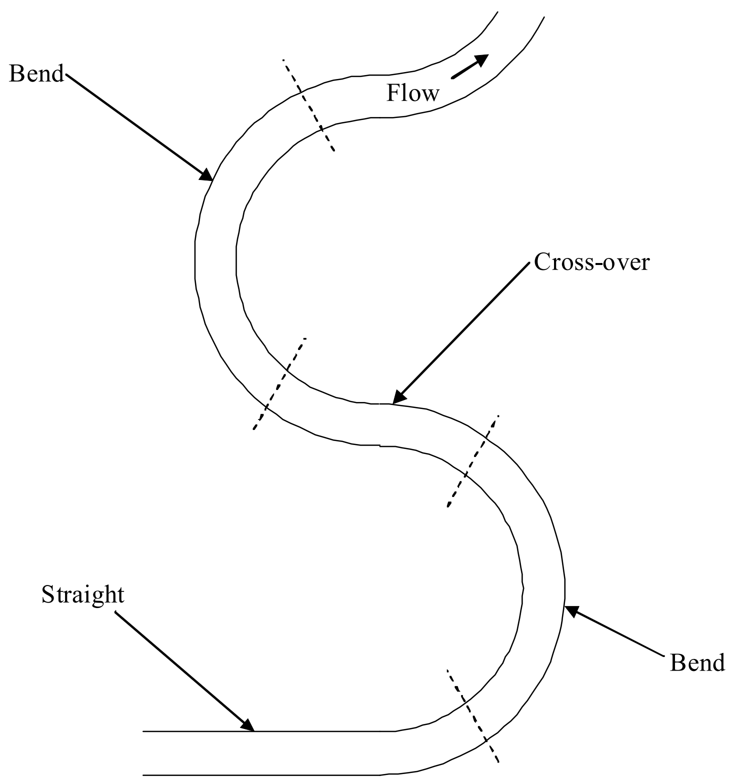

Figure 7. Channel morphology locations used in parameter descriptions.

2.4. Hydrology

Structure permanence is influenced by the magnitude of flows that it has experienced since construction. A hydrologic analysis (Appendix B) was conducted as part of this investigation to improve linkages between structure failures (as defined by constituent rock mobility) and flood discharges. The objectives of the hydrologic analysis were to provide:

- A description of streamflow data available at rock structure locations;
- Flood frequency analyses at rock structure locations, and
- Estimated recurrence intervals and magnitudes of the largest flood since construction of each rock structure.

First, the nearest USGS stream gages were identified for the structures surveyed during field investigations. If a USGS stream gage with sufficient data was not located on the same tributary, the nearest gage with the most representative flow patterns was assumed to be applicable to the tributary of interest. Previously completed flood frequency analyses for the Entiat and Methow Rivers (Sutley, 2009; Sutley, 2006) were incorporated into this report for structures on the Entiat River, Chewuch River, and Beaver Creek. For all other structures visited, annual peak discharge data for the USGS stream gages were used to estimate the frequency discharges. Estimates of discharges with 2-, 5-, 10-, 25-, 50-, and 100-year recurrence intervals were calculated for each gage and adjusted by contributing drainage area to account for the distance between the gage and the structure location. From this information, an approximation of the recurrence interval and magnitude of the greatest discharge since the construction was developed for each structure location (Table 2). A complete description of the hydrologic analysis is provided in Appendix B.

Table 2. River Spanning Rock Structure recurrence interval and magnitude since construction.

Structure Location	Earliest Potential Construction Year	Date of Largest Discharge between Contruction Year and Site Visit	Magnitude of Discharge (ft^3/s)	Estimated Recurrence Interval of Discharge (years)
Bear Creek	1999	5/30/2003	2200	100
Beaver Creek	2000	5/19/2006	690	25
Catherine Creek	1998	5/30/2003	1900	>100
Chewuch River	2007	5/17/2007	2800	2
East Fork of the Salmon River	1998	5/21/2006	2500	30
Salmon River	2006	5/21/2008	1800	25
Entiat River, Structures 3.1, 3.2, and 4.6	2001	5/19/2006	4700	5
Entiat River, Structure 3.4	2006	6/4/2007	3600	~2
Entiat River, Structure 5.1	2007	5/19/2008	3400	~2
Grande Ronde	1998	6/16/1999	3200	3.5
Lemhi	2002	5/31/2003	1300	3
Middle Fork of the Little Snake River	2001	5/31/2003	nodata	3
North Fork of the Little Snake River	2001	5/31/2003	nodata	3
Middle and North Forks of the Little Snake River	2001	5/31/2003	nodata	3
South Fork of the Little Snake River	2001	5/31/2003	nodata	3
Rio Blanco	1999	5/23/2005	2300	3
San Juan River	1995	5/23/2005	4700	6.5

3. Results

3.1. Distribution of each structure by type and whether failure, partial failure, or success.

In the qualitative investigation of in-channel structures (Mooney et al., 2007), a total of 127 structures were evaluated to identify potential failure mechanisms. These included a myriad of structure types, including river spanning rock structures (RSRS) and non-river spanning rock structures. This quantitative report focuses on river spanning rock structures. Further, the type and amount of data collected at each structure varied greatly, limiting the number of structures that could be included in this quantitative analysis. Some of the structures with limited data in Mooney et al., 2007 were revisited in 2008 to complete the data set for these structures. Additional sites were visited as well in 2008 to extend the size of the data set. A total of 69 river spanning rock structures were identified as suitable for quantitative analysis. Note, however, that the degree of failure could not be identified for one of the asymmetrical U-weirs, yielding a dataset of 68 structures. Table 3 presents the structures by type, where a “usable” structure is a river spanning rock structure with sufficient data to describe the physical geometry of said structure. Table 4 presents the river spanning rock structures that are considered in the results section, broken down by degree of failure. A-weirs and A-weir rock ramps from Table 1 are combined as simply A-weirs in Table 4. The same is true for V-weirs and V-weir rock ramps.

Two W weirs and one VW weir were surveyed with enough data to be usable during the field evaluation. Due to the small number of each of these structures, no quantitative comparison could be conducted across the individual structure types to determine which parameters are important to structure success. Note that both the W-weirs and the VW-weir failed. In addition to these 65 structures to be considered (68 less two W-weirs and one VW-weir), eight U-weir structures on Red Hill Creek in Canada were surveyed in November 2007. These eight Red Hill Creek structures were surveyed shortly after installation and had not experienced any substantial flows, so identifying a degree of failure and failure mechanisms for these structures was not possible (Bill Annable, 2007). These Red Hill structures were compared to the other U-weirs in terms of geometry, but not in terms of failure. The Red Hill structures are not included in Table 3 or Table 4.

Some structures are installed in series along a river, sometimes with structure types varying from one to the next. Distances to upstream and to downstream structures were identified for those that were not stand-alone structures. Also, if the next structure (upstream or downstream) was not a river spanning rock structure, then the distance to the next river spanning rock structure was measured. Table 5 presents the number of structures that did have upstream and downstream structures, whether river spanning or not.

The parameters presented in section 2.3 help to describe the geometry of a structure. Some parameter values common to U-weirs, V-weirs, A-weirs, and asymmetrical U-weirs are presented in Table 6, along with maximum, minimum, and mean values, delineated by degree of failure. Table 7 presents additional data relevant to A-weirs in terms of cross bars. Figures 3, 5, and 6 show how the parameters in Table 6 and Table 7 are measured for a typical A-weir. Note that one A-weir had dual cross bars. Also note that only one A-weir was categorized as a non-failure. Due to a lack of A-weirs that did not fail and the fact that only two A-weirs were full failures, further observation of trends between degree of failure and various parameters is invalid. U-weirs and V-weirs are combined into the category U,V-weirs, with V-weirs being described as U-weirs with throats of zero-width. Trend observations will primarily be focused on U,V-weirs and asymmetrical U-weirs.

Table 3. Structures visited by type between 2005 and 2008

Structure Type	Count	Usable
Angled Rock Dam	1	0
Asymmetric U-weir	22	19
A-weir	12	9
A-weir Rock Ramp	1	1
J-hook	28	0
Partial Channel Rock Ramp	1	0
Pole Weir	2	0
Rock Ramp	1	0
Triple U	1	0
Unknown	4	0
U-weir	44	33
U-weir with support	8	0
V-weir	7	3
V-weir Rock Ramp	2	1
VW-weir	1	1
W-weir	3	2
Total	**138**	**69**

Table 4. Usable structures considered by degree of failure

Structure Type	Count	Usable	No Fail	Partial Fail	Fail	Unknown
Asymmetric U-weir	22	19	3	9	6	1
A-weir	13	10	1	6	3	0
U-weir	44	33	7	11	15	0
V-weir	9	4	0	1	3	0
VW-weir	1	1	0	0	1	0
W-weir	3	2	0	0	2	0
Total	**92**	**69**	**11**	**27**	**30**	**1**

Table 5. Count of structures with upstream (US) and downstream (DS) structures of any type and with upstream/downstream river spanning rock structures (RSRS).

Structure Type	US Structure	US RSRS	DS Structure	DS RSRS
Asymmetric U-weir	19	16	18	18
A-weir	5	4	5	5
U-weir	28	27	27	24
V-weir	2	2	3	2
VW-weir	0	0	0	0
W-weir	0	0	2	1

Table 6. Value ranges for select parameters common to all structures by degree of failure.

	2yr Discharge (cfs)	General Structure Spacing (river widths)	Max arm Length / Structure Width	Max Plan Angle	Max Scour (ft)	Max Structure Arm Profile Slope (ft/ft)	Open angle A (deg)	Recurrence Interval of High Flow	RSRS Spacing (river widths)	Scour Depth / Scour Offset	Scour Depth A (ft)	Structure Opening (ft)	Structure Width (ft)	Thalweg Slope (ft/ft)	Throat width (ft)	Tie in length (ft)	Tie in length / Structure Width
max	3100.0	12.3	1.7	61.9	4.7	0.2	99.1	100.0	12.3	1.0	4.7	103.4	140.4	0.0	39.5	34.8	0.6
min	230.0	0.5	0.2	13.8	1.0	0.0	20.7	2.0	0.5	0.1	1.0	19.1	13.1	0.0	0.0	0.0	0.0
mean	1671.9	3.1	0.8	35.7	2.8	0.1	56.7	18.5	3.4	0.4	2.8	41.8	52.9	0.0	10.5	4.1	0.1
max	2130.0	4.0	1.2	31.9	3.9	0.2	59.1	3.0	5.6	0.6	3.9	45.5	69.1	0.0	15.3	34.8	0.6
min	2130.0	0.9	0.5	13.8	2.1	0.0	20.7	3.0	1.5	0.2	2.1	19.7	15.2	0.0	7.4	0.0	0.0
mean	2130.0	2.3	0.9	24.2	2.9	0.1	39.3	3.0	3.2	0.4	2.9	30.4	43.4	0.0	10.6	14.5	0.2
max	3100.0	12.3	1.7	60.3	4.5	0.2	99.1	25.0	12.3	1.0	4.5	92.7	104.7	0.0	20.7	31.7	0.5
min	230.0	0.5	0.3	13.9	1.0	0.0	28.0	2.0	0.5	0.1	1.0	19.8	13.1	0.0	0.0	0.0	0.0
mean	1456.7	3.8	0.9	38.9	2.5	0.1	57.1	9.4	3.9	0.4	2.5	35.0	41.9	0.0	8.4	2.6	0.0
max	3100.0	7.0	1.6	61.9	4.7	0.2	94.1	100.0	7.0	0.9	4.7	103.4	140.4	0.0	39.5	17.8	0.3
min	230.0	0.5	0.2	18.8	1.4	0.0	33.8	3.0	0.5	0.2	1.4	19.1	14.2	0.0	0.0	0.0	0.0
mean	1637.2	2.9	0.8	38.6	3.1	0.1	63.3	31.1	3.1	0.4	3.1	50.7	63.9	0.0	11.9	1.0	0.0

Black = All Structures, Blue = No Failure, Green = Partial Failure, Red = Failure

Table 7. Value ranges for select parameters relevant to A-weirs by degree of failure.

	CrossBar Width (ft)	CrossBar 2 width (ft)	CrossBar offset from throat (ft)	CrossBar 2 offset from throat (ft)	CrossBar Scour to Crest offset (ft)	CrossBar 2 Scour to Crest offset (ft)	Cross bar scour depth (ft)	CrossBar 2 scour depth (ft)	CrossBar Drop Height (ft)	CrossBar 2 Drop Height (ft)
max	56.6	43.6	37.0	69.5	26.1	8.0	5.5	4.5	1.9	1.8
min	14.6	0.0	8.0	0.0	2.9	0.0	1.7	0.0	0.4	0.0
mean	28.6	43.6	21.5	69.5	13.8	8.0	2.9	4.5	1.0	1.8
max	30.4	43.6	24.4	69.5	17.5	8.0	3.5	4.5	1.9	1.8
min	14.6	43.6	8.0	69.5	2.9	8.0	1.7	4.5	0.8	1.8
mean	20.6	43.6	16.9	69.5	9.9	8.0	2.4	4.5	1.2	1.8
max	39.9	N/A	37.0	N/A	18.2	N/A	5.5	N/A	0.5	N/A
min	33.1	N/A	19.4	N/A	17.0	N/A	1.7	N/A	0.5	N/A
mean	35.4	N/A	25.9	N/A	17.5	N/A	3.4	N/A	0.5	N/A

Black = All Structures, Green = Partial Failure, Red = Failure

3.2. Observable Trends of Degree of Failure with Structure Parameters

A comparison of parameters across all structures was presented in Table 6. A further comparison of parameters by degree of failure for each structure type was performed. U-weirs and V-weirs were combined, with V-weirs being described as U-weirs with throats of zero-width. U-weirs and V-weirs are combined into the category U,V-weirs. A table listing select parameter ranges by structure type and degree of failure is presented in Appendix C. In this section, ranges of selected structure parameters are compared across the degree of structure failure to determine the presence of trends and identify where strong relationships exist between specific design parameters and degrees of failure.

3.2.1. Planform Location

One variable evaluated was the planform location of the structure. As described in the previous section, the four optional locations are left bend, right bend, crossing, or straight reach. Degree of failure was compared to structure location. Of the structures that were visited, nearly two times more structures were located in bends than in straight reaches or crossings. One hypothesis relating this parameter to failure was that structures located along meander bends may experience more lateral channel migration process and therefore be more prone to failure. Delineating relationships with structure location was difficult because we did not visit an equal number of structures located in straight sections, crossings, and bends, and because the majority of the structures visited had been subject to some degree of failure. In fact, 67 percent of all structures included in the analysis were located on bends and over 80 percent experienced failure. Based on all the structures evaluated across all failure mechanisms, the following patterns were identified:

- 78 percent of structures located on bends failed either fully or partially, while 91 percent of structures in straight reaches or crossings experienced some degree of failure.
- 63 percent of all structures that either partially or fully failed by any mechanism were located on bends.
- All (100 percent) of structures that were identified to have failed either partially or fully by general bank migration/flanking were located along bends.
- 68 percent of the structures that either partially of fully failed as a result of sliding or rolling were located on bends.

Figure 8 presents degree of failure by structure location.

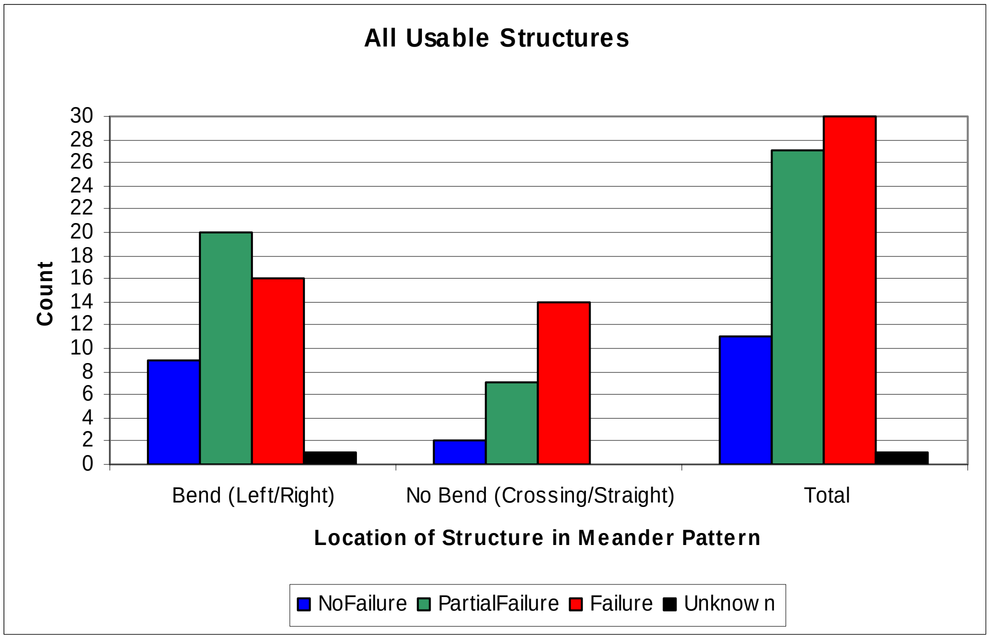

Figure 8. Degree of failure by structure location in meander pattern.

3.2.2. Scour Depth

Scour depth relative to foundation depth was expected to be an important factor in the success of a structure's stability due to the fact that the most prevalent mode of failure observed was the growth of scour pool and subsequent slumping of the header and footer rocks (Mooney et al. 2007). Figure 9 illustrate ranges of scour depths by degree of failure for U,V-weirs. The data indicate no trend directly relating the scour depth to the degree of failure, and this is true for A-weirs and asymmetrical U-weirs as well. However, the lack of a trend may be due to one of two factors; the fact that for some structures the maximum depth of scour was too deep to safely survey, or the fact that scour depth was not evaluated as a ratio of foundation depth (i.e. footer depth). Evaluating scour depth as a ratio of foundation depth is expected to be the most useful way to non-dimensionalize this parameter. Foundation depths were not surveyed in the original data collection effort. Therefore, attempts were made to non-dimensionalize scour depth to parameters that showed significant correlations to scour depth, including arm length, throat width, and structure width. However, these ratios involving scour depth did not show improved trends with degree of failure. Future analyses that incorporate foundation depths may display improved trends with the degree of failure.

Other parameters were explored to capture failure by growth of the scour pool and slumping of the header or footer rocks. The first was to identify the location of the measured maximum scour and calculate its distance to the nearest surveyed crest rock at any point along the structure. The distance was non-dimensionalized using structure width so that the values were comparable across all structures. As Figure 10 and Figure 11 exemplify, the results indicate that structures tended to experience higher degrees of failures with decreasing values (i.e., as the max scour was closer to the crest). In evaluating just those structures that failed primarily or secondarily through growth of scour pool, this relationship strengthened (Figure 12).

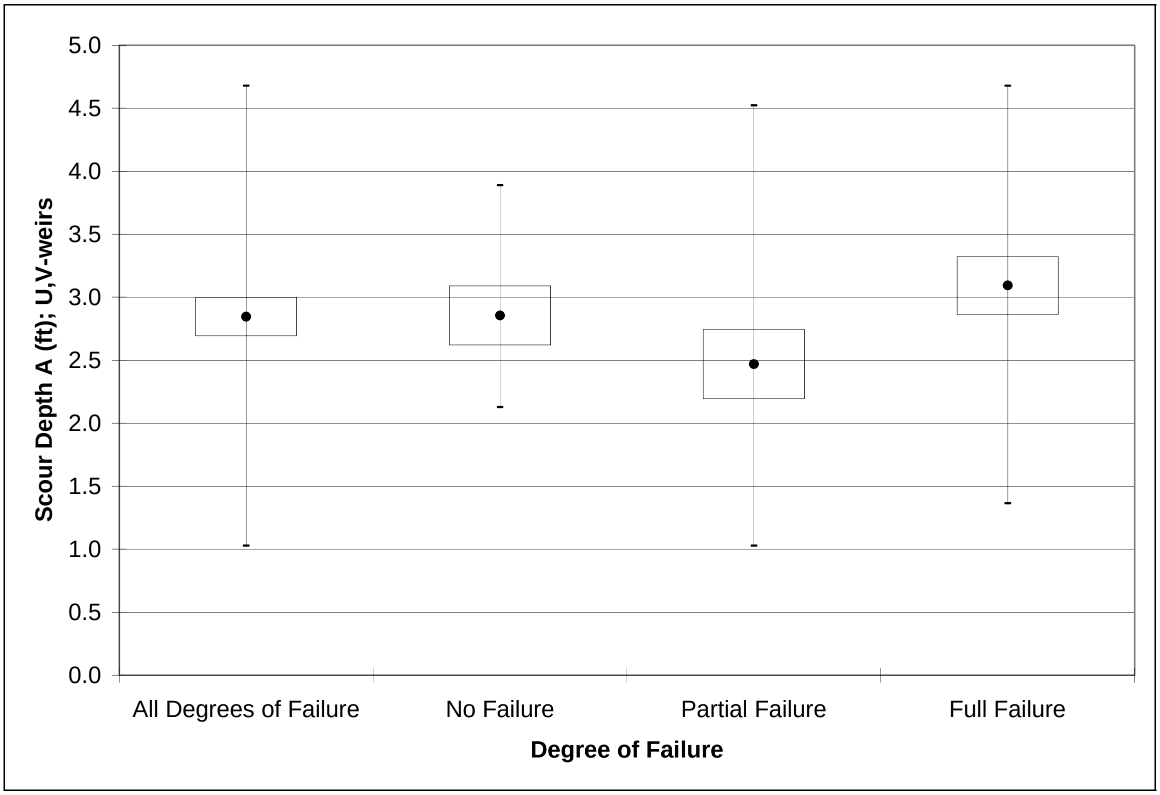

Figure 9. Scour depth by degree of failure for U,V-weirs.

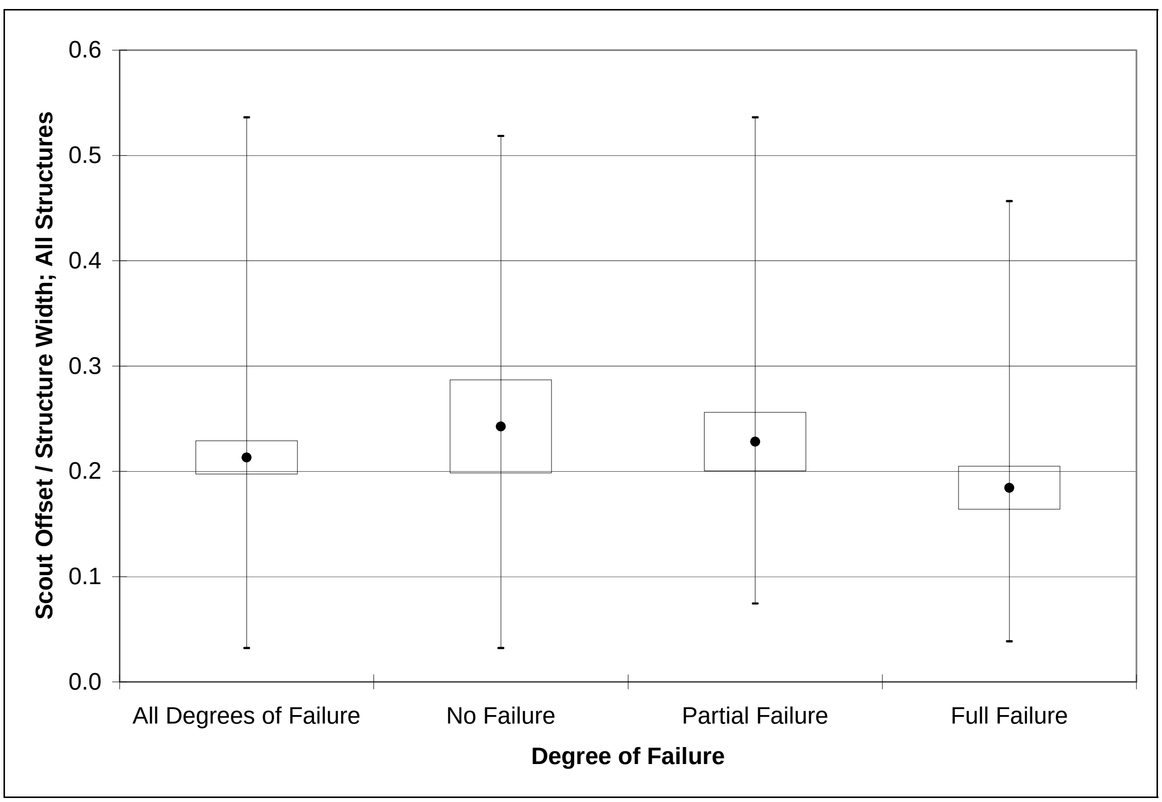

Figure 10. Ratio of scour offset to structure width for all structures.

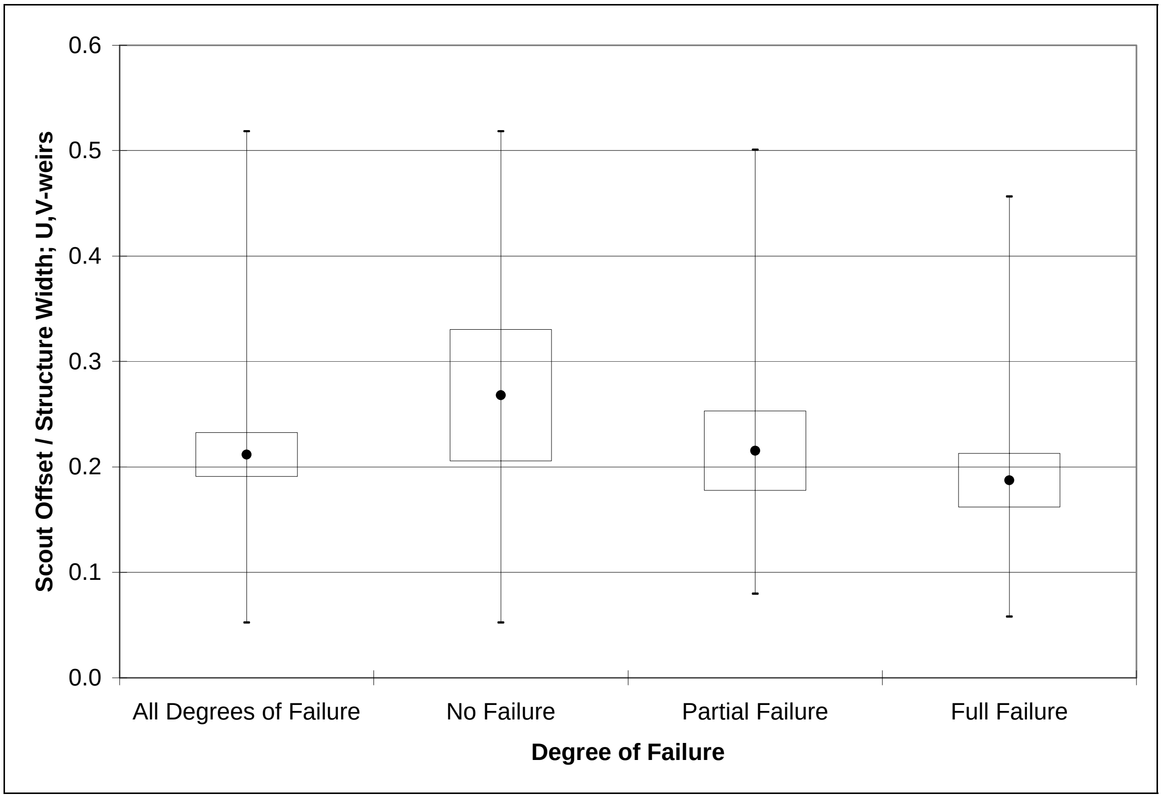

Figure 11. Ratio of scour offset to structure width for U,V-weirs.

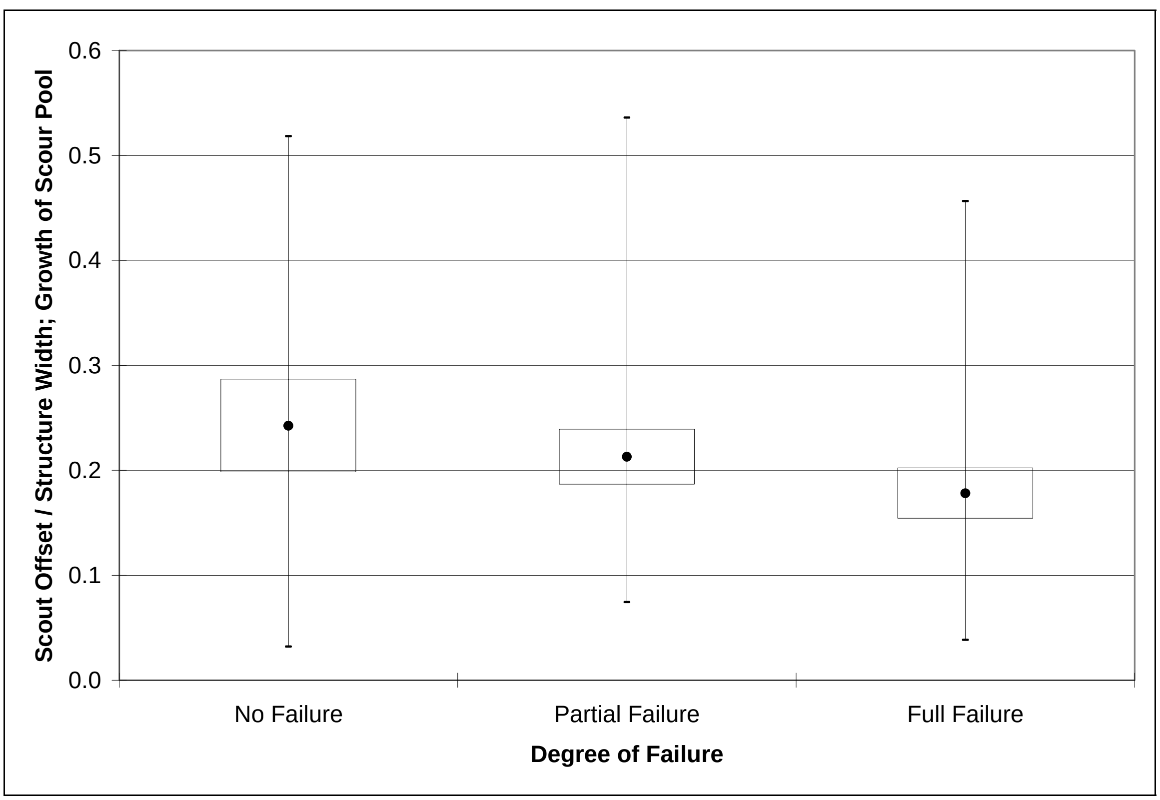

Figure 12. Ratio of scour offset to structure width for structures failing by scour pool growth.

3.2.3. Recurrence Interval of High Flows

Based on the results of the hydrologic analysis, the relationship between high flows and the degree of structure failure was investigated. The hypothesis for this relationship was that structures which have experienced the greatest recurrence intervals since construction were most likely subjected to some degree of failure. Results support this hypothesis, as can be seen in Figure 13. Structures that have not experienced any failure were subject to a high flow discharge with an average of a 3-year return period; structures that have partially failed averaged a high flow discharge with a 7-year return period, and those structures that have completely failed averaged a high flow discharge with a 21-year return period. Figure 14 presents the degrees of failure for structures whose primary or secondary failure mechanism was sliding or rolling. Structures which did not fail or failed only partially by sliding and rolling never experienced a flow event greater than a 10-year return interval storm. These results further support the idea that long-term maintenance of these structures is critical if their purpose is to remain stable, particularly for maintaining a specific head for irrigation diversion.

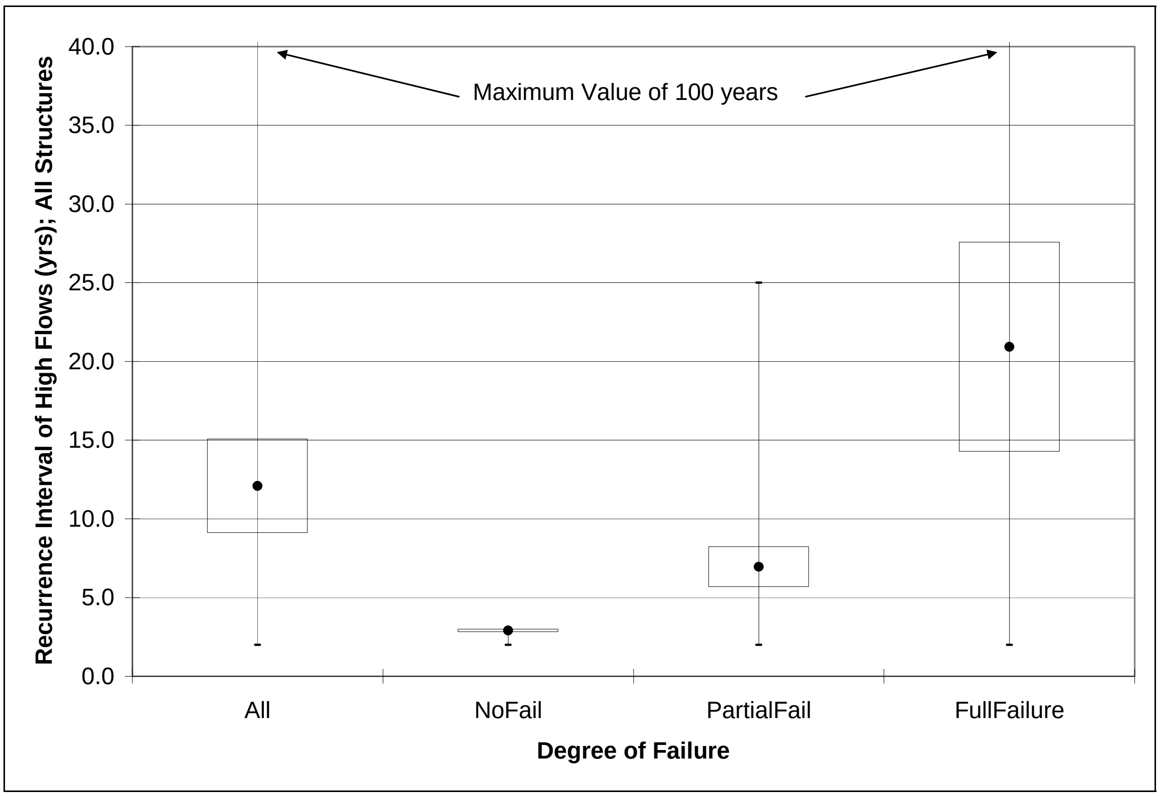

Figure 13. Recurrence interval of high flow events by degree of failure for all structures.

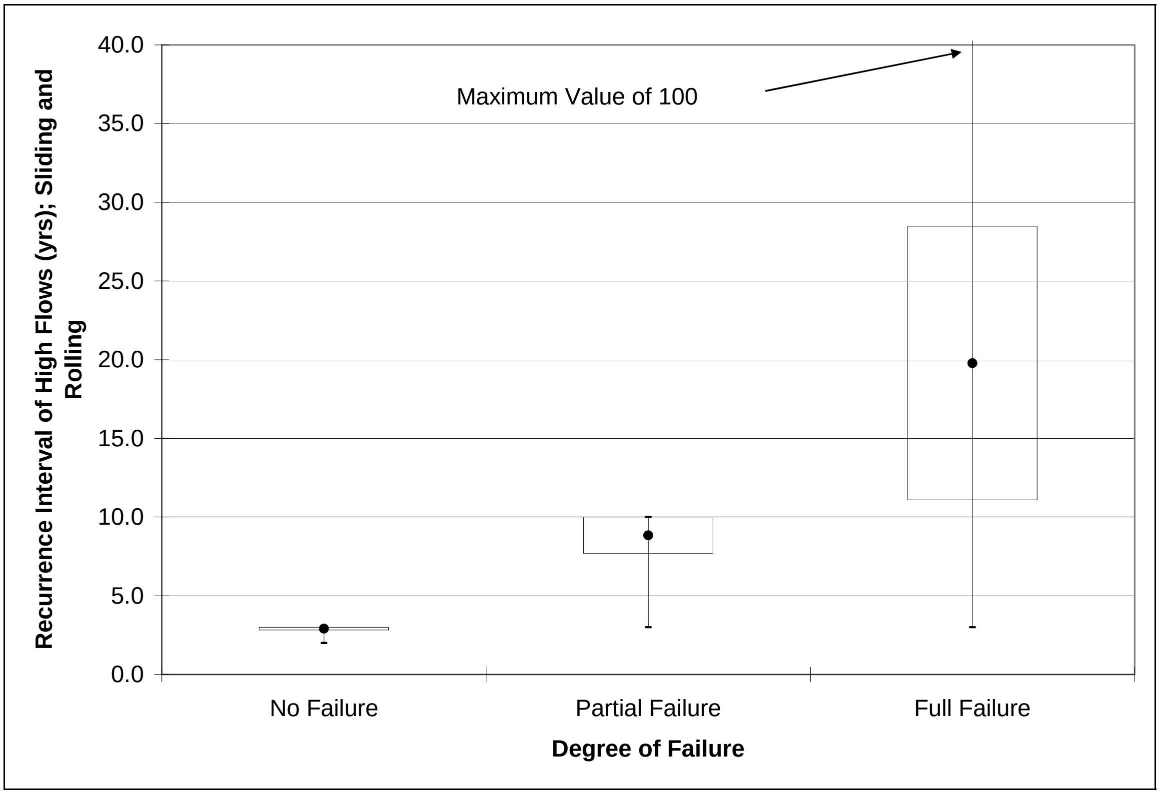

Figure 14. Recurrence interval of high flow events for structures which failed primarily or secondarily by sliding and rolling.

3.2.4. Plan Angles

Three parameters defined under this category include the maximum plan angle (angle between structure arm and bank), the structure arm angle (interior angle between the structure arm and the throat), and the open angle (angle that structure arms make relative to each other). All of these angles are positively correlated to each other to some degree. The correlation coefficient will be presented after a parameter in parentheses to quantify the degree of relation with another variable. Correlation coefficients greater than approximately 0.25 are considered significant ($p = 0.05$) based on the 68 structures included in the measurements.

In evaluating the maximum plan angle, observed patterns suggest that the greater the angle is, the more likely the structure will experience failure. An increasing relationship with the degree of failure is evident when all structures are combined and when analyzing just those structures failing as a results of scour pool growth (Figure 15). For U,V-weirs, a structure was more likely to experience some degree of failure when this angle exceeded 35 degrees (Figure 16). For asymmetrical U-weirs, full failures tended to occur when the angle approached 40 degrees, while partial and no failures had a similar average of approximately 33 degrees (Figure 17). As this angle increases, the structure configuration becomes more perpendicular to flow and the profile arm slope along structure arm increases (0.43), leading to large drop across the structure arm.

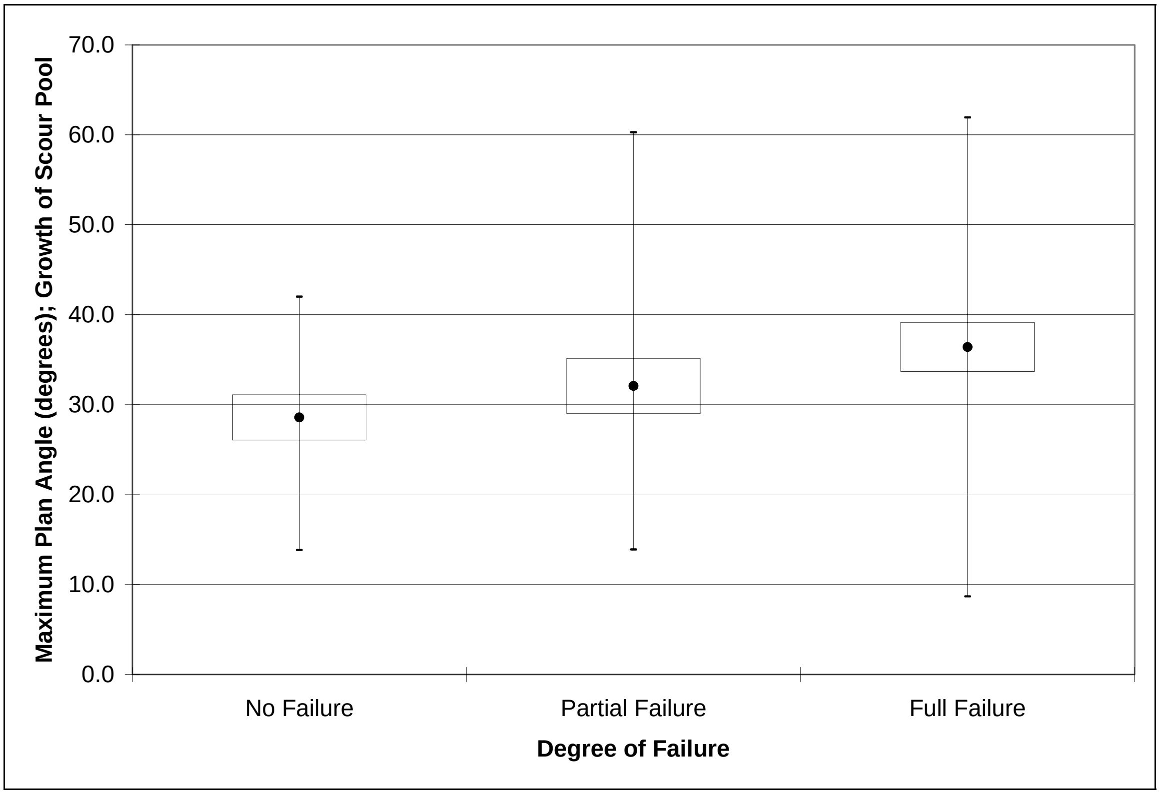

Figure 15. Maximum plan angle by degree of failure for structures failing by scour pool growth.

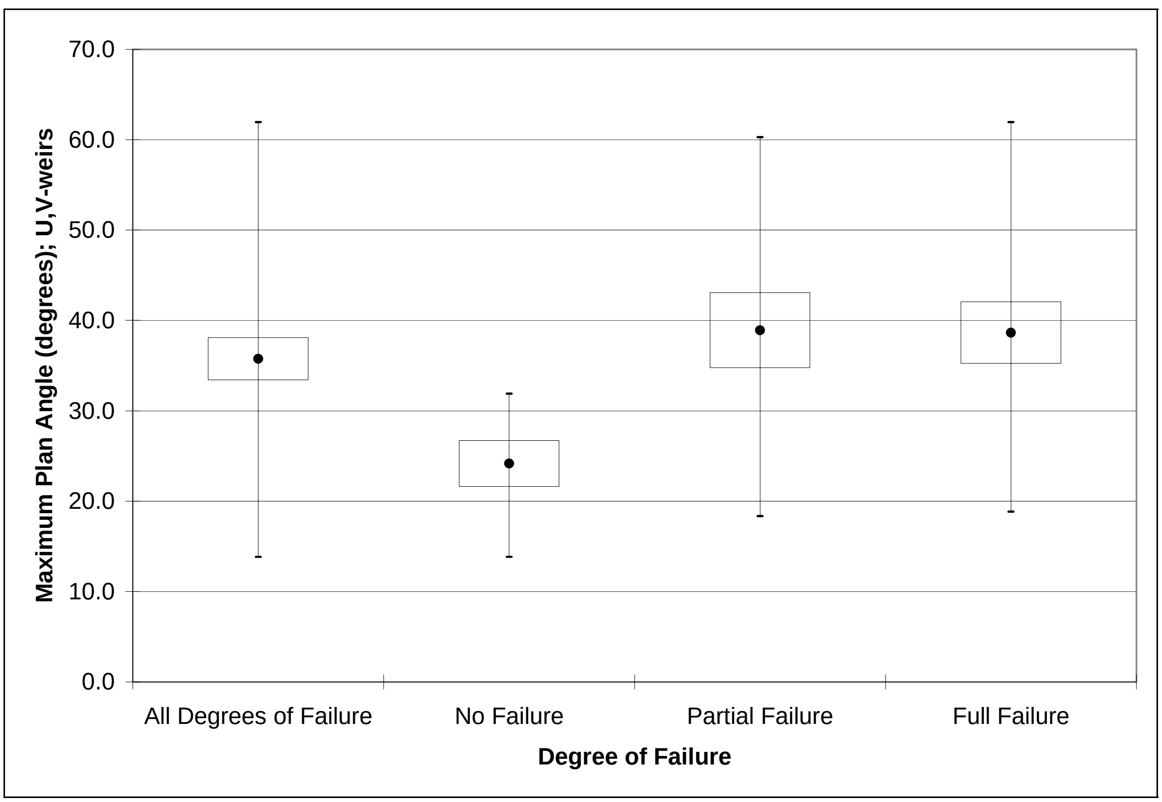

Figure 16. Maximum plan angle by degree of failure for U,V-weirs.

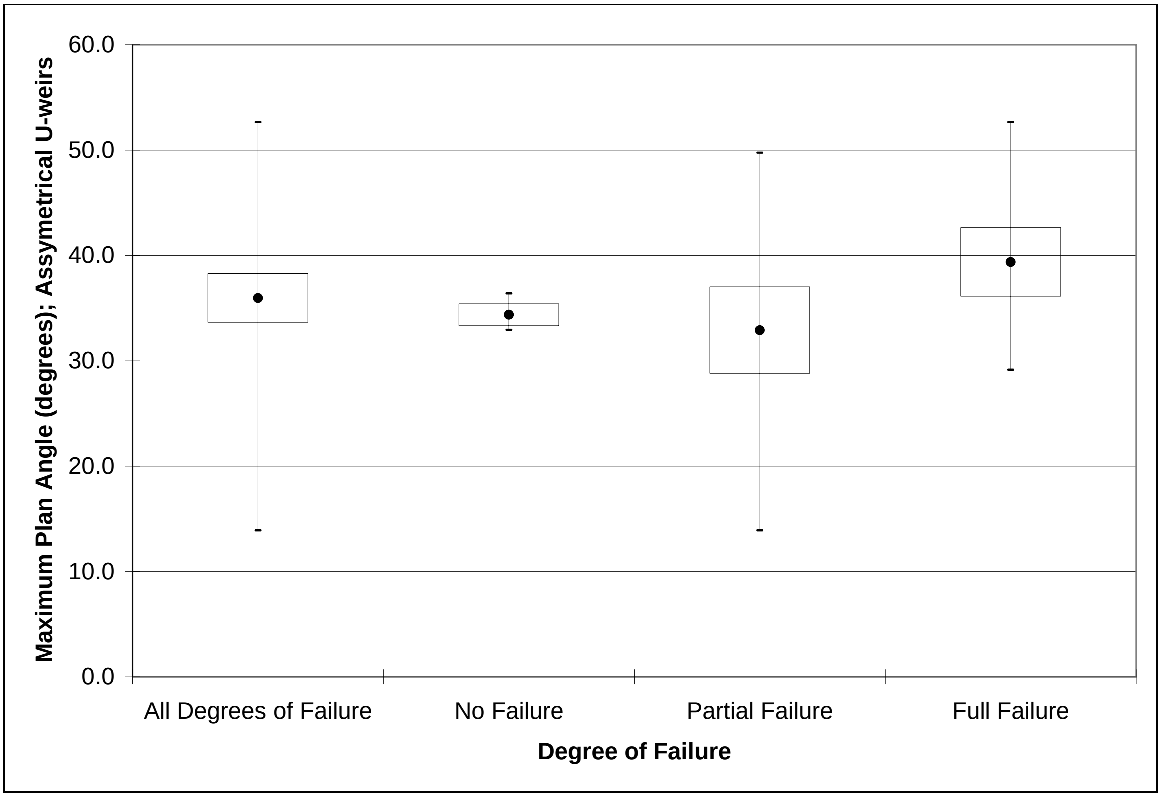

Figure 17. Maximum plan angle by degree of failure for asymmetrical U-weirs.

Maximum structure arm angle and maximum plan angle were significantly correlated (0.55) in a positive direction (i.e., the greater the plan angle, the greater the structure arm angle). However, the measurements can differ substantially because the bank lines are not always parallel to each other at the location of the structures, and the structure throats are not always perpendicular to the banks. Max arm angle relationships were less clearly defined than maximum plan angle relationships. Based on the average measured maximum arm angles, U,V-weirs with angles exceeding 45 degrees (Figure 18)and asymmetrical U-weirs with angles exceeding 60 degrees (Figure 19) had greater rates of complete failure than structures with smaller angles. An increasing relationship between the degree of failure and the maximum arm angle also exists for structures failing due to general bank migration/flanking (Figure 20). This can be explained by the fact that structures with at least one high arm angle have a high likelihood of being on a bend and experiencing migration.

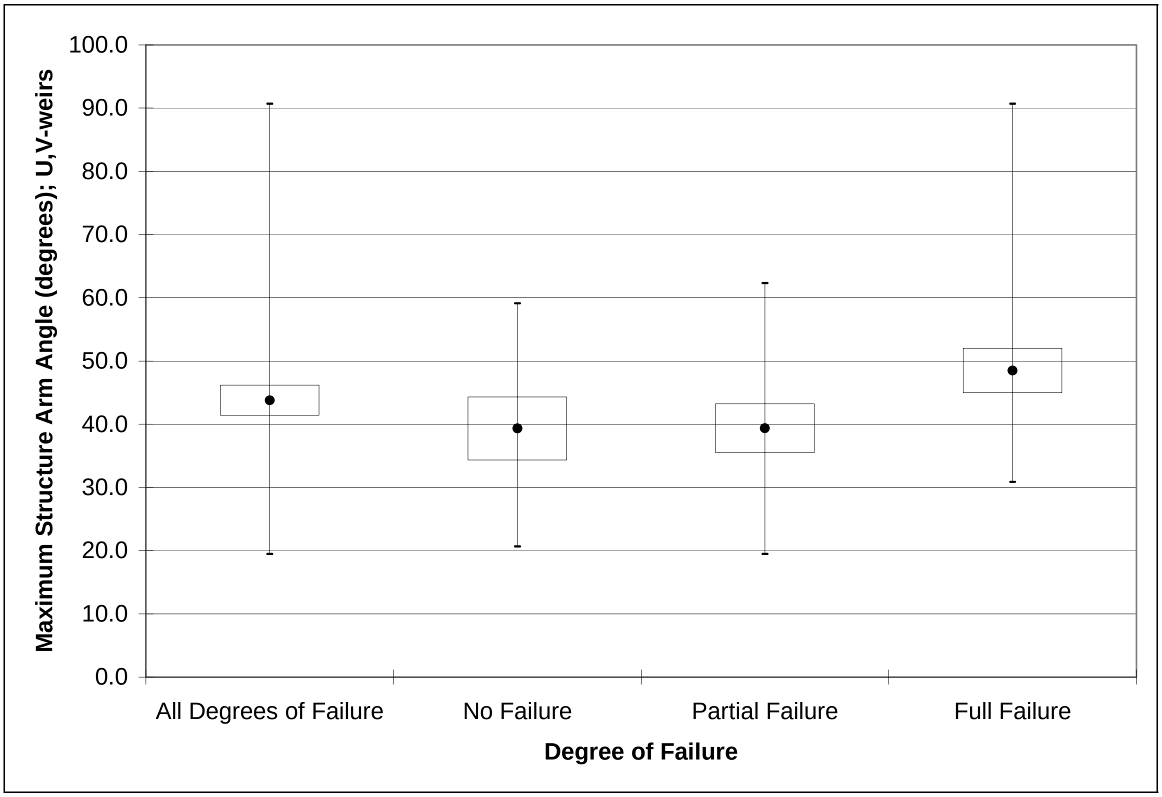

Figure 18. Maximum structure arm angle by degree of failure for U,V-weirs.

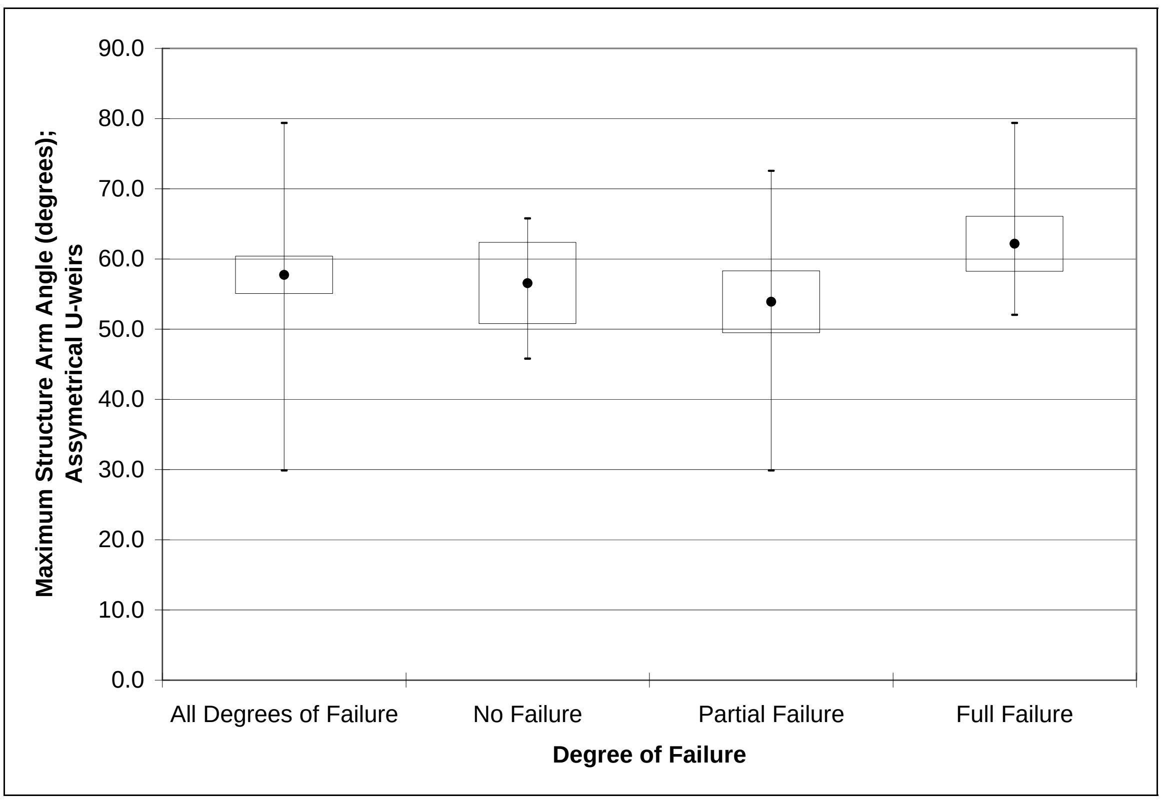

Figure 19. Maximum structure arm angle by degree of failure for asymmetrical U-weirs.

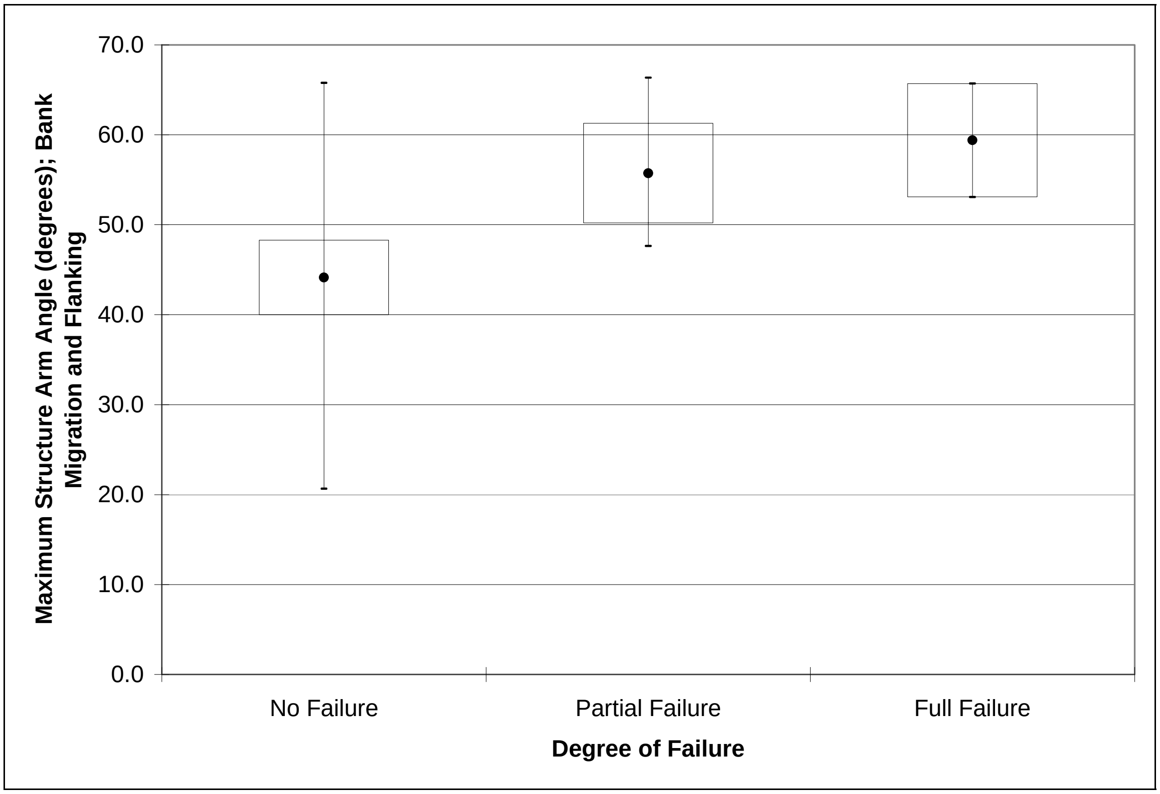

Figure 20. Maximum structure arm angle by degree of failure for structures failing by general bank migration and flanking.

The last angle in this category, open angle, shows an increasing relationship with degree of failure for all structures combined (Figure 21), U-V weirs (Figure 22), and A-weirs. For asymmetric U-weirs, full failures tend to have the highest open angle, but the measured values for the partial failures convolute a clear increasing trend (Figure 23). Structures having partially or fully failed as a result of the growth of scour pool and slumping of the constituent rocks illustrated an increasing relationship between open angle and degree of failure (Figure 24). On average, structures with an open angle greater than 60 degrees are more likely to either partially of fully fail.

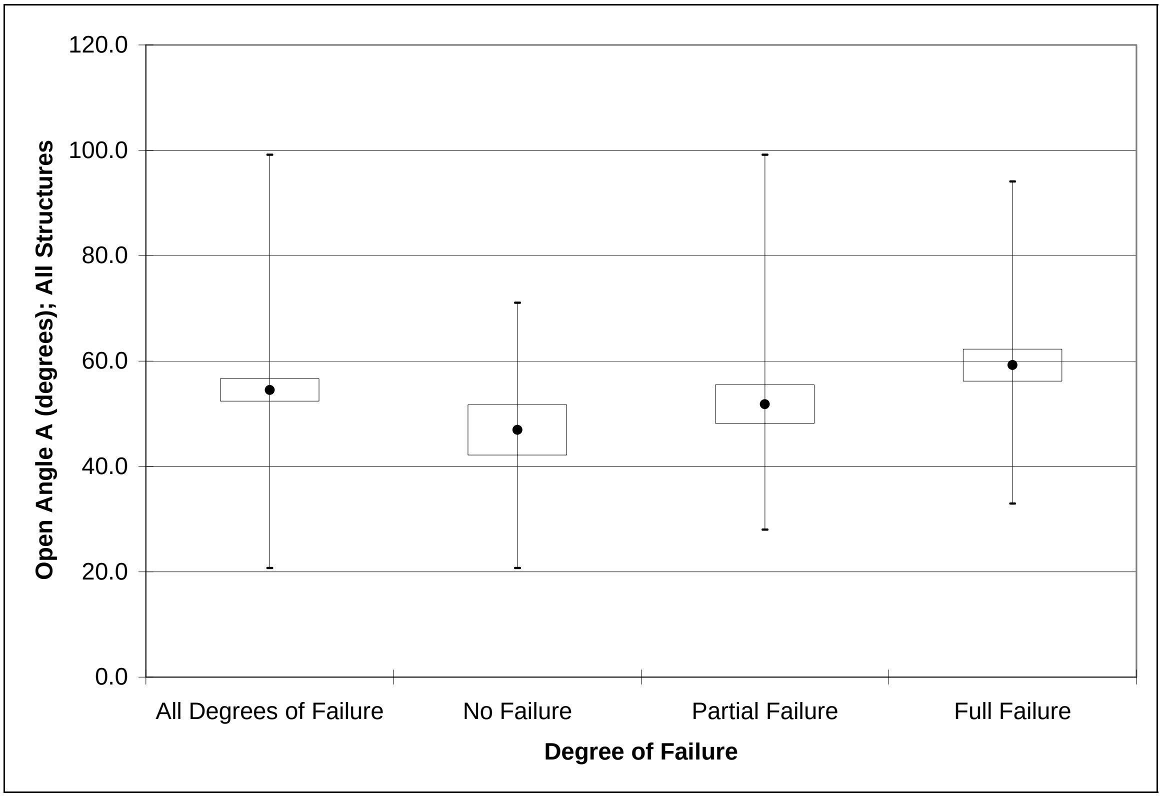

Figure 21. Open angle for all structures by degree of failure.

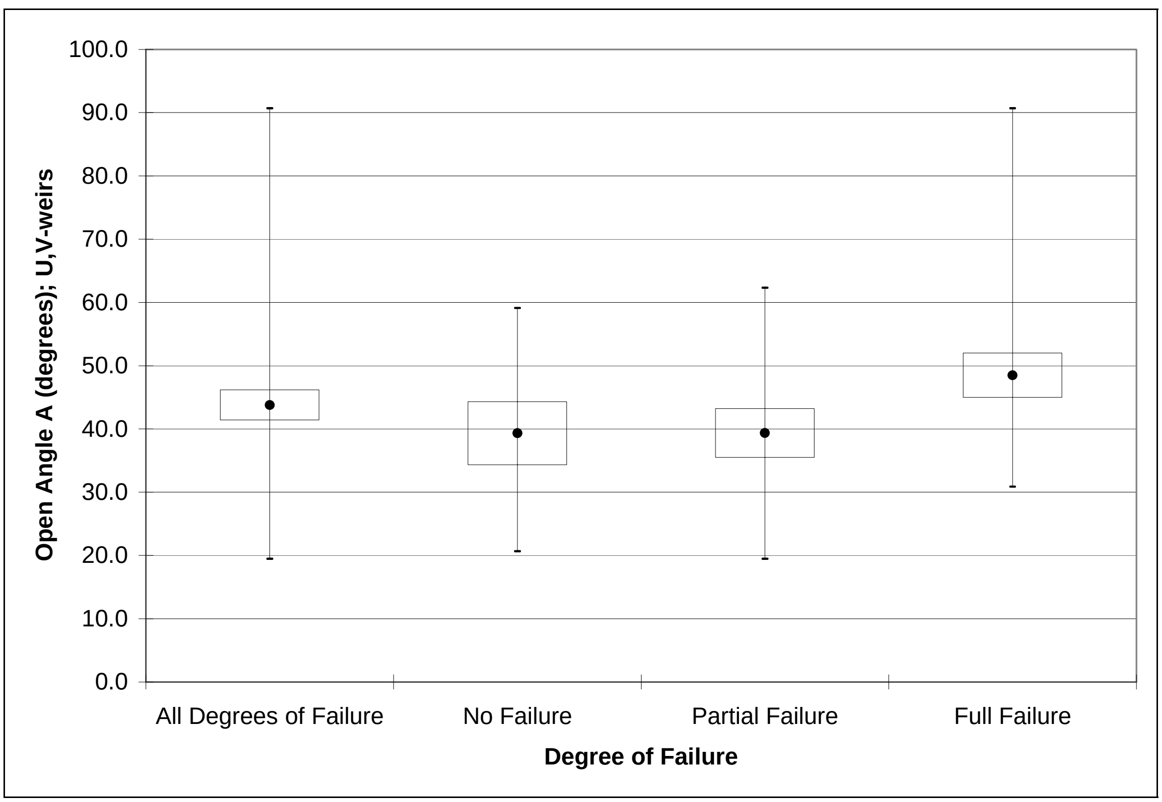

Figure 22. Open angle for U,V-weirs by degree of failure.

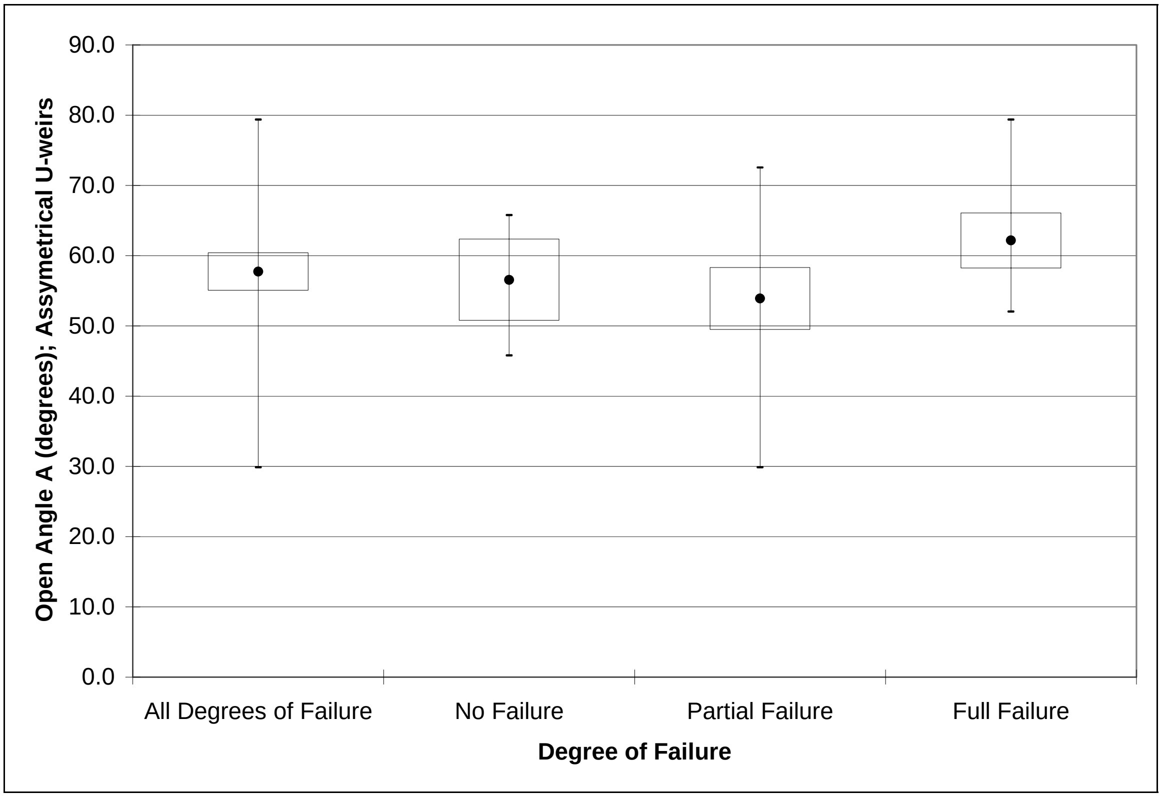

Figure 23. Open angle by degree of failure for asymmetrical U-weirs.

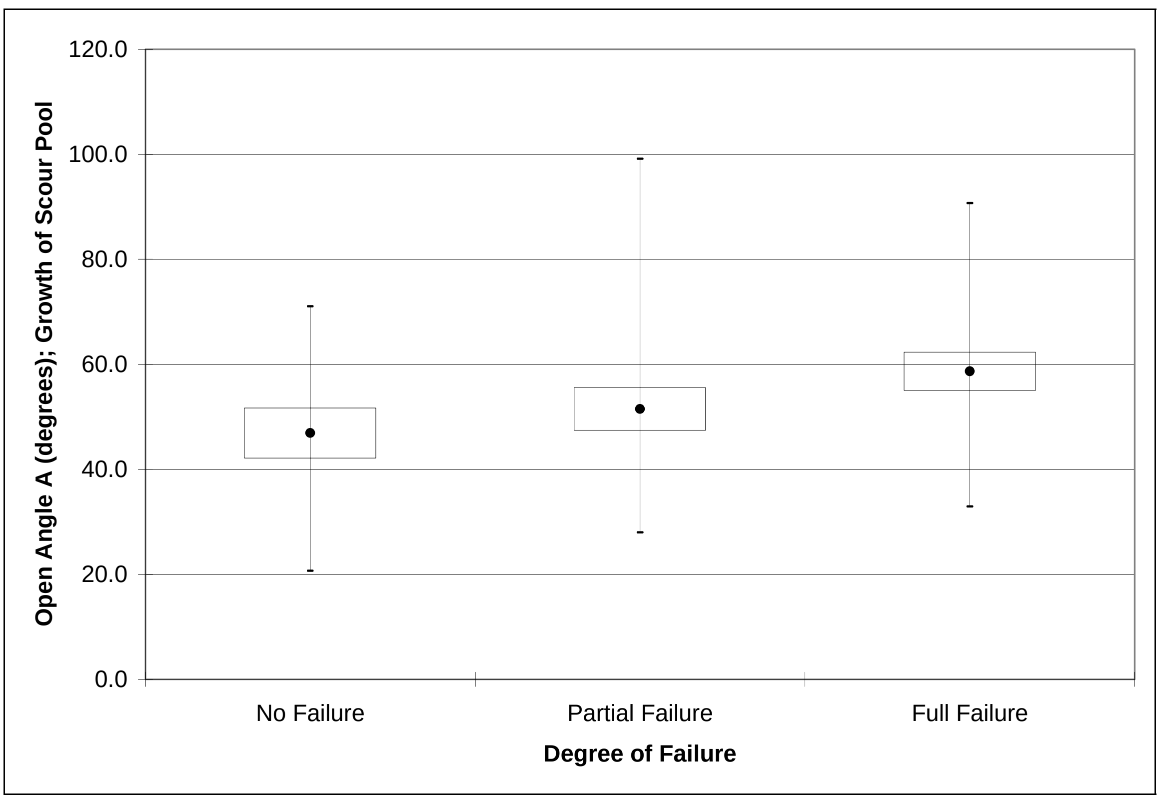

Figure 24. Open angle be degree of failure for structures failing by scour pool growth.

3.2.5. Arm Profile Slope

Both the maximum and minimum arm slopes were determined for each structure. An initial hypothesis related to arm profile slope was that the greater the arm slope, the more rapid the increase in head drop over the arm from the throat to the bank. This, in turn, would result in a greater chance for development of a deep scour pool along the structure arm and increase the potential for slumping of the footer rocks. However, no strong relationship was discernible between the maximum arm profile slope and the degree of failure. In evaluating all structures, no clear relationship between mean values was present, but the ranges in slopes increased from no failure to full failure. U,V-weirs that partially or fully failed had higher average slopes (8-10 percent) than the no failures (~7 percent). For structures that failed primarily or secondarily by growth of scour pool, a slight increasing trend was apparent with maximum arm profile slope (Figure 25).

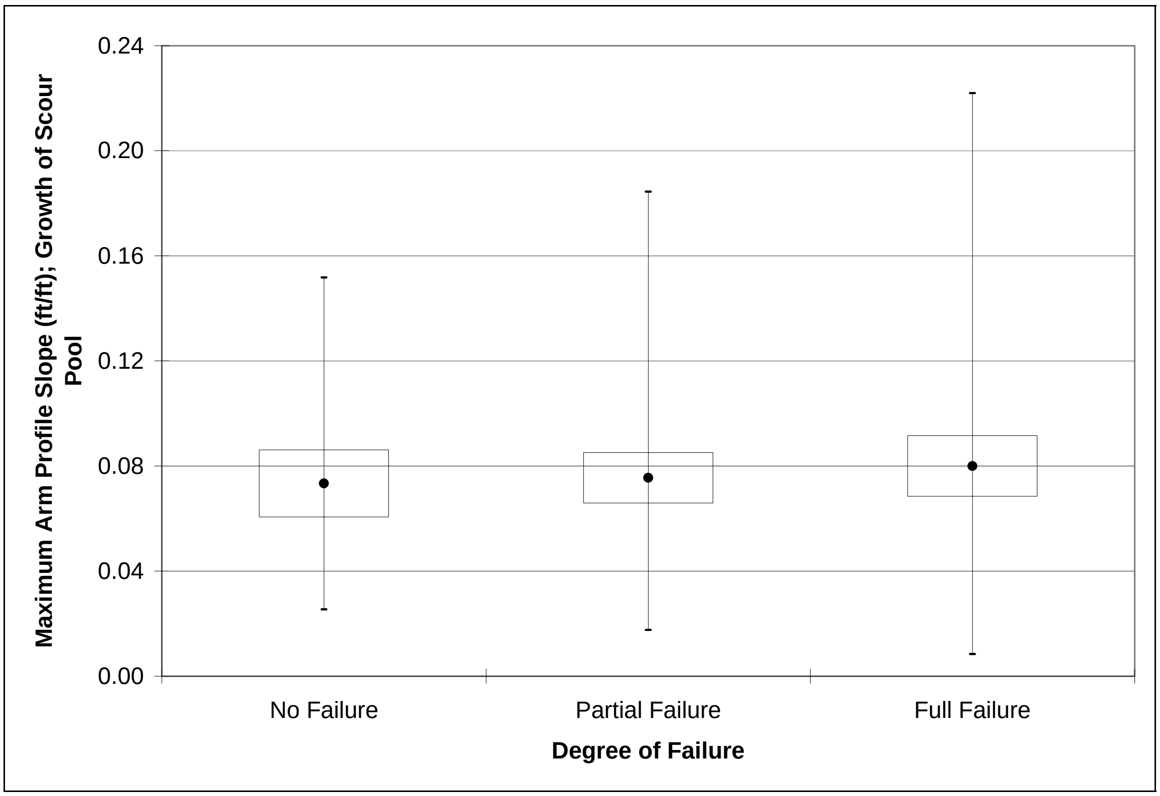

Figure 25. Maximum arm profile slope for structures failing primarily or secondarily by scour pool growth.

3.2.6. Distance Between Structures

Proximity of structures to the next upstream or downstream structure showed a trend with the degree of failure for U,V-weirs. U,V-weirs with closer proximity to other structures (regardless of type), in either the upstream (Figure 26) or downstream (Figure 27) direction, were less likely to fail than widely spaced structures. However, the asymmetrical U-weirs, all of which were located on the three forks of the Little Snake River, displayed the opposite trend with structure spacing. Figure 28 presents the spacing in river widths to the next upstream structure for asymmetrical U-weirs, and Figure 29 is likewise for the downstream direction. Meyer (2007) observed pool volume loss along the Little Snake due to closely-spaced structures in the form of (1) sedimentation upstream of a structure filling in the pool of the closest upstream structure or (2) a structure located in the middle of the pool of the closest upstream structure. At some point, structures can be spaced so closely that the scour pool of the upstream structure may be hindering the stability of the next downstream structure.

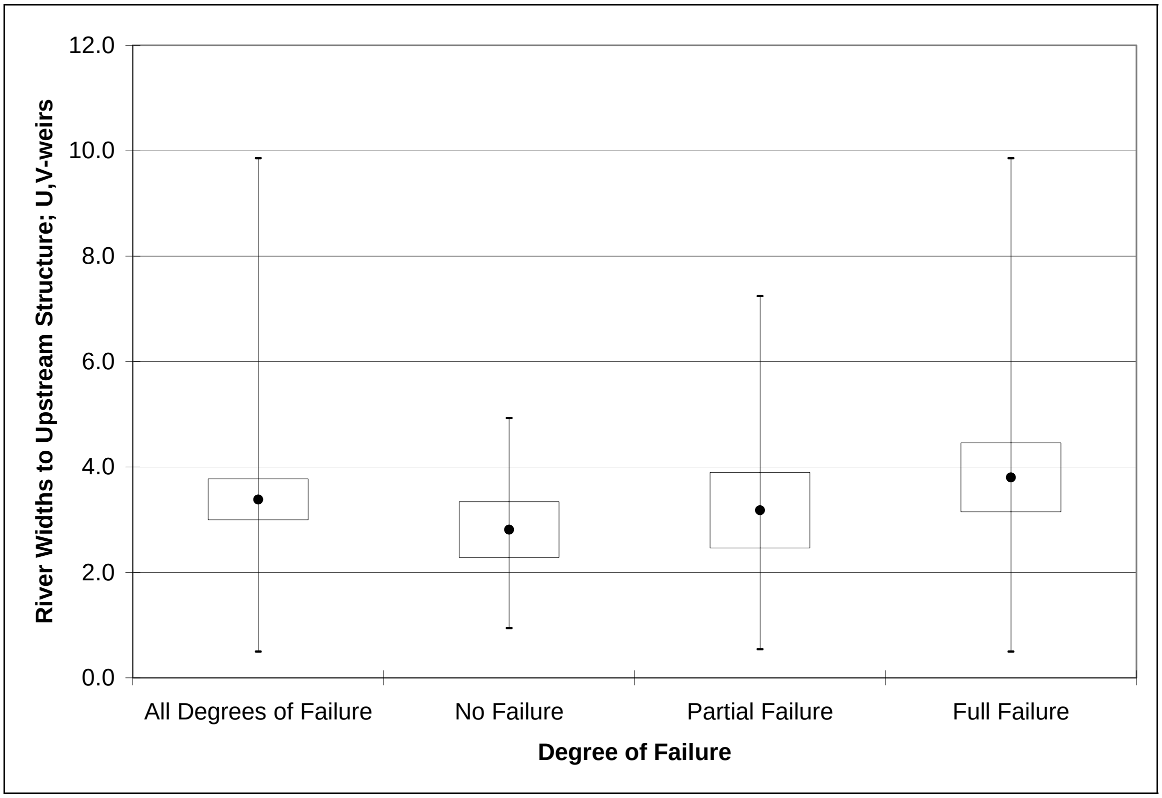

Figure 26. Spacing to next upstream structure; U,V-weirs.

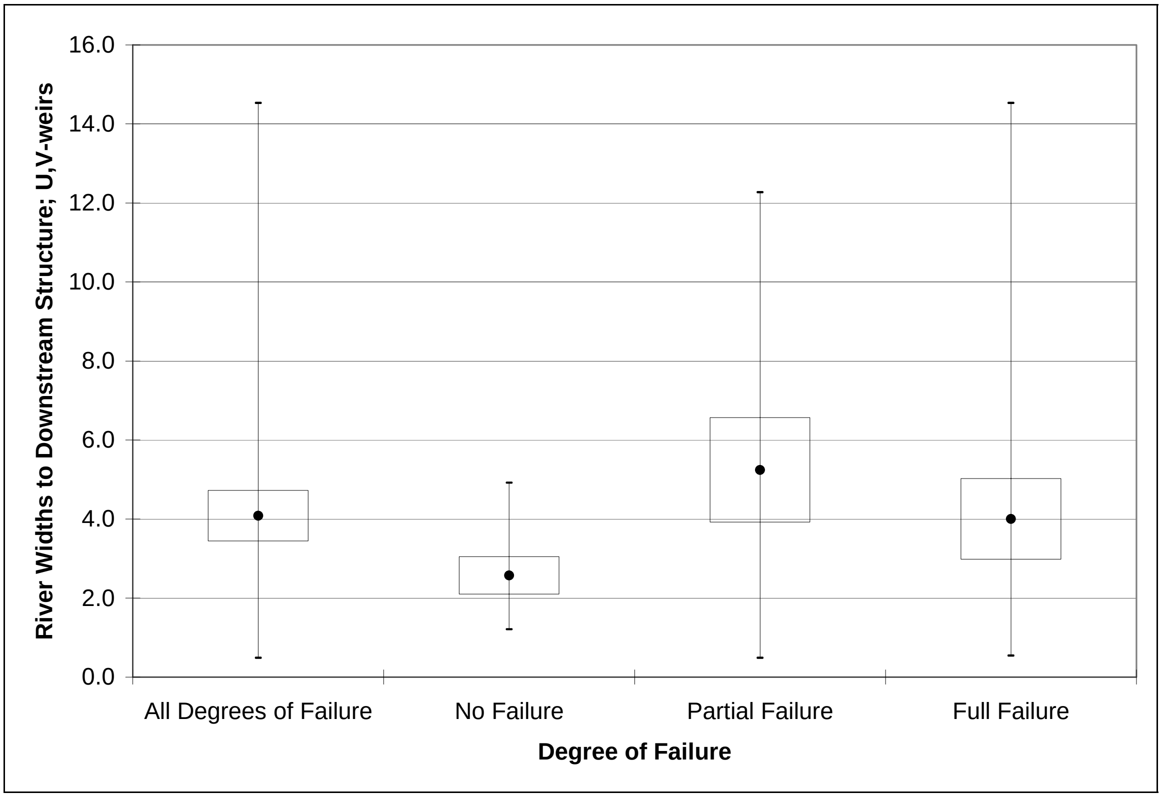

Figure 27. Spacing to next downstream structure; U,V-weirs.

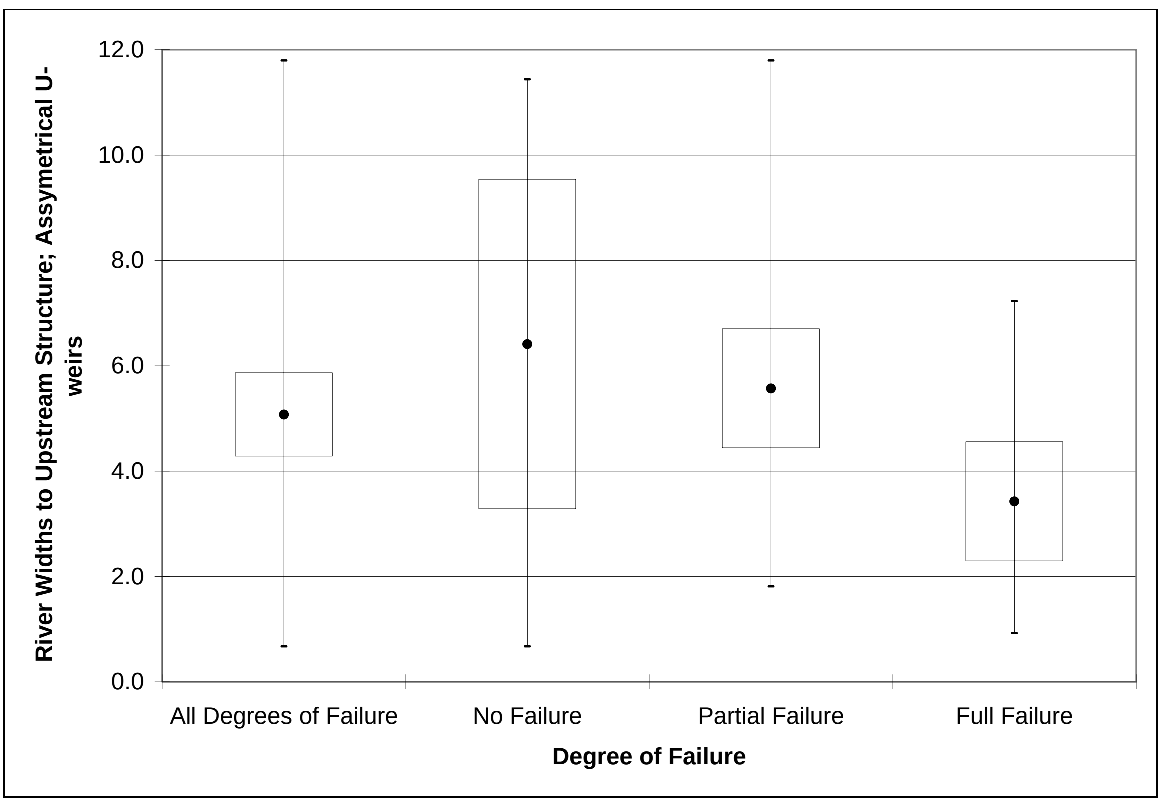

Figure 28. Spacing to next upstream structure; Asymmetrical U-weirs.

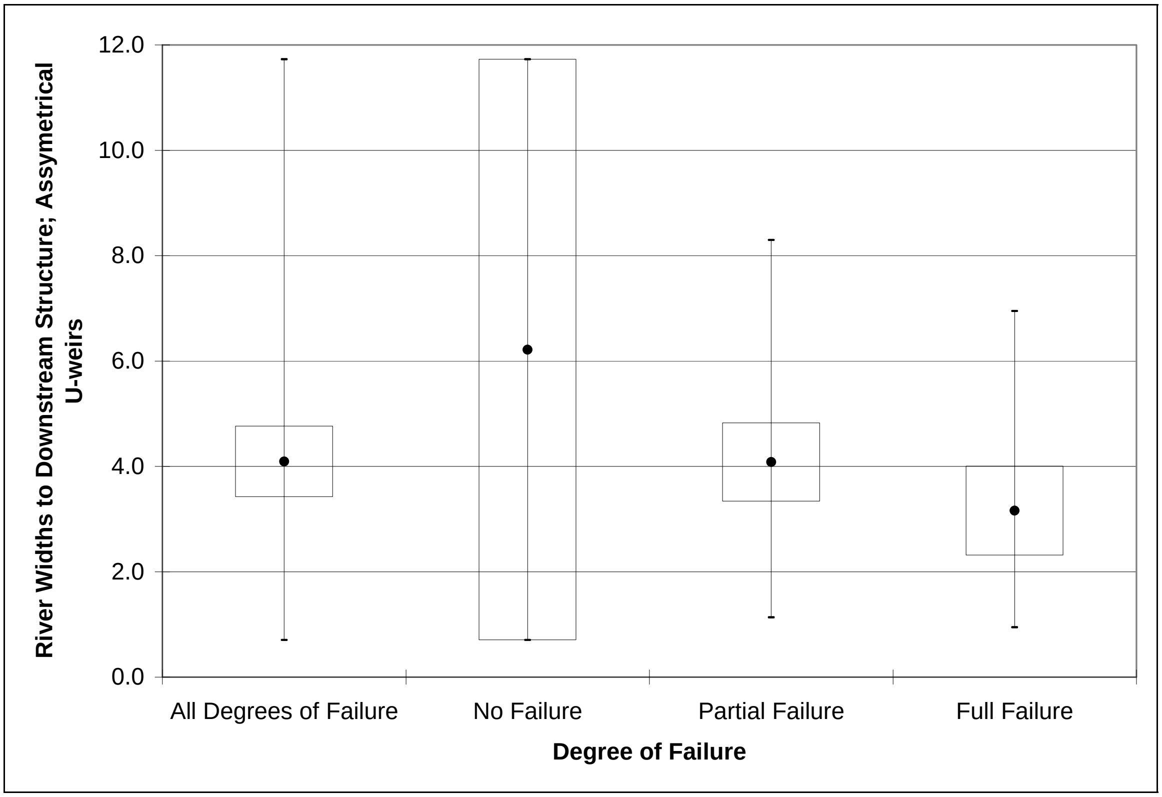

Figure 29. Spacing to next downstream structure; Asymmetrical U-weirs.

3.2.7. Throat Width

Another parameter that suggested some correlation with the degree of failure was throat width. To compare throat width across varying river conditions, throat width was non-dimensionalized through the use of the ratio of throat width to channel width. One channel width is assumed to be equal to the structure opening. Throat width relative to channel width shows trends with degree of failure for U,V-weirs and asymmetrical U-weirs. There is one no-failure instance for A-weirs but it cannot be used for trend observation as it is just a single point. Figure 30 presents throat width divided by channel width for U,V-weirs. This plot suggest that the more of the channel width blocked by the throat (as opposed to arms), the less likely the structure is to fail.

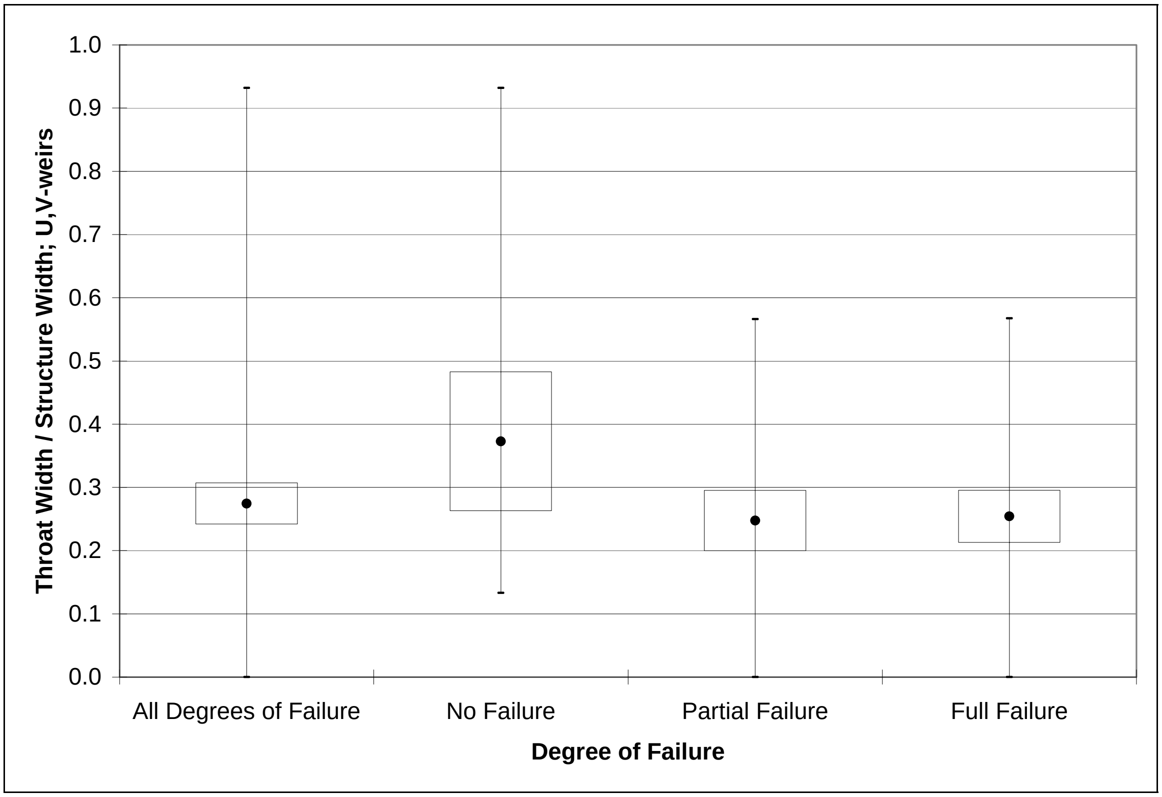

Figure 30. Throat width divided by structure width; U,V-weirs.

3.2.8. Thalweg Slope

The thalweg slope was flatter for structures that experienced no failures. Structures that experienced either partial or full failure averaged slopes near 1 percent while those experiencing no failure averaged slopes near 0.7 percent (Figure 31). An increasing trend was present between thalweg slope and degree of failure for structures failing by growth of scour pool (Figure 32). In general, the greater the bed slope, the higher the unit stream power is expected to be and therefore, the greater the scouring potential becomes over the structure.

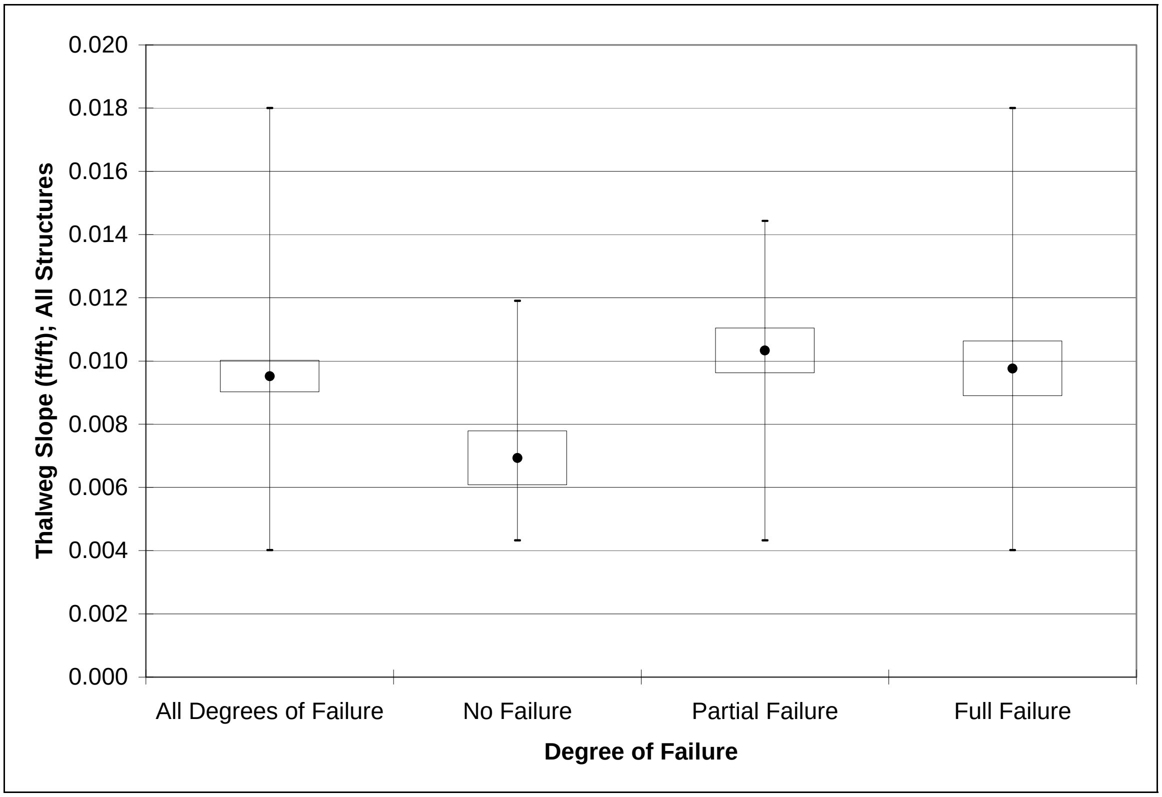

Figure 31. Thalweg slope for all river spanning rock structures.

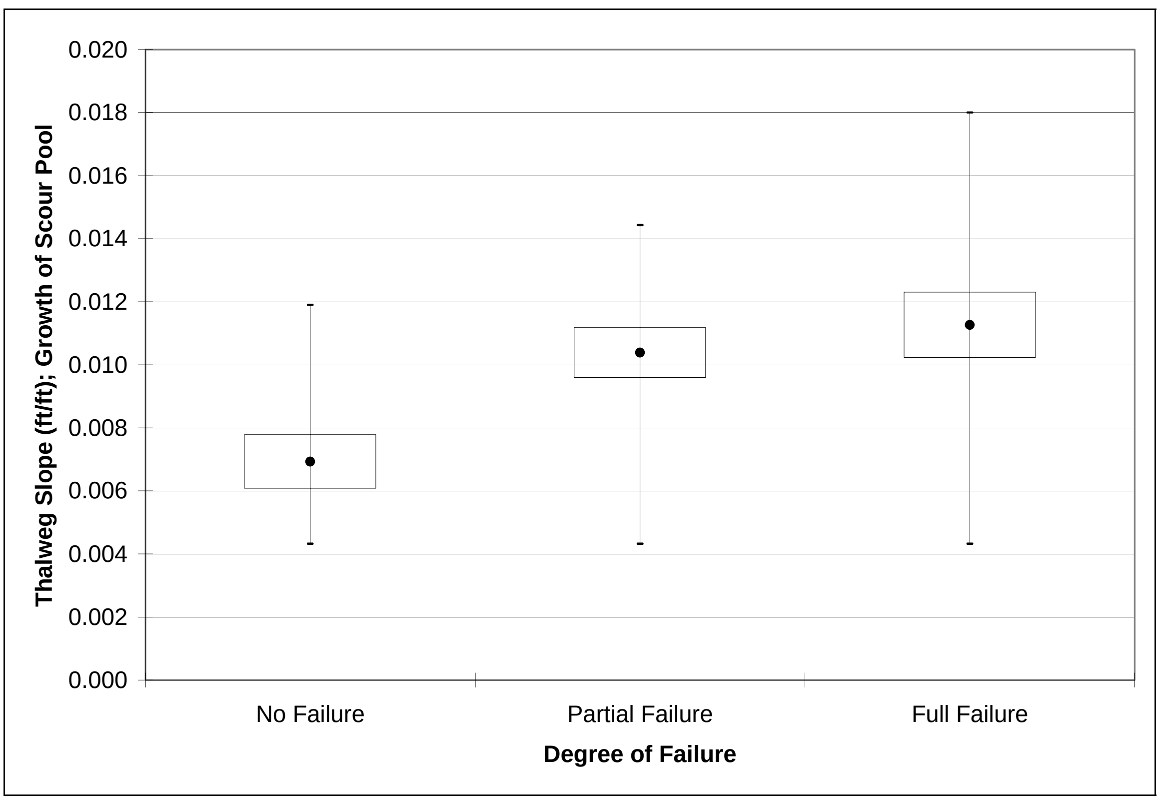

Figure 32. Thalweg slope for all structures that failed primarily or secondarily by scour pool growth.

3.2.9. Channel Width

Structure opening and structure width are indicators of the channel width and are significantly correlated (0.78). In general, the greater the structure opening and/or width, the more likely the structure is to experience failure. Structure widths greater than approximately 60 feet failed more frequently. U,V-weirs with structure openings greater than 45 feet tended to experience full failure (Figure 33). While this variable seems to indicate that wider channel may require more maintenance, it may also reflect the fact that the larger the channel, the lower down the structure is in the watershed and potentially the greater the sediment load. For structures failing either partially or fully by filling and burying, the average structure opening and structure width were substantially greater than those with no failure (Figure 34).

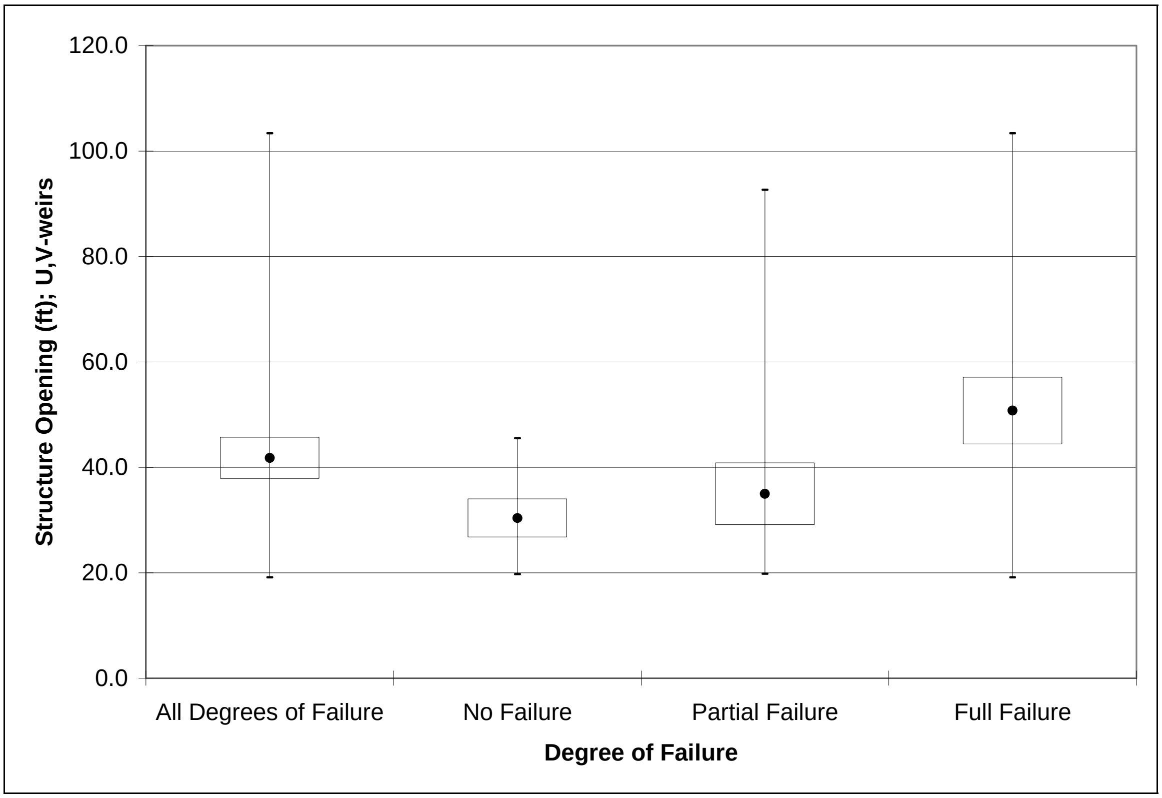

Figure 33. Structure opening by degree of failure for U,V-weirs

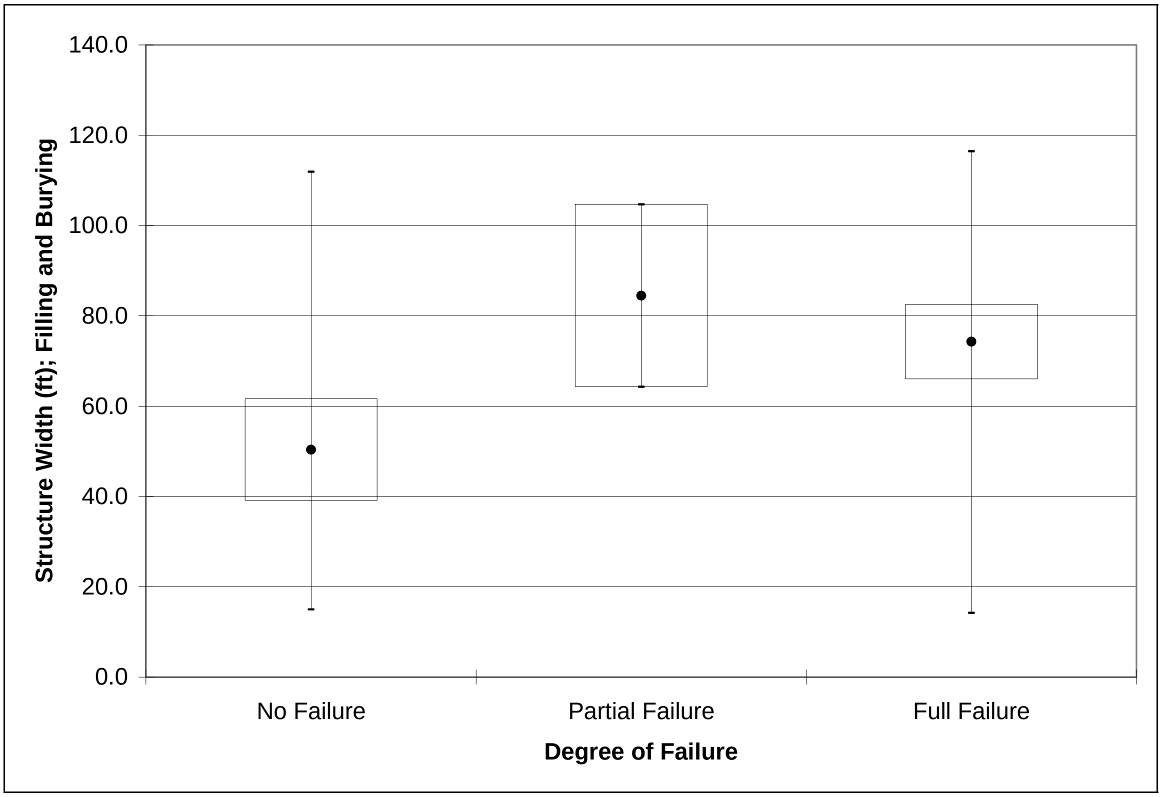

Figure 34. Structure width for structures failing primarily or secondarily by filling and burying.

3.2.10. Tie-In length

Structure tie-in length was a measure of how well the structure was connected to the bank to avoid potential flanking of the structure arms. The tie-in length was non-dimensionalized by structure width to compare equally across all structures under the assumption that wider structures located on larger streams should have longer tie-ins. Structures with a higher tie-in length to structure width ratios were less likely to fail than those with lower values (Figure 35). This supports the idea that bank tie-ins are an important design parameter. Structures that did not fail averaged a tie-in length of 11.5 feet or a ratio of 0.15. Structures that experienced partial or full failures due to general bank migration/flanking had significantly smaller values of tie-in length and tie-in length/structure width than no failures (Figure 36).

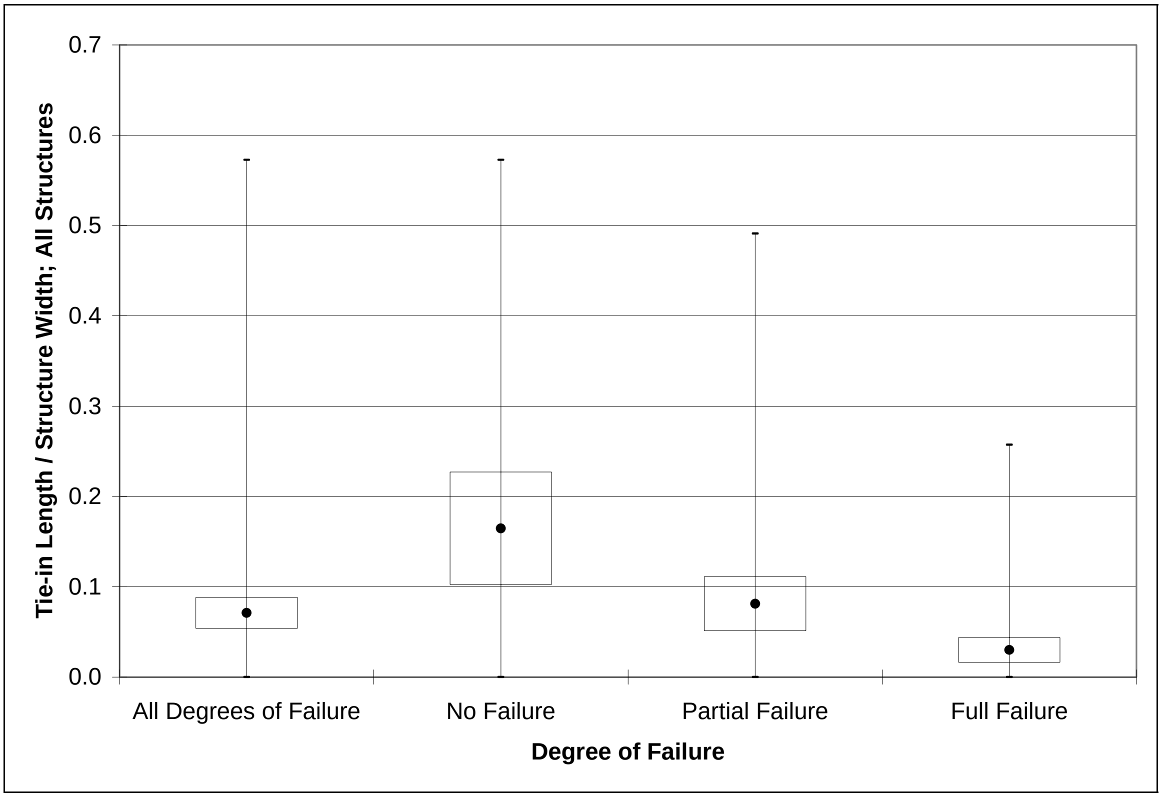

Figure 35. Ratio of tie-in length to structure width by degree of failure for all structures.

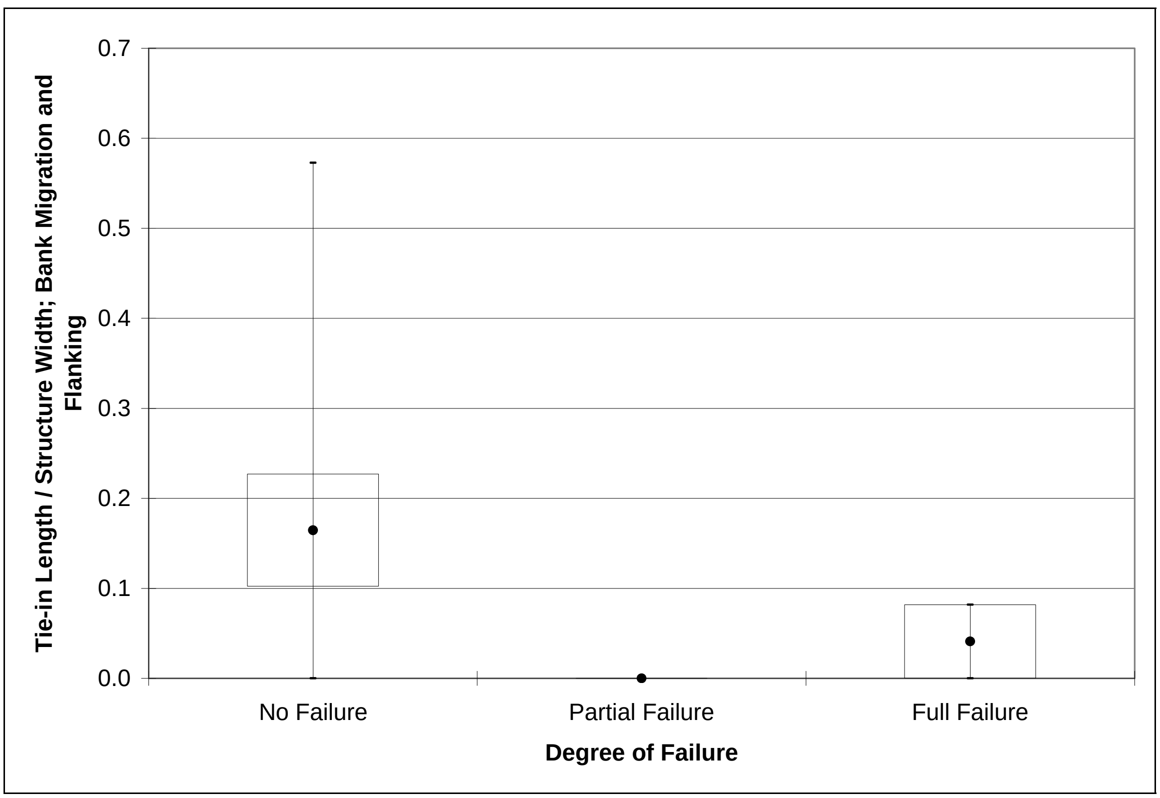

Figure 36. Ratio of tie-in length to structure width for structures primarily or secondarily failing by general bank migration and flanking.

3.2.11. Stream Power

Stream power influences river systems through impacts on channel form, pattern, and channel forming processes, such as sediment transport and channel migration (Knighton 1998). Total stream power per unit length of channel is defined by:

$$\Omega = \gamma QS$$

Where:

γ = the specific weight of water (N/m^3 or lb/ft^3)

Q = water discharge (m^3/s or ft^3/s)

S = energy gradient (m/m or ft/ft)

Because γ is a constant, total stream power was evaluated in this investigation as the product of a 2-yr discharge and surveyed bed slope (QS with units of ft^3/s). An attempt was made to capture differences in the ability of each site to perform geomorphic work through evaluating the total stream power at each site where the information was available. No clear trend in stream power was observable across all structures. However, U,V-weirs tended to experience less failure when stream power was smallest. Partial and full failures averaged stream power values of approximately 15 ft^3/s (Figure 37).

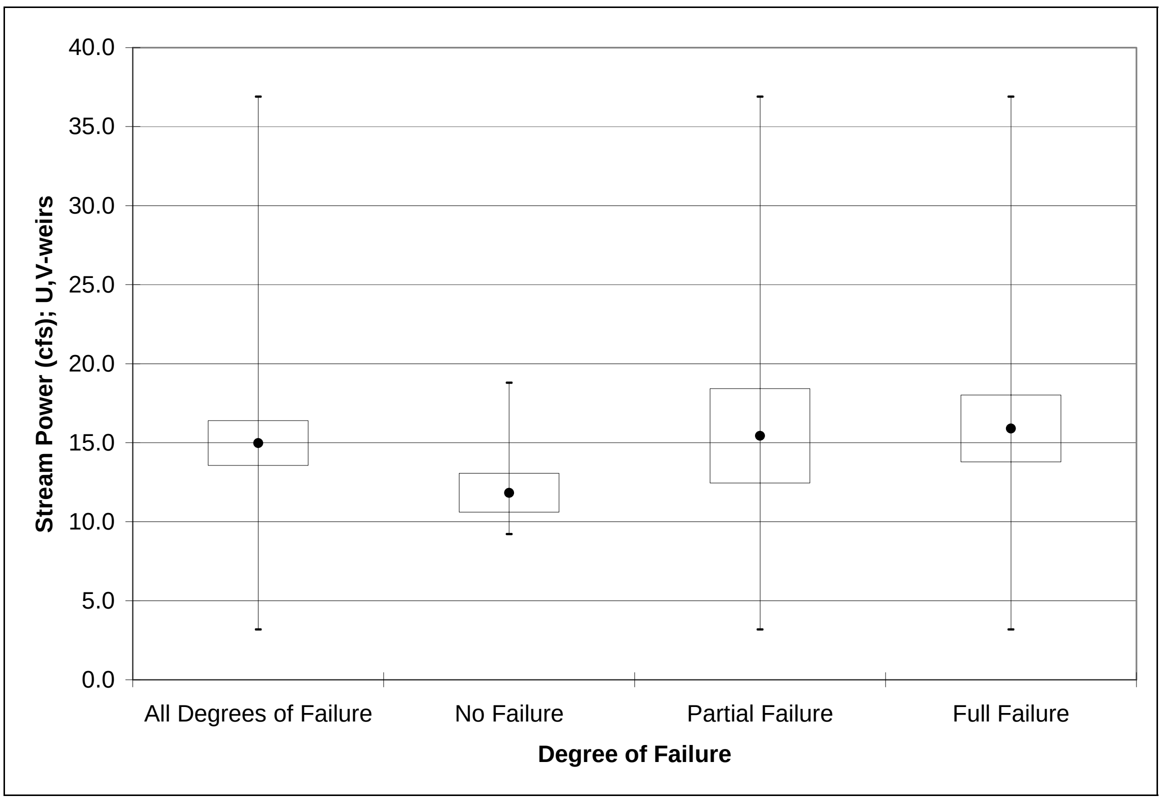

Figure 37. Stream power by degree of failure for U,V-weirs.

One limitation of this parameter is that the measured slopes were localized to a couple of channel widths upstream and downstream of the structure post-construction, which may vary substantially from the slope of the entire reach. A better indicator of each reach's stream power would require reach-averaged slopes and possibly a hydraulic model to capture reach-averaged friction slopes. Once a hydraulic model is developed, a local friction slope across the structure could aid in assessing the likelihood of rolling and sliding of the structure rock.

3.2.12. Structure Rock Size

Average rock size used in the construction of each structure was quantified for 16 of the sites that were visited, primarily those visited in October 2008. One hypothesis for the relationship between rock size and structure failure was that structures with smaller rock material would fail more frequently or to a higher degree than those with larger rock sizes. Based on the sites where this information was collected, only one of the sites was a no failure, 3 were partial failures, and 12 were complete failures. No clear relationship between degree of failure and rock size can be drawn from this data. The one no failure where rock size information was collected was located on the Entiat River and has not been subject to similar flows as the other structures. The lack of a strong trend supports the idea that mobilization of constituent rocks is not solely dependent on the size of the structure rocks. Rocks may be sized to withstand a given flow, and if a structure has been subject to more than that design flow, some mobilization may be expected. However, regardless of the design rock size, if a footer rock is undermined due to scouring flows, mobilization of the header and footer rocks is inevitable due to gravitational forces. Figure 38 illustrates the average structure rock material size, the recurrence interval of the high flows experienced by each structure, and the degree of failure for 16 structures visited in October 2008. The data show that complete failure may occur at relatively low flows, even for structures comprised of large (~1m diameter) rock material.

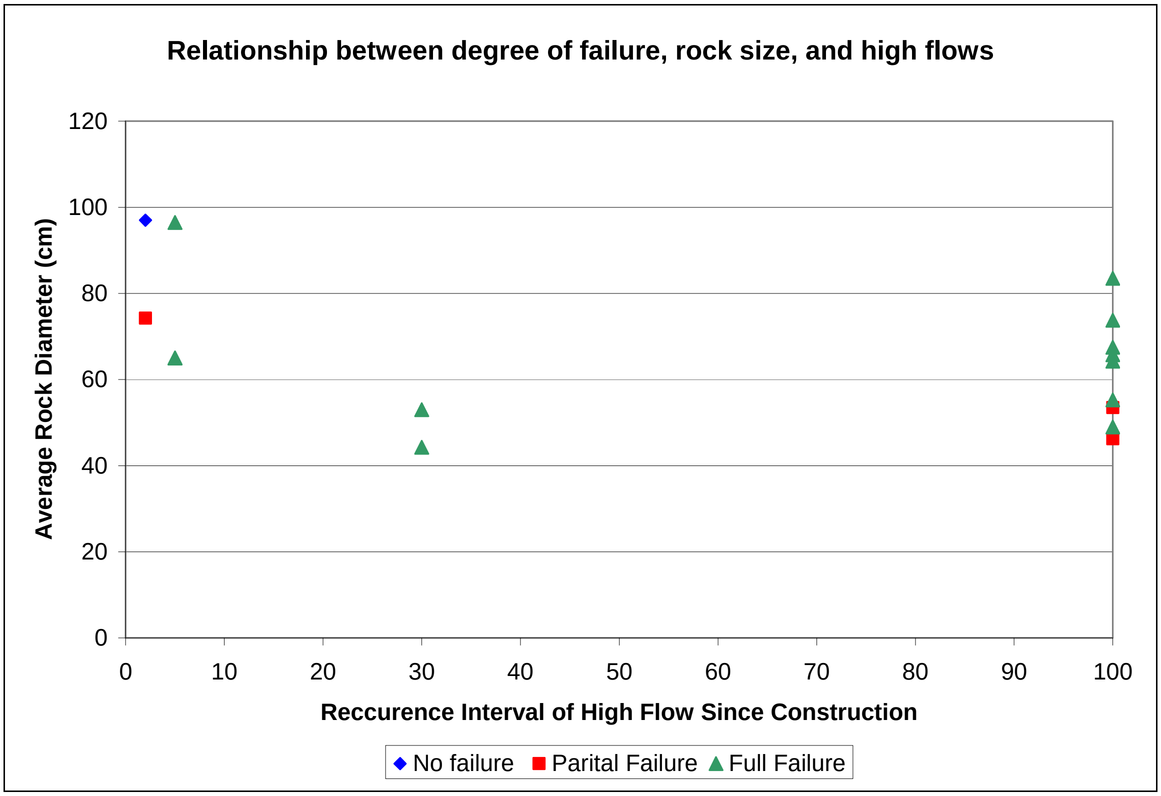

Figure 38. Graph showing the average rock size of each structure versus the recurrence interval of the highest flow experienced presented relative to the degree of failure.

3.2.13. Bed Material Size

Sediment samples were collected using traditional Wolman pebble count methods at ten of the sites to evaluate potential interactions between the bed material size and failure mechanisms. No relationship could be detected between the degree of failure and the size of the sediment based on the sample population. The bed material samples show that structure failures occur across all ranges of bed material sizes. Based on this small subset of structures, no patterns could be distinguished in relating failure mechanism to bed material size (Figure 39). Both failure by growth of the scour pool and by filling and burying occurred in reaches with smaller bed material (d_{50} ~ 50 to 60 mm) and larger bed material (d_{50}~100 to 150 mm). Finally, the samples were used to evaluate how sediment size varies locally just upstream and downstream of the structures. At 5 of the sites visited, sediment samples were collected upstream of the structure (typically 2 channel widths upstream) and then also downstream of the scour pool of the structure. Upstream and downstream pebble counts at a given site were either both taken in channel or both taken on an exposed bar for consistency and comparability. Results of this analysis indicate that sediment size does not tend to uniformly increase or decrease from upstream to downstream based on the 5 sites evaluated (Figure 40). In some cases, the gradations increased through the structures, while in others, a decrease was noted. An increase in the sample size may elucidate a more distinct trend in the future.

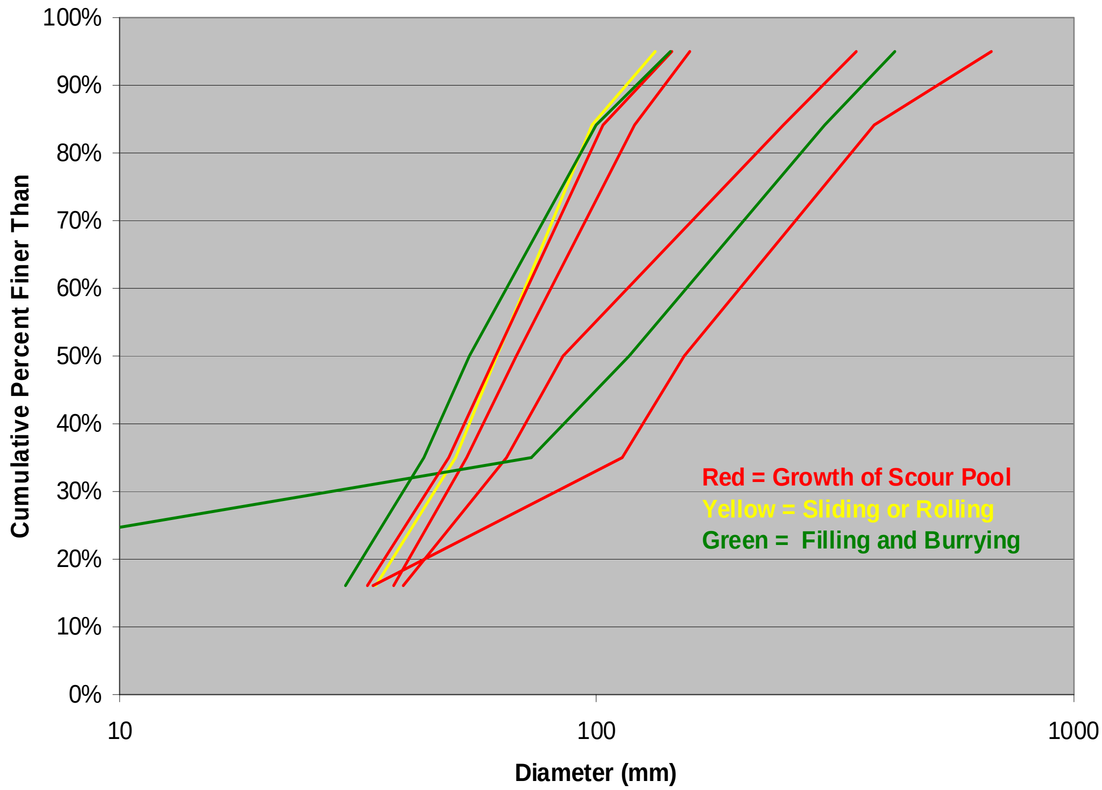

Figure 39. Sediment gradations by failure mechanism.

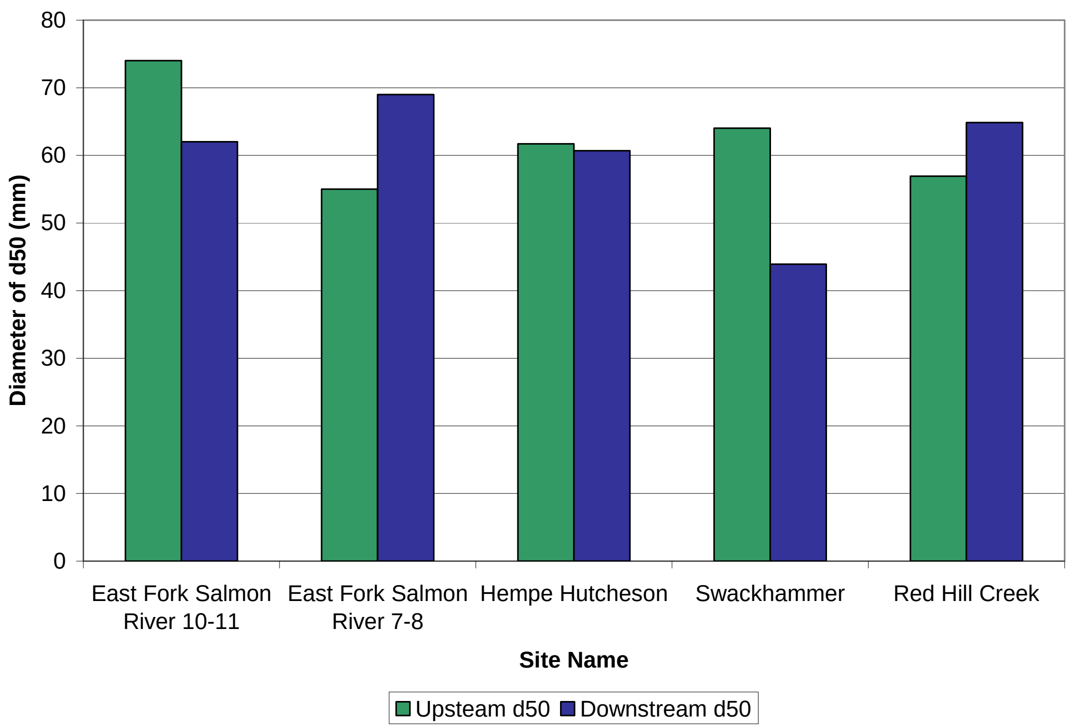

Figure 40. Median grain size for 5 sites where samples were collected upstream and downstream of the structures.

3.2.14. Summary of Observations

Observable trends were categorized as increasing or decreasing, depending on if the likelihood of failure was observed to increase with an increase or decrease of the parameter value. For instance, open angle was determined to have an increasing relationship with the degree of failure since the greater the value, the more likely the structure was to have experienced some degree of failure. Throat width divided by structure width was determined to have a decreasing relationship with the degree of failure since the lower the value, the more likely the structure was to have experienced some degree of failure. In cases where a clear upward or downward trend across all three degrees of failure was not present, a description was noted where the full failure value was higher or lower than both the partial and no failure values as denoted by "Full Fail, Higher" or "Full Fail, Lower"; and where the no fail value was higher or lower than both the partial and full failure, the notation "No Fail, Higher" or "No Fail, Lower" was used.

Table 8 presents descriptions of the relationships between structure parameters and degree of failure. These trends are strictly observations and are not described based on their statistical significance. Since all but one A-weir structures were observed to have partially or fully failed, the relationships presented only indicate whether there is a tendency for the structure parameter to have experienced a greater degree of failure with a variation in the parameter value. Incorporating fully functioning A-weirs that have never required maintenance could change the relationships as they are presented in this paper.

Table 8. Observations of trends by structure type for select parameters.

	All	By Structure, All Mechanisms			By Failure Mechanism, All Structures		
Variable	All Structures / All Failure	A-weir (only 1 NoFail)	U,V-weir	Assymetric U-weir	Growth of Scour Pool	Filling and Burying	Sliding or Rolling
2yr unit Discharge (ft^2/s)	Decreasing	None	(NoFail, Higher)	None	Decreasing	None	Decreasing
General Structure Spacing (river widths)	None	None	(NoFail, Lower)	Decreasing	(FullFail, Lower)	None	(NoFail, Higher)
Max Arm Length (ft)	None	Increasing	(FullFail, Higher)	None	None	None	(FullFail, Higher)
Max arm Length / Structure Width	None	Decreasing	(FullFail, Lower)	None	(FullFail, Lower)	None	None
Max Plan Angle	Increasing	None	(NoFail, Lower)	(FullFail, Higher)	Increasing	(NoFail, Lower)	(FullFail, Higher)
Max Scour (ft)	(FullFail, Higher)	None	None	None	(FullFail, Higher)	None	(FullFail, Higher)
Max Scour to Crest Offset (ft)	(FullFail, Higher)	Increasing	None	None	None	None	None
Max Structure arm angle (deg)	(FullFail, Higher)	Increasing	(FullFail, Higher)	(FullFail, Higher)	(FullFail, Higher)	None	None
Max Structure Arm Profile Slope (ft/ft)	None	None	None	(NoFail, Higher)	(FullFail, Higher)	None	None
Open angle A (deg)	Increasing	Increasing	Increasing	None	Increasing	None	None
Recurrence Interval of High Flow	Increasing	Increasing	Increasing	None	Increasing	(FullFail, Higher)	Increasing
River Widths to Downstream RSRS (ft)	None	None	None	None	None	None	None
River Widths to Downstream Structure (ft)	None	None	(NoFail, Lower)	Decreasing	None	None	None
River Widths to Upstream RSRS (ft)	(FullFail, Lower)	None	(NoFail, Lower)	(FullFail, Lower)	(FullFail, Lower)	None	Decreasing
River Widths to Upstream Structure (ft)	None	None	Increasing	Decreasing	(FullFail, Lower)	None	(FullFail, Lower)
RSRS Spacing (river widths)	(FullFail, Lower)	None	None	Decreasing	(FullFail, Lower)	None	Decreasing
Scour Depth / Structure Opening	(FullFail, Lower)	None	Decreasing	None	(FullFail, Lower)	(NoFail, Higher)	(NoFail, Higher)
Scour Depth / Structure Width	(FullFail, Lower)	None	Decreasing	None	(FullFail, Lower)	(NoFail, Higher)	Decreasing
Scour Offset / Structure Width	(FullFail, Lower)	Decreasing	Decreasing	None	Decreasing	None	(NoFail, Higher)
StreamPower (ft^3/s)	None	None	(NoFail, Lower)	None	None	None	(FullFail, Lower)
Structure Opening (ft)	(FullFail, Higher)	Increasing	Increasing	None	(FullFail, Higher)	(NoFail, Lower)	(FullFail, Higher)
Structure Width (ft)	(FullFail, Higher)	Increasing	(FullFail, Higher)	None	None	(NoFail, Lower)	(FullFail, Higher)
Thalweg Slope (ft/ft)	(NoFail,Lower)	None	(NoFail, Lower)	None	Increasing	(NoFail, Lower)	None
Throat width (ft)	(FullFail, Higher)	Increasing	None	None	(FullFail, Higher)	None	None
Throat Width / Structure Width	Decreasing	Decreasing	(NoFail, Higher)	None	Decreasing	(NoFail, Higher)	Decreasing
Tie in length (ft)	Decreasing	None	(NoFail, Higher)	(NoFail, Lower)	Decreasing	(NoFail, Higher)	(FullFail, Lower)
Tie in length / Structure Width	Decreasing	None	(NoFail, Higher)	(NoFail, Lower)	Decreasing	(NoFail, Higher)	(FullFail, Lower)

4. Discussion

4.1. Limitations of Results

The parameters included in this analysis did not address system wide processes that would need to be evaluated on a larger scale to evaluate potential impacts to structure stability. Lateral channel migration, sediment transport throughout the system, changes in slope through the system, and root strength present along the channel banks, among many other system-wide processes, may all play a role in how well a structure performs in a given location. One substantial finding of this analysis is that an understanding of the fluvial geomorphic processes must be gained prior to installing these structures to best recognize their potential for success in reaching a specific project objective and to realize the level of maintenance required at each site.

While hydraulic processes can partially explain the relationships between the parameters and failure, limitations are recognized by the sample size of each of the parameters. In some cases, results indicate counter-intuitive responses of a specific parameter and failure. This is most likely due to the fact that each parameter was evaluated based on its individual relationship with structural integrity. However, multiple parameters are interacting to produce a specific outcome of structural stability. By separating out relationships between the structure and individual parameters, the analysis is lacking the ability to incorporate all potential interactions between the predictor variables. Using the current data that were collected in the field across a relatively small sample size, these relationships represent a first attempt to correlate specific parameters with degrees of failure. A more robust analysis could be performed if the response variables were described with discrete quantitative values (i.e., percent mobility of constituent rocks) as opposed to categories of degree of failure or failure mechanisms. Based on field findings, quantification of each of the response variables requires additional data. For example, to quantify failure through growth of the scour pool and subsequent slumping of the header and footer rocks, an appropriate response variable would be depth of the scour pool relative to the depth of the footer rocks (depth of structure foundation). Unfortunately, the depth of the footer rocks was not a parameter that could be easily collected during site visits and could not be consistently interpreted from a limited amount of design information available at the sites. Therefore, multiple regression analyses were not conducted on this set of data, but may be further evaluated in the future with an increase in the database size and consistency in parameters collected at each site.

Data Uncertainties

Multiple elements create uncertainty in the results presented. One of the primary issues identified during the analysis is the subjectivity involved in digitizing the structures. Ideally, the centerline along the structure rocks could be consistently captured from the survey data. However, the variability in rock shape and rock placement makes digitizing an in-exact process. For example, East Fork Salmon structure 7-8 was visited and surveyed in 2005 and 2008. The East Fork Salmon River structure 7-8 survey data and digitized header rocks from 2005 and 2008 are shown in Figure 41 and Figure 42, respectively. The surveyed rocks were not always located in clear straight lines. Structure headers were digitized using the best fit between the survey points.

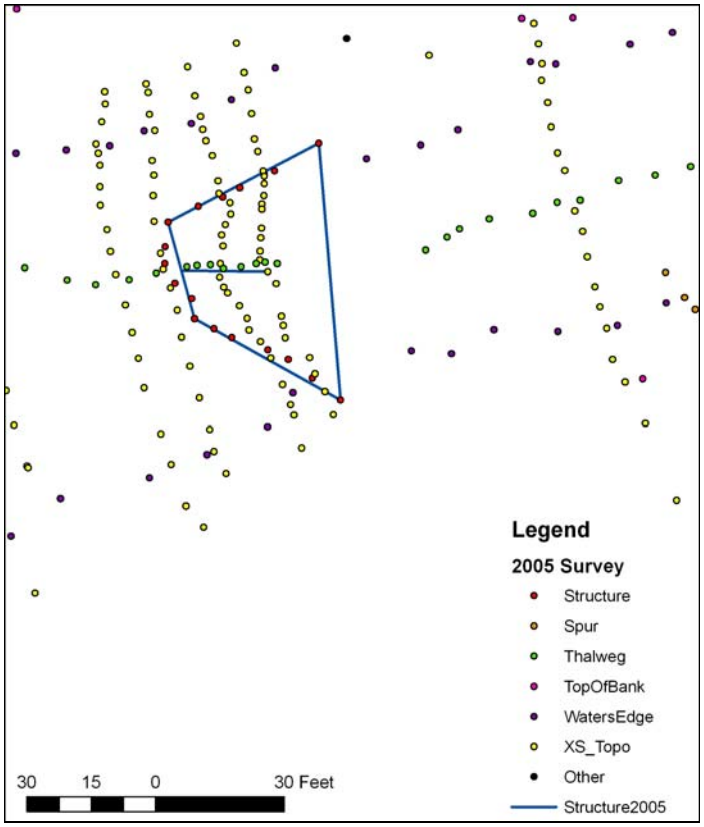

Figure 41. East Fork Salmon Structure 7-8 2005 survey data and digitized structure lines.

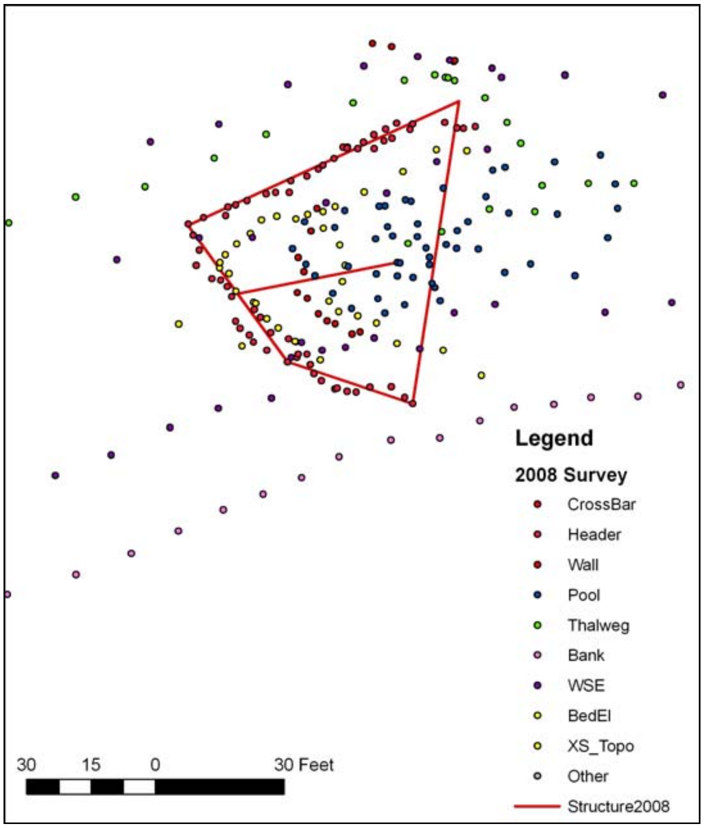

Figure 42. East Fork Salmon Structure 7-8 2008 survey data and digitized structure lines.

Another point of uncertainty is that the survey represents the structure configuration at a single point in time. The parameters measured from the survey data may or may not be associated with the parameters that caused the structure to fail. Figure 43 overlays the digitized header rocks from 2005 and 2008. This comparison shows that the structures are not static and confirms the uncertainty with using structure data from a single time. In this case, the structure experienced the largest discharge (2,500 ft^3/sec) since construction in May 2006, which occurred between the two survey dates. In addition, the structure has been altered by local landowners to maintain their diversion capacity. For this example, the decision as to whether the 2005 or 2008 survey data should be used for the quantitative analysis was difficult. The 2008 survey data was included because it represented the structure after a major high flow event (the event of May 2006, was estimated to be a 30-year flood).

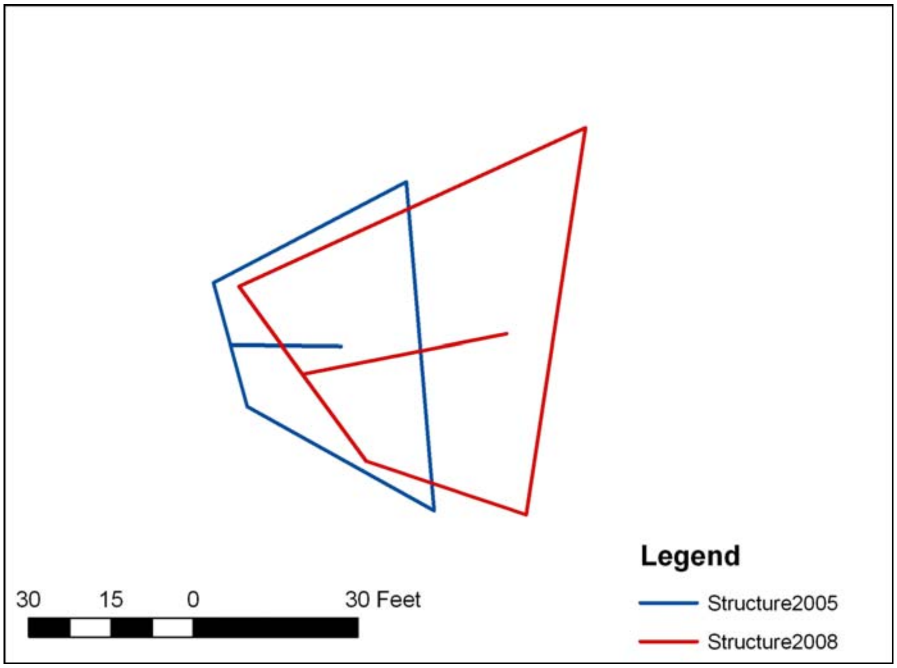

Figure 43. Comparison of East Fork Salmon Structure 7-8 digitized structure lines from 2005 and 2008.

The example of the East Fork Salmon Structure 7-8 shows that the rock weirs are dynamic. However, not all weirs change to this degree, which provides validity for conducting the quantitative analysis. The U-weir at river mile (RM) 3.1 on the Entiat River was surveyed in 2005 and again in 2008. Figure 44 and Figure 45 present the survey data collected and the digitized lines in 2005 and in 2008, respectively. Figure 46 shows the overlap of the 2005 and 2008 digitization. The Entiat River experienced a 5-year flood in 2006 with a magnitude of 4,700 ft^3/s. In this comparison, although a large flood occurred between surveys, the structure geometry changed very little.

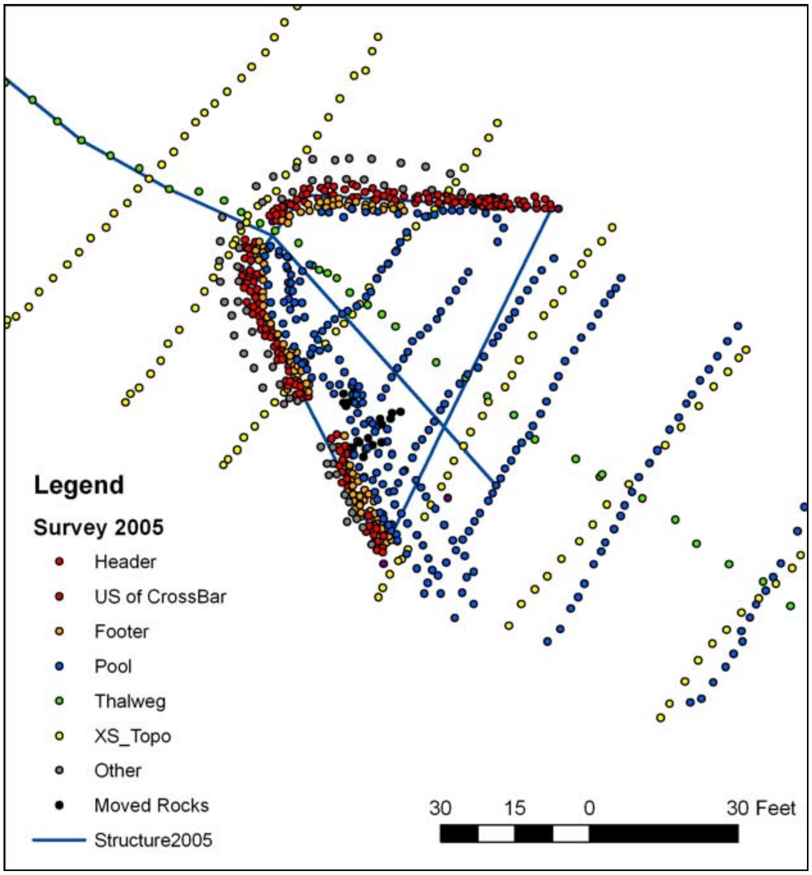

Figure 44. Entiat River structure at RM 3.1 2005 survey data and digitized structure lines.

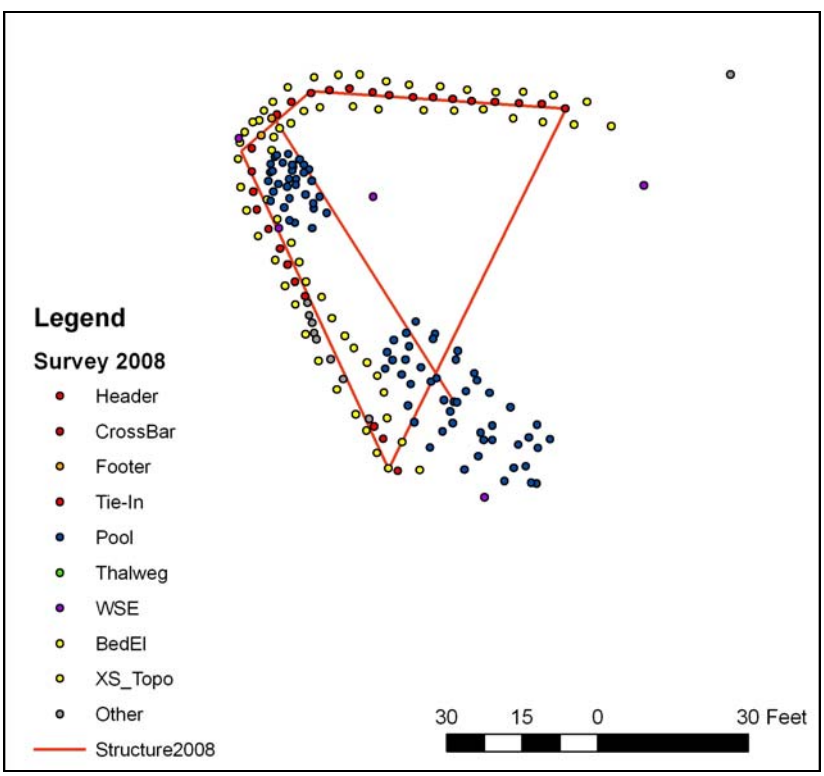

Figure 45. Entiat River structure at RM 3.1 2008 survey data and digitized structure lines.

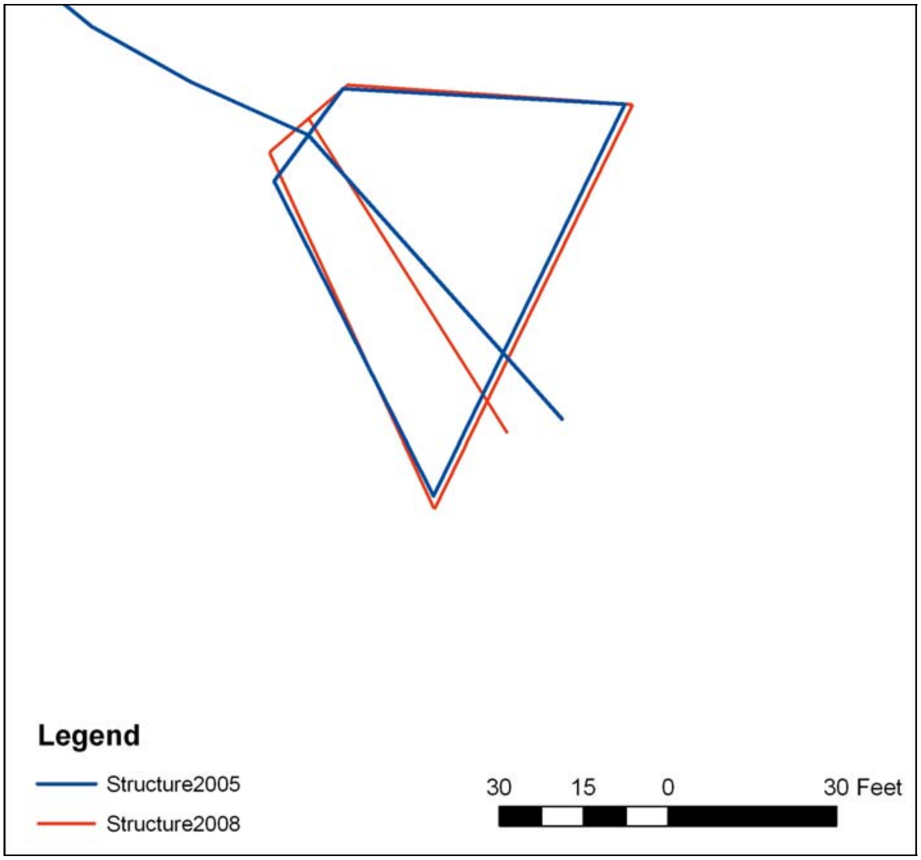

Figure 46. Comparison of Entiat River structure at RM 3.1 digitized structure lines from 2005 and 2008.

Subjectivity in digitizing and surveys limited to a single point in time when structures are dynamic does create uncertainties in the analysis. In addition, the small data set for certain types of structures or failure mechanisms limited the extent to which analysis conclusions could be made. However, for the information available, the results do illustrate the existence of relationships between certain structure parameters and rates of failures.

4.2. Links between structure parameters and fail/no fail

Several discernible relationships were identified between structure parameters and degree of failure. The most notable include the relationships with recurrence interval of high flows, throat width, planform angles, and the scour offset from structure. In addition, structure spacing, scour depth, and planform location are important variables that relate to structure performance, but clear relationships were difficult to discern in this investigation.

The recurrence interval of the highest flows that each structure has been subject to since construction is a good indication of the degree of structure failure. Based on the hydrologic analysis presented in Appendix B, structures having experienced fairly high flows (greater than 3-year discharge) had a high likelihood of some degree of failure.

From the qualitative analysis (Mooney et al, 2007), most of the failures resulted from the higher flows developing plunging pools along the structure arms and throat that undermined the integrity of the footer rocks, often leading to shifting of the crest rocks. This is contrary to common belief that the mobilization of rocks is a result of sliding and rolling due to undersized structure rocks. Since most streams are likely to be subject to recurrence intervals exceeding 3-year discharges over the design life of the structure, some degree of maintenance is likely to be needed if the purpose of the structure is to remain stable. For structures designed for pool habitat or fish passage improvements, mobilization of a few of the constituent rocks may create increased complexity or lead to more accessible passage through crevices for different life stages of species. In these cases, maintenance may not be as much of concern as designs for irrigation diversion. The remaining parameter findings may guide designs for reduced maintenance, but some level of structure maintenance should be expected in implementing relatively stable structures in dynamic systems.

In general, structures with wider throats relative to the channel width failed less frequently than those with smaller or no throats (V-weirs). This relationship was identified for all types of structures. From a physical processes standpoint, this relationship is likely due to less flow constriction to the center of the channel. During the identification of failure mechanisms, most structures failed as a result of increased scour pool growth and slumping of the header and footer rocks, most frequently along the structure arms where the energy gradient across the structure is the greatest. The elevation of the throat tends to be the lowest elevation across the entire crest of the structure. With a greater throat width, flow convergence to a narrower width in the channel is limited; the head loss over the center of the channel is smaller than when a large portion of flow is routed over the arms. This hydraulic process results in smaller scour depths and a geometry that spreads low flows across a greater width of the channel resulting in less failure.

Investigation of the planform angles suggests that the more perpendicular the structure arms are to flow, the more likely the structure is to experience some degree of failure. With respect to failure, the greater the open angle of the structure, the more likely the structure was to have experienced some degree of failure. The open angle of the structure was positively correlated to the maximum arm angle measured perpendicular to the structure throat (0.68) and the maximum plan angle with respect to the bank (0.78). The open angle is also significantly correlated with the maximum profile arm slope (+0.54) and negatively correlated to structure throat width (-0.42). Based on the correlations, a greater open angle results in a greater profile arm slope. A potential hypothesis for the relationship between the open angle and failure is that as the open angle of the structure increased, the throat of the structure tended to decrease in width, which results in a greater flow constriction with more flow directed through a narrower section of channel. Another potential reason is that with an increase in the profile angle of the arm, the head drop over the arm of the structure increases, which was observed to create plunging flows over the arms at higher flows and scour along the bottom of the footer rocks. In most of the designs, as the crest elevation increases from the throat of the structure to the top of bank, the footer elevations also increase. Therefore, the depth of the footer rocks below the bed is least where the greatest head loss over the structure occurs. This hydraulic process suggests the need for increased depth of the foundation along the structure arms (possibly an additional row of footer rocks), particularly where arm angles exceed 9 percent (as measured from surveyed data of U,V-weirs).

The relationships described for open angle were observed primarily for U,V-weirs and were not as prevalent for asymmetric U-weirs and A-weirs. For asymmetric U-weirs, one arm was typically much shorter than the other and did not always tie-in to as high an elevation as the opposite arm, and therefore, a smaller percentage of scouring flows are routed over a greater head drop during high flows. One A-weir was observed to be fully successful without mobilization of some of the constituent rocks; therefore, a true relationship between failure and design parameters of these structures was not discernible.

A non-dimensionalized method to evaluate the location of the maximum measured scour depth relative to the structure crest rocks proved to be related to the degree of structure failure. Although the scour depth alone could not be directly related to the degree of failure in this study, a hypothesis was developed to explain failure based on scour depth location: As the distance between the maximum measured scour depth location and the nearest crest rock along the arms, throat or cross bar (scour offset) increases, the likelihood that the structure fails due to growth of scour pool should decrease. This parameter was non-dimensionalized by structure width to account for differences in the size of each river. For example, a scour offset of 10 feet may mean that the maximum measured scour lies in the middle of a 30-foot wide channel or lies pretty close to a structure arm in a 100-foot wide channel. Based on the observations of this study, the closer the maximum scour depth is to the structure crest, the more likely the structure is to have experienced failure. These results further support the need for a foundation depth of sufficient depth around the entire structure to prevent undermining of the footer rocks. Although the idealized location for the development of a scour pool lies nearly half a channel width downstream from the structure throat and equidistant between the structure arms, field observations show that this is rarely the location of the maximum measured scour. Scour pools downstream from river spanning rock structures are often asymmetrical and form over time as a result of adjustment and settling of structure rocks, minor differences in rock size and shape, and often stochastically as a result of some alluvial input, such as a piece of woody debris. The maximum measured scour for many of the structures visited for this investigation was measured along the structure arms, where the head drop over the structure is greatest, the footer protection is least, and high velocities have the greatest potential to scour.

Structure spacing is an important parameter to consider for structure design. Structures that were more closely spaced tended to have greater success rates than those that were spaced farther apart. However, structures spaced too closely may result in increased potential for failure. Multiple structures in sequence tended to outperform individual structures. Structures in sequence offer a more stepped approach to energy dissipation than a single drop structure, which seems most appropriate when the objective of the project is irrigation diversion or fish passage. The spacing of structures will be highly dependent upon the goal of the project. When the project objective is to create a large pool volume for holding habitat, structures closely spaced in sequence may limit the maximum volume attainable. Asymmetrical U-weirs on the three forks of the Little Snake River often failed more frequently as the structure spacing decreased, which may result from the development of a scour pool hindering the stability of the subsequent downstream structure. Meyer (2007) observed pool volume loss along the Little Snake due to closely-spaced structures in the form of sedimentation upstream of a structure filling in the pool of the closest upstream structure or a structure located in the middle of the pool of the closest upstream structure. Structures in sequence should be spaced at least far enough apart such that the next downstream structure creates a submerged condition for the upstream structure at the design flow. This would protect the downstream structure from experiencing increased velocities and shear stresses that result from the plunging flows over the upstream structure. Computational hydraulic modeling would be necessary to determine the appropriate spacing for a sequence of structures since the hydraulics will be dependent upon the channel and structure geometries. For full formation of pools, spacing of structures should entail a geomorphic investigation to determine the average size and frequency of pools that the system can sustain.

Two parameters that were evaluated as part of this analysis but were difficult to interpret due to study limitations included scour depth and the structure location relative to lateral channel morphology.

Despite current knowledge of the importance of scour depth in predicting failure, this study was unable to capture a strong relationship between scour depth and failure. Since scour depth is such an important part of whether structures fail due to growth of scour pool and slumping of header and footer rock, preliminary hypotheses suggested that the scour depth would be correlated to failure. However, such a trend was not perceptible upon further investigation of the quantitative data. The inability of this study to identify a trend between scour depth and structure failure can be explained by several factors. First, to accurately quantify failure through geotechnical slumping, knowledge of the maximum scour depth relative to the foundation depth is essential. If the scour depth exceeds the foundation depth, this mechanism of failure would predominate. However, minimal data were available on the foundation depths of each structure in order to develop this ratio. In addition, the maximum scour depth just before a structure fails is almost impossible to measure given the field circumstances that would be required during data collection. Maximum scour depths determined in the field were typically measured post-failure and the previously scoured areas were filled with the slumped header and/or footer rocks at the time of survey; or the hydraulics had been modified following failure to the extent that the maximum depth of scour was not discernible. In some cases, the scour pool appeared to have partially filled in with incoming bed material, likely on the receding limb of a high flow event. The evaluation of maximum scour depths for structures experiencing some degree of failure is best ascertained in a physical laboratory or a numerical model where the development of the scour pool can be analyzed throughout the process of development. The point at which the scour exceeds the footer or foundation depth would be measured as the maximum scour depth attainable prior to the onset of structure instability or failure.

Determining a relationship between structure planform location and degree of failure was complicated by the study design. During field visits, it was hypothesized that the location of the structure relative to a meander bend was important in determining the structure success or failure. Structures placed at or anywhere along meander bends were expected to have higher rates of failure than those in straight or cross-over reaches of channel due to lateral channel migration and the inherent instability with placing a structure along an actively changing channel. In addition, during high flows, the greatest velocities occur along the outside of the meander bend, which coincides with the location of maximum head loss over the arms. The bend hydraulics result in the development of a large scour pool along the outside arm where foundation depths are generally shallow, ultimately leading to slumping of the footer and header rocks. Despite these known processes, a strong relationship between structure failure and bends could not be defined across all the structures visited. This may be in part related to the sample population. As mentioned in the results section of this report, over 80 percent of the structures visited had experienced mobilization of constituent rocks and 67 percent of structures observed were located on bends. A better representation of no failures and structures located in straight sections or crossings would improve the sample distribution to more appropriately compare failure with structure planform location. Another reason for the lack of a clear relationship may possibly be related to system interactions that can not be captured with geometric parameters of the structures alone. For instance, some meander bends may be relatively stable over longer time periods (e.g. 50 yrs) than others that migrate rapidly on a more frequent basis (e.g. 2 yrs). One notable trend associated with planform location was that all structures failing as a result of general bank migration/flanking were located on bends. This result supports the need for a preliminary analysis of historical channel position prior to determining the best location for the structure and also points toward the importance of structure tie-ins to the bank.

4.3. Using Results in Support of Designs

The relationships identified in this study spanned a variety of structure types having a broad range of measured values for each parameter. As an initial step in understanding limitations on the design of these structures, a summary table of the values at which structures are noted to experience some degree of failure was developed (Table 9). The values represent averages and can be used to guide decision making related to the design of the structure configuration. The exact values do not guarantee a structure's success, but can be consulted to determine if the values for a given design falls within the ranges observed in this field analysis. The use of the information in this table must be coupled with an understanding of the physical processes of the system. Because scour along the structure arms was found to be a substantial factor in the structural performance of the rock weirs visited, an additional row of footer rocks is recommended to increase the foundation integrity along structure arms.

Table 9. Values for select parameters above which some degree of failure was noted. Except where noted, the values are representative of general failure and do not differentiate between partial and full.

Structure Parameter	Structure Type	Value at Which Some Degree of Failure was Noted
High Flow Recurrence Interval	All	greater than 7-year (partial), 21-year (full)
Throat Width/Structure Width	All	less than 0.28 (partial), 0.21 (full)
Maximum Plan Angle	U-,V-weirs	greater than 35 degrees
	asymmetrical U-weirs	greater than 33 degrees (partial), 40 degrees (full)
Maximum Arm Angle	U-,V-weirs	greater than 50 degrees
	asymmetrical U-weirs	greater than 60 degrees
Open Angle	All	greater than 60 degrees
Maximum Arm Profile Slope	All	greater than 8-10 %
Structure Width	All	greater than 60 feet
Tie-in Length	All	less than 12 feet
Tie-in Length/Structure Width	All	less than 0.20
Stream Power (QS)	U-,V-weirs	greater than 15 ft^3/s
Thalweg slope	All	greater than 1%

The most commonly referenced guidance for the design of river spanning rock structures is Rosgen (1996 and 2001). Within the documentation, Rosgen identifies acceptable parameters for design of cross-vanes. The range of values that Rosgen recommends for U-weirs are compared with the values measured in this investigation in Table 10. One primary difference between the two sets of information is that the recommendations are for design, while the measured values were acquired following structure implementation and adjustment to high flows. The ranges surveyed in the field are comprised mostly of structures that were subject to some degree of failure.

Table 10. Comparison of recommended cross-vane design ranges (Rosgen 1996, 2001) and ranges for U-,V- weirs surveyed in the field.

	Rosgen Recommendations	Ranges Surveyed in Field	Ranges of Surveyed Structures Experiencing No Failure
Throat Width	1/3 bankfull (structure) width	0 to 9/10 structure width	1/8 to 9/10 structure width
Arm Profile Slope	2 to 7% slope	-14 to 22%	-3 to 15%
Plan Angle (between bank and arm)	20 to 30 degrees	4 to 62 degrees	7 to 32 degrees

4.4. Foundation Depth Design

4.4.1. Application of Equations Developed by Colorado State University

Since the most common failure mechanism identified by Mooney et al (2007) was geotechnical slumping of the footer rocks, the depth of foundation relative to the depth of maximum scour is a critical design parameter that impacts the success of the structural stability of a rock weir. Based on these findings from field investigations, Colorado State University was contracted to construct a series of physical models to quantify maximum equilibrium scour depth. An extensive literature review of scour depths prediction methods was conducted by Scurlock (2009). Scurlock determined that the equation formulated by D'Agostino and Ferro (2004) for scour of alluvial beds downstream of grade control structures was best suited for the development of an equation to predict scour downstream of rock weirs because the form of the equation allowed for representation of all variables manipulated during the test matrix in a single, verified equation format (Equation 1).

Equation 1

$$\frac{y_{SE}}{z_i} = a_1 \left(\frac{b_i}{z_i}\right)^{a_2} \left(\frac{y_t}{H}\right)^{a_3} \left(\frac{Q}{b_i z_i \sqrt{g(\Delta - 1) d_{90}}}\right)^{a_4} \left(\frac{d_{90}}{d_{50}}\right)^{a_5} \left(\frac{b_i}{B}\right)^{a_6}$$
$$i = \{A, U, W\}$$

where:

a_1 = coefficient fit to data;

a_2, a_3, a_4, a_5, and a_6 = exponents fit to data;

B = channel width;

d_{50} = sediment diameter where 50 percent of total is smaller by size;

d_{90} = sediment diameter where 90 percent of total is smaller by size;

H = piezometric drop across structure;

Δ = specific gravity of material;

b_i = effective weir length for weirs i = {A, U, W};

z_i = mean weir height above bed for weirs i = {A, U, W};

g = gravitational acceleration;

y_t = tailwater depth;

y_{SE} = maximum equilibrium scour depth;

Q = discharge;

From data collected in a physical model of thirteen rock weir configurations, coefficients and exponents were fit to the data for development of predictive equations for rock weirs. Because the original equation was developed for scour downstream from grade control structures, Scurlock (2009) used an effective weir length to represent the weir width and a mean weir height above the bed to represent the drop over the structure.

Table 11. Dimensional analysis scour depth equation results

Weir	Predictive equation	
A	$$\frac{y_{SE}}{z_A} = 2.167E-5\left(\frac{b_A}{z_A}\right)^{-6.201}\left(\frac{y_t}{H}\right)^{-0.491}\left(\frac{Q}{b_A z_A\sqrt{g(\Delta-1)d_{90}}}\right)^{1.254}\left(\frac{d_{90}}{d_{50}}\right)^{20.299}\left(\frac{b_A}{B}\right)^{87.900}$$	
	Predicted vs. observed scour depths	
	Correlation Coefficient	0.984
	Mean Square Error (ft²)	0.00179
U	$$\frac{y_{SE}}{z_U} = 9074.589\left(\frac{b_U}{z_U}\right)^{-2.958}\left(\frac{y_t}{H}\right)^{-0.438}\left(\frac{Q}{b_U z_U\sqrt{g(\Delta-1)d_{90}}}\right)^{1.127}\left(\frac{d_{90}}{d_{50}}\right)^{10.284}\left(\frac{b_U}{B}\right)^{-0.609}$$	
	Predicted vs. observed scour depths	
	Correlation Coefficient	0.787
	Mean Square Error (ft²)	0.0437
W	$$\frac{y_{SE}}{z_W} = 0.478\left(\frac{b_W}{z_W}\right)^{1.758}\left(\frac{y_t}{H}\right)^{-0.894}\left(\frac{Q}{b_W z_W\sqrt{g(\Delta-1)d_{90}}}\right)^{1.610}\left(\frac{d_{90}}{d_{50}}\right)^{-3.196}\left(\frac{b_W}{B}\right)^{-4.462}$$	
	Predicted vs. observed scour depths	
	Correlation Coefficient	0.879
	Mean Square Error (ft²)	0.0135

These relationships were applied to surveyed field data to investigate how well they translate to full-scale rock weirs measured in the field. Two of the structures visited in the field were initially chosen for analysis; one structure from the Entiat River (RM 5.1) and one structure from the East Fork Salmon River (EF 7-8). From the complete survey data collected between 2005 and 2008, sufficient information was available to implement the predictive equations for equilibrium scour depth. A HEC-RAS model was constructed for each site to develop a rating curve for depth and flow at the downstream cross-sections outside the influence of the hydraulic structure. For each structure, the rating curve was used to determine the tail water depth downstream from the structure (assuming normal depth conditions at the downstream boundary). This tail water depth was then used in the stage-discharge relationship developed by Meneghetti (2009) to compute the piezometric head drop across the structure. All other variables in the predictive equations were determined from the survey data.

Results of these two scenarios suggested that the equations developed in the laboratory tended to over predict scour measured in the field. Within the equations, the non-dimensional terms from the laboratory data had a relatively small range compared with field conditions (Table 12). The fourth term, sediment gradation (d_{90}/d_{50}), for the Entiat structure was outside the laboratory range, while the first and fifth terms for the East Fork Salmon structure fell outside of the range of application. Furthermore, the coefficients and exponents determined for each equation were derived from a small number of total tests. Both the A- and W-weir equations were developed from 9 tests, while the U-weir equation was developed from 19 tests. These results illustrate a pronounced need for additional tests to strengthen the rigor of the equations and extend their application to field conditions.

Table 12. Range of terms used in the development of the equations from the physical laboratory compared with the values from two field cases. The highlighted values represent terms that are outside of the range of applicability.

	Term 1	**Term 2**	**Term 3**	**Term 4**	**Term 5**
	b_i/z_i	y_t/H	Q/(b$_i$*z$_i$*(sqrt(g*1.65*d90))	d_{90}/d_{50}	b_i/B
Physical Laboratory Ranges					
Minimum	69.64	0.72	0.40	1.43	1.33
Maximum	129.35	5.93	2.89	1.88	2.93
Entiat RM 5.1	91.99	3.90	0.71	3.07	1.89
East Fork Salmon 7-8	54.37	4.88	0.66	1.77	0.91

Upon further investigation of the equation of D'Agostino and Ferro (2004), we found that the coefficients and exponents developed for the original equation (Equation 2) were derived from 248 data points across a much broader range of laboratory conditions. However, the original form of the equation was developed for scour downstream of grade control structures. Drawing from Scurlock's work, effective weir length (b_i) and mean drop height over structure (z_i) were substituted into the equation for weir width (b) and drop height over weir (z_d). In addition, d_{90} was substituted into term 3 for d_{50} as recommended by Scurlock to develop Equation 3 for further analysis.

Equation 2

$$\frac{y_{SE}}{z_d} = 0.540\left(\frac{b}{z_d}\right)^{0.593}\left(\frac{y_t}{H}\right)^{-0.126}\left(\frac{Q}{bz_d\sqrt{g(\Delta-1)d_{50}}}\right)^{0.544}\left(\frac{d_{90}}{d_{50}}\right)^{-0.856}\left(\frac{b}{B}\right)^{-0.751}$$

Where

b = width of weir;

z_d = drop height over weir;

and all other variables previously defined.

Equation 3

$$\frac{y_{SE}}{z_i} = 0.540\left(\frac{b_i}{z_i}\right)^{0.593}\left(\frac{y_t}{H}\right)^{-0.126}\left(\frac{Q}{b_i z_i\sqrt{g(\Delta-1)d_{90}}}\right)^{0.544}\left(\frac{d_{90}}{d_{50}}\right)^{-0.856}\left(\frac{b_i}{B}\right)^{-0.751}$$

Where all variables have been previously defined.

Equation 3 was applied to the field test cases to compare if the predicted values for maximum scour depth improved with the modified coefficients and exponents. Results comparing both methods for a 2-year discharge are shown in Table 13 along with the scour depths measured during the topographic surveys. Both sites have experienced a 2-year discharge. However, the maximum scour depth was difficult to obtain at the sites due to depths that were either unreachable or pools that had partially filled with material from the rock weir or incoming sediment, and due to a lack of bed topography prior to construction with which to compare the surveyed topography. The maximum scour depths reached at each site are likely larger than the reported measured scour depths.

Table 13. Results of applying the predictive equations to two field cases for a 2-year discharge.

	Predicted Scour Depths (ft)		Measured Scour Depth (ft)
	Scurlock (2009)	Modified D'Agastino and Ferro (2004)	
Entiat RM 5.1	871.4	4.2	4.1
East Fork Salmon 7-8	18.4	6.9	4.7

As previously mentioned, equations developed by Scurlock (2009) overpredicted scour depths for the two test cases, each of which had at least one term outside of the range of applicability. Scurlock's equations proved to be highly sensitive to gradations outside the range of development. Additional physical modeling tests are necessary to improve the use of the equations outside of the current ranges. The modified version of the D'Agastino and Ferro (2004) equation proved to have results that are within a reasonable range of expectation for the two cases examined. Further validation of this equation is necessary to determine it suitability for all rock weirs. However, preliminary results hold promise of its application for foundation depth design.

4.4.2. Potential Future Investigations

Another possible method for predicting scour downstream from rock weirs is to apply equations for scour prediction downstream from dams. One computational method that is commonly used to estimate scour formed from free-jet conditions over a dam was developed by Anandale (2006). This same approach may be applicable to rock-weirs through manipulation of the predictor variable and calibration of the equation's coefficients and exponents. Calibration of these data could be accomplished through further analysis of the physical laboratory data from Colorado State University. An outline of this proposed methodology is presented below.

Table 14 provides definitions for each of the symbols used in the computations presented, and Figure 47 illustrates the physical representation of several of the design parameters. This method is presented for a simplified case in which the geometry of the channel and the structure are fairly uniform. Incorporating more complex design geometry will require additional 2- or 3-dimensional modeling to accurately predict hydraulic conditions over the structure at different points along the structure crest.

Table 14. Definition of symbols used in the equations presented within this section of the report. Units are presented as length (L), time (T), or mass (M).

Symbol	Representation
τ	Plunging shear acting on the channel bed (M/LT^2)
τ_c	Critical shear stress required to mobilize bed material (M/LT^2)
ρ	Fluid density (M/L^3)
v	Velocity at a particular location along the bed (L/T)
C_p	Average dynamic pressure coefficient
Δh	Head loss over structure (L)
g	Gravity (L/T^2)
γ	Specific weight of water (M/(L^{2}T^2))
γ_s	Specific weight of sediment (M/(L^{2}T^2))
C_1	Jet dissipation coefficient
θ_c	Critical Shield's parameter
D_s	Depth of scour measured relative to tail water elevation (L)
h_d	Flow depth over structure (L)
h_t	Flow depth of tailwater elevation downstream from structure influence (L)
D_f	Depth of foundation below original bed elevation (L)
d_{50}	Mean sediment size of the bed material present in the location of the scour pool (L)
F_s	Factor of safety

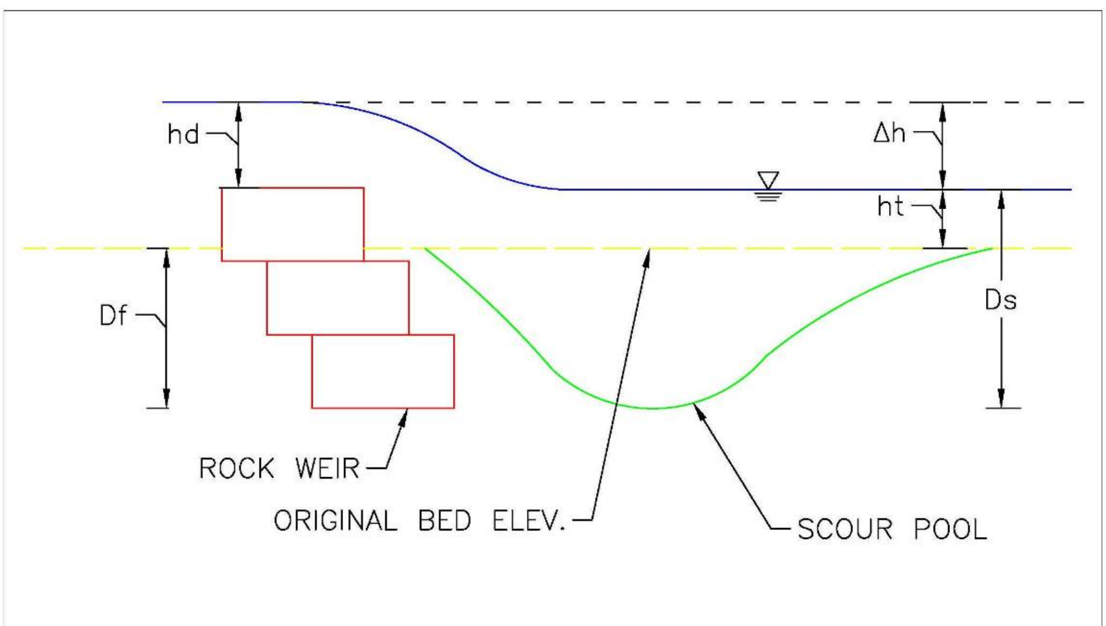

Figure 47. Definition sketch for rock weir scour development under non-submerged condition. Schematic represents an idealized profile down the centerline of the structure.

Assumptions: Scour is presumed to stop and the bed will become stable when shear stresses through the structure no longer exceed critical shear stress; when $\tau = \tau_c$.

Equations:

$\tau = C_p \frac{1}{2} \rho v^2$ can be used to compute the plunging shear stress acting on the bed.

$v = \sqrt{2g\Delta h}$ represents the velocity resulting from the head drop, and therefore,

$\tau = C_p \gamma \Delta h$.

$C_p = a \exp(-C_1 ({D_s}/{h_d}))$ represents the average dynamic pressure coefficient as a function of the ratio of scour pool depth to flow depth over the structure (synonymous to the dimensionless plunge pool depth). For jet dissipation assuming free jet, $C_1 \approx 0.07$ and $a \approx 1$ (Annandale, 2006).

$\tau_c = \theta_c (\gamma_s - \gamma) d_{50}$ calculates critical shear stress where $\theta_c \approx 0.02 - 0.04$. The ratio of the plunging force acting on the bed to the critical shear stress then becomes:

$$\frac{\tau}{\tau_c} = \frac{C_p \gamma \Delta h}{\theta_c (\gamma_s - \gamma) d_{50}} = \frac{C_p \Delta h}{\theta_c (s-1) d_{50}}$$

The scour depth criteria that must be met for structure stability can then be expressed as

$\dfrac{a * \exp(-C_1 ({D_s}/{h_d}))\Delta h}{\theta_c (s-1) d_{50}} = 1$; $C_1, a\ \&\ \theta_c$ need to be calibrated first to data from the physical lab and then expanded to apply to field data.

Design foundation depth could then be determined as a function of the maximum expected scour depth.

$D_f = F_s (D_s - h_t)$

A factor of safety of 1.2 to 1.5 is recommended initially to design the foundation beyond anticipated scour depths.

Several uncertainties still exist with this type of design equation due to assumptions associated with design flows and calibrating field parameters to conditions that existed prior to the implementation of the structures. Data from one structure on the Entiat River was initially used to determine how much scour was predicted during a 2-year discharge. The equation predicted scour depths up to 100 feet using the uncalibrated coefficients and exponents. One problem encountered in using the field data to develop appropriate calibrations of the coefficients and exponents is that no accurate measured maximum scour depths are available at each of the field sites. In many cases, scour depths measured during field surveys are available, but these data points can not capture the maximum scour obtained during a flood discharge or just prior to experiencing mobility of the footer rocks. Following initial attempts to apply this equation to field data, it is recommended that additional analysis of data from the physical model be utilized to determine the appropriateness of the methodology and to initially calibrate coefficients and exponents.

4.5. Data Gaps

4.5.1. Additional information or parameters that may be correlated to structure failure

A large amount of topographic information was collected during the site evaluations. During the analysis, several parameters were identified that could be used to further the statistical analysis and might be strongly correlated to the degree of structure failure and possibly the mechanism for failure. However, these parameters could not be quantified from the survey data at each site. The parameters include:

- Sediment size in structure scour pools
- Depths of structure foundation rocks
- Distance of the spacing or voids between the rocks used in the structure
- Throat and cross-bar drop heights from the original bed elevation

In addition, during the analysis, parameters were identified that were not consistently collected throughout the first two stages of site evaluations. These were:

- Dimensions of the scour pool
- Delineation of the river banks near the structures.
- Sediment size upstream of the structure
- Sediment size downstream of the structure
- Size of rock material used in structure

For future field investigations, the parameters listed above should be collected in addition to the information that has been consistently collected in previous site evaluations (structure planform and profile parameters). A bed slope that is more representative of the reach should also be collected to more accurately capture reach scale properties. While the focus of this research effort has been on defining the structure success related to motion of the constituent rocks, the ability of each structure to provide habitat value has emerged as an important issue for further investigation. Input from local biologists was collected in October 2008, to reach a broader definition of success that encompasses habitat suitability. Input from local biologists should continue to be collected to assist with determining structures' habitat value.

4.5.2. Sites with incomplete data

Field visits were conducted between 2005 and 2008, to collect information for use in the development of design guidelines. Some sites had more extensive surveys and data collection than other sites. Table 15 illustrates which sites had less extensive topographic surveys and the information lacking at each site.

Table 15. Sites visited previously with more information collection needs

Site	Structures (A, W, V/U Weirs)	Additional Information Needed
Grande Ronde	V weirs	No scour pool survey for B structure
Rio Blanco	A, V, and U weirs	No scour pool survey for B and D structure
North Fork Little Snake	U and asymmetric-U weirs	Unclear pool surveys
Middle Fork Little Snake	A, U and asymmetric-U weirs	Unclear pool surveys
South Fork Little Snake	U and asymmetric-U weirs	Unclear pool surveys
Lower Beaver Creek	U weirs	Bank lines
Red Hill	U with crossbars weirs	No scour pool survey for E, F structures, Incomplete thalweg.

Although many sites were evaluated previously, multiple sites in the qualitative report did not have topographic surveys. These sites are listed in Table 16. For sites that were partially surveyed, the number of structures listed represents the number of structures remaining to be surveyed at that site.

Table 16. Sites visited previously with no topographic information collected.

Site	Number of Structures to Survey
Lemhi L3A	A weirs (3)
Lemhi L3	Angled Rock Dam (1)
Lemhi L9	Partial Rock Ramp (1)
Tulley Hill	Pole and V weirs (3)
Muddy Creek	Unknown
Garden Creek	Rock Ramp (1)
North Fork Little Snake *partial survey	U weirs (4)
South Fork Little Snake *partial survey	Asymmetric U and U weirs (13)
East Fork Salmon *partial survey	U weirs (1)
Bear Creek *partial survey	U weirs (approximately 11)

5. Conclusions

Within the context of this report, quantitative analyses were performed on 69 structures that contained adequate survey information for development of a consistent data base of information. Surveys collected were processed and digitized in ArcGIS and Microsoft Excel to quantify up to 80 geometric parameters at each site. These parameters were used as predictor variables to understand the relationships between the individual variables and the degree of failure associated with each structure type. The analysis did not incorporate any interactions between the predictor variables since the response variables were categorical (no failure, partial failure, full failure) and not defined in a quantitative manner that would have allowed for a multiple regression or principal component analysis.

Through this analysis, several parameters used in the design of rock weirs were linked to the potential for a structure to experience failure. Structures having experienced flows with a higher recurrence interval had a higher likelihood of failure than those subject to flows with smaller recurrence intervals. In general, structures with wider throats relative to the channel width failed less frequently than those with smaller or no throats (V-weirs). Structures characterized by wider plan, arm, and open angles tended to have greater arm profile slopes, and were observed to fail more frequently than those with narrower angles and milder slopes. With an increase in the profile angle of the arm, the head drop over the arm of the structure increases. The footers along the structure arms then become the most susceptible to the greatest energy dissipation during high flows. This hydraulic process implicates a need for increased depth of the foundation along the structure arms, particularly where arm angles exceed 9 percent (as measured from surveyed data of U,V-weirs). Another valuable relationship detected in this investigation was between the scour offset from the structure and the degree of failure. Based on the observations of this study, the closer the maximum scour depth is to the structure crest, the more likely the structure is to have experienced failure.

Additional parameters were identified as being important variables that relate to structure performance, but lacked clear relationships with failure, including structure spacing, planform location and scour depth. U,V- weirs placed in close sequence tended to fail less frequently than those with wide distances between structures. However, asymmetric U-weirs spaced too closely had a higher incidence of failure than those spaced farther apart. Installation of multiple structures should consider the project objectives and the hydraulics to determine adequate spacing so as not to limit pool development, cause the pool to inhibit stability of the next downstream structure, or impact structure submergence at lower than design flows. The scour depths surveyed did not represent the maximum scour experienced by the structure prior to failure. Measuring the scour depth that would be responsible for structure failure requires knowledge of the foundation depth of each structure below the existing bed elevation and would entail intensive surveying during high flow conditions, which is extremely difficult and often unsafe. The location of the structure with respect to meander pattern did not illustrate a strong relationship with the degree of failure. This may be in part related to the sample population, or possibly related to system interactions, such as lateral channel migration, that cannot be captured with geometric parameters of the structures alone. One finding associated with planform location was that all structures failing as a result of general bank migration/flanking were located on bends.

The quantitative analysis performed in this study must be coupled with the results from the laboratory and numerical modeling components to best understand the relationships between geometric variables of the structure and structure stability. Interactions between the variables add further complexity to the ability to evaluate an individual variable. An evaluation of the interactions of multiple variables is more suitable to numerical and physical modeling. However, the relationships identified through the field investigation help inform inputs to the numerical and physical modeling and can be used to assess ranges of specific parameters that drive structure stability. Furthermore, the information from the field investigations can be used to determine how well relationships developed from the physical and numerical modeling translate to the field.

As a first step in synthesizing field findings with other aspects of the research, scour relationships developed through the physical model were applied to the field data. Equations developed by Scurlock (2009) from physical model results at Colorado State University overpredicted scour depths for two test cases evaluated, each of which had at least one term outside of the range of applicability. A modified version of the D'Agastino and Ferro (2004) equation proved to have results that are within a reasonable range of expectation for the two cases examined. Further validation of both equations is necessary to determine the suitability for foundation depth design across a broad range of rock weirs, but preliminary results are promising.

The parameters included in this analysis did not address system wide processes that would need to be assessed on a larger scale to evaluate potential impacts to structure stability. Results of this analysis indicate that an understanding of the fluvial geomorphic processes is key to the success of the structure and the required level of maintenance. Lateral channel migration potential, sediment transport characteristics, flow regime characteristics, and other historical information related to channel morphology are essential to determining the potential use and location of river spanning rock structures and understanding design limitations and future maintenance associated with their finite presence in dynamic systems. Geomorphologists are currently working to develop a guide for determining what processes are important, how to evaluate those processes most efficiently, and how knowledge of those process increase the potential for project success.

This investigation identified several important relationships between structure parameters and the degree of failure. The most notable include the relationships with recurrence interval of high flows, throat width, planform angles, and scour offset from structure. Other design variables that influence structure performance include structure spacing, scour depth, and planform location. As the numerical modeling progresses and the database on field measured parameters continues to increase over time, current understanding of the importance of various structure parameters, their interactions with other structure parameters, and the most successful ranges will improve.

6. References

Annable, W.K. (2007). Personal communications with W.K. Annable between March and November 2007. Department of Civil and Environmental Engineering, University of Waterloo, Waterloo, Ontario, Canada.

Annandale, G.W. (2006). Scour Technology: Mechanics and Engineering Practice. McGraw-Hill, New York, pp 175-180.

D'Agostino, V. and Ferro, V. (2004). "Scour on alluvial bed downstream of grade control structures." J. Hydr. Engrg., ASCE, 130(1). 24-37.

Meneghetti, Anthony M. (2009). "Stage discharge relationships for U-, W-, and A-weirs," thesis presented to Colorado State University, at Fort Collins, Colo., in partial fulfillment of the requirements for the degree of Masters of Science.

Meyer, J. (2007) Restoration Structure Inventory and Survey Results from the Little Snake River and Tributaries, Northern Colorado. Masters Thesis, Colorado State University, Fort Collins.

Mooney, D., Holmquist-Johnson, C., and E. Holburn (2007) Qualitative Evaluation of Rock Weir Field Performance. Bureau of Reclamation, Technical Service Center, Sedimentation and River Hydraulics Group, Denver, CO.

Rosgen, D.L. (1996). Applied River Morphology. Wildland Hydrology Books, Pagosa Springs, CO.

Rosgen, D.L. (2001). The Cross-vane, W-weir and J-hook Vane Structures...Their Description, Design and Application for Stream Stabilization and River Restoration. Pagosa Springs, CO.

Scurlock, M. (2009) Equilibrium Scour Downstream of Three-dimensional Grade Control Structures. Masters Thesis, Colorado State University, Fort Collins.

Sutley, David E. (2006) Methow River Drainage Basin, Hydrology Data and GIS for the Methow River, In-Stream Habitat Restoration Project, Bureau of Reclamation, Technical Service Center, Denver, CO.

Sutley, David E. (2009) Appendix B-Hydrology Data and GIS, Entiat River Tributary Assessment, Bureau of Reclamation, Technical Service Center, Denver, CO.

United States Department of Agriculture (USDA). (2009) National Agriculture Imagery Program (NAIP). Accessed from http://gis.apfo.usda.gov/naipcoverage/ . Last accessed October 2009.

Appendix A: Description of All Parameters Measured

Table 1. List and description of all structure parameters evaluated.

Parameter	Description
Structure Opening (ft)	Distance between furthest arm extents
Structure Width (ft)	Width of structure in direction perpendicular to flow
Thalweg Slope (ft/ft)	General slope of river bed elevation
Throat width (ft)	Width of structure throat
Left arm length (ft)	Length of left arm
Left arm tie in (ft)	Length of left tie-in
2 arm length (ft)	Length of second arm
3 arm length (ft)	Length of third arm
4 arm length (ft)	Length of fourth arm
5 arm length (ft)	Length of fifth arm length
Right arm length (ft)	Length of right arm
Right arm tie in (ft)	Length of right tie-in
Tie in Length (ft)	Length of structure that extends laterally past bankline
Tie in Length / Structure Width	Tie in Length non-dimensionalized by width of structure.
CrossBar width (ft)	Width of cross-bar
CrossBar 2 width (ft)	Width of secondary cross-bar
Left lateral constriction (ft)	Distance between left bankline and left arm extent
Right lateral constriction (ft)	Distance between right bankline and right arm extent
Left arm plan angle (deg)	Angle between left arm and left bank
Right arm plan angle (deg)	Angle between right arm and right bank
Open angle A (deg)	Open angle for opening A
Open angle B (deg)	Open angle for opening B
Open angle C (deg)	Open angle for opening C
Left arm structure angle (deg)	Angle between left arm and throat
Left arm structure angle relative to throat (deg)	Angle between left arm and throat - 90 degrees
Right arm structure angle (deg)	Angle between right arm and throat
Right arm structure angle relative to throat (deg)	Angle between right arm and throat - 90 degrees
CrossBar offset from throat (ft)	Distance from throat midpoint to crossbar midpoint

Parameter	Description
CrossBar 2 offset from throat (ft)	Distance from throat midpoint to secondary crossbar midpoint
Crest A offset (ft)	Distance from A upstream point to left bank
Crest B offset (ft)	Distance from B upstream point to left bank
Crest C offset (ft)	Distance from C upstream point to left bank
Crossbar offset (ft)	Distance from crossbar midpoint to left bank
Crossbar 2 offset (ft)	Distance from secondary crossbar midpoint to left bank
Scour A to Crest offset (ft)	Distance from A scour point to A throat
Scour B to Crest offset (ft)	Distance from B scour point to B throat
Scour C to Crest offset (ft)	Distance from C scour point to C throat
CrossBar Scour to Crest offset (ft)	Distance from crossbar scour to crossbar midpoint
CrossBar 2 Scour to Crest offset (ft)	Distance from secondary crossbar scour to secondary crossbar midpoint
Scour A to Left Bank offset (ft)	Distance from A scour point to left bank
Scour B to Left Bank offset (ft)	Distance from B scour point to left bank
Scour C to Left Bank offset (ft)	Distance from C scour point to left bank
CrossBar scour to Left Bank offset (ft)	Distance from crossbar scour point to left bank
CrossBar 2 scour to Left Bank offset (ft)	Distance from secondary crossbar scour point to left bank
Upstream Structure (ID)	Identification of next upstream structure
Distance to Upstream Structure (ft)	Distance to next upstream structure
Upstream RSRS (ID)	Identification of next upstream river spanning rock structure
Distance to Upstream RSRS (ft)	Distance to next upstream river spanning rock structure
River Widths to Upstream Structure (ft)	Distance to next upstream structure divided by structure width
River Widths to Upstream RSRS (ft)	Distance to next upstream river spanning rock structure divided by structure width
Downstream Structure (ID)	Identification of next downstream structure
Distance to Downstream Structure (ft)	Distance to next downstream structure
Downstream RSRS (ID)	Identification of next downstream river spanning rock structure
Distance to Downstream RSRS (ft)	Distance to next downstream river spanning rock structure
River Widths to Downstream Structure (ft)	Distance to next downstream structure divided by structure width
River Widths to Downstream RSRS (ft)	Distance to next downstream river spanning rock structure divided by structure width
General Structure Spacing (river widths)	Minimum spacing toe either next upstream or downstream structure
RSRS Spacing (river widths)	Minimum spacing toe either next upstream or downstream river spanning rock structure
Structure Location (Type)	Structure location in meander pattern (left bend, right bend, crossover, straight)
Scour A Elevation (ft)	Elevation of A scour point

Parameter	Description
Scour B Elevation (ft)	Elevation of B scour point
Scour C Elevation (ft)	Elevation of C scour point
CrossBar Scour Pool Elevation (ft)	Elevation of crossbar scour point
CrossBar 2 Scour Pool Elevation (ft)	Elevation of secondary crossbar scour point
Scour Depth A (ft)	Difference between A throat elevation and A scour point elevation
Scour Depth B (ft)	Difference between B throat elevation and B scour point elevation
Scour Depth C (ft)	Difference between C throat elevation and C scour point elevation
Cross bar scour depth (ft)	Difference between crossbar elevation and crossbar scour point elevation
CrossBar 2 scour depth (ft)	Difference between secondary crossbar elevation and secondary crossbar scour point elevation
Crest Elevation (ft)	Elevation of structure throat
CrossBar Elevation (ft)	Elevation of crossbar
CrossBar 2 Elevation (ft)	Elevatoin of secondary crossbar
CrossBar Drop Height (ft)	Difference in throat elevation and crossbar elevation
CrossBar 2 Drop Height (ft)	Difference in throat elevation and secondary crossbar elevation
Left Tie-In Elevation (ft)	Elevation of left tie-in
Right Tie-In Elevation (ft)	Elevation of right tie-in
Left Tie-In Height (ft)	Difference in left tie-in elevation and structure throat elevation
Right Tie-In Height (ft)	Difference right left tie-in elevation and structure throat elevation
Left Arm Profile Slope (ft/ft)	Slope of left arm
2 Arm Profile Slope (ft/ft)	Slope of second arm
3 Arm Profile Slope (ft/ft)	Slope of third arm
4 Arm Profile Slope (ft/ft)	Slope of fourth arm
5 Arm Profile Slope (ft/ft)	Slope of fifth arm
Right Arm Profile Slope (ft/ft)	Slope of right arm
Min Structure Arm Profile Slope (ft/ft)	Minimum arm slope (from left, second, third, fourth, fifth, and right arms)
Max Structure Arm Profile Slope (ft/ft)	Maximum arm slope (from left, second, third, fourth, fifth, and right arms)
Min Arm Length (ft)	Minimum arm length (from left, second, third, fourth, fifth, and right arms)
Max Arm Length (ft)	Maximum arm length (from left, second, third, fourth, fifth, and right arms)
Min Structure arm angle (deg)	Minimum structure arm angle (from left, second, third, fourth, fifth, and right arms)
Max Structure arm angle (deg)	Maximum structure arm angle (from left, second, third, fourth, fifth, and right arms)
Throat Width / Structure Width	Throat width non-dimensionalized by structure width
Min arm Length / Structure Width	Minimum arm length non-dimensionalized by structure width

Parameter	Description
Max arm Length / Structure Width	Maximum arm length non-dimensionalized by structure width
Min Plan Angle	Minimum arm plan angle (from left, second, third, fourth, fifth, and right arms)
Max Plan Angle	Maximum arm plan angle (from left, second, third, fourth, fifth, and right arms)
Max Scour (ft)	Maximum measureable scour depth
Max Scour to Crest Offset (ft)	Distance from maximum measureable scour depth location to nearest crest (arm or throat) rock
Scour Offset / Structure Width	Max scour to crest offset non-dimensionalized by structure width
Scour Depth / Scour Offset	Maximum measureable scour depth non-dimensionalized by max scour to crest offset
Scour Depth / Structure Opening	Maximum measureable scour depth non-dimensionalized by structure opening
Scour Depth / Structure Width	Maximum measureable scour depth non-dimensionalized by structure width
Scour Depth / Throat Width	Maximum measureable scour depth non-dimensionalized by throat width
High Flow Since Construction (cfs)	Estimated discharge of high flow event since structure construction
Date of High Flow	Date of high flow event experienced by the structure
Recurrence Interval of High Flow	Estimated return interval of high flow event experienced by structure
2yr Discharge (cfs)	Estimated two year discharge based on data of nearest discharge gage.
2yr unit discharge (ft2/s)	Two year discharge divided by structure width
StreamPower (cfs)	Estimated two year discharge multiplied by channel slope.
25yr Discharge (cfs)	Estimated twenty-five year discharge based on data of nearest discharge gage.
Arm Rock Width (cm)	Dimension of structure arm rock in the direction of flow
Arm Rock Depth (cm)	Dimension of structure arm rock in the vertical direction
Throat Rock Width (cm)	Dimension of structure throat rock in the direction of flow
Throat Rock Depth (cm)	Dimension of structure throat rock in the vertical direction
Arm Rock Diameter (cm)	Average of arm rock width and arm rock depth
Throat Rock Diameter (cm)	Average of throat rock width and throat rock depth
Crossbar Rock Diameter (cm)	Representative rock size of strructure crossbar rock
Rock Size (cm)	average of available structure rock sizes (throat and/or arm and/or crossbar)
Fines <6mm Upstream (%)	Percentage of fines in bed material upstream of structure
Upstream D16 (mm)	Size representing the 16th percentile of the upstream bed material grain size distribution
Upstream D35 (mm)	Size representing the 35th percentile of the upstream bed material grain size distribution
Upstream D50 (mm)	Size representing the 50th percentile of the upstream bed material grain size distribution
Upstream D84 (mm)	Size representing the 84th percentile of the upstream bed material grain size distribution
Upstream D95 (mm)	Size representing the 95th percentile of the upstream bed material grain size distribution
Upstream %Bedrock	Percentage of bedrock in bed material upstream of structure

Parameter	Description
Fines <6mm Downstream(%)	Percentage of fines in bed material downstream of structure
Downstream D16 (mm)	Size representing the 16th percentile of the downstream bed material grain size distribution
Downstream D35 (mm)	Size representing the 35th percentile of the downstream bed material grain size distribution
Downstream D50 (mm)	Size representing the 50th percentile of the downstream bed material grain size distribution
Downstream D84 (mm)	Size representing the 84th percentile of the downstream bed material grain size distribution
Downstream D95 (mm)	Size representing the 95th percentile of the downstream bed material grain size distribution
Downstream %Bedrock	Percentage of bedrock in bed material downstream of structure
D50 (mm)	Average of upstream and downstream D50

APPENDIX B – HYDROLOGY TASKS

RIVER SPANNING ROCK STRUCTURE RESEARCH

U.S. BUREAU OF RECLAMATION

TECHNICAL SERVICE CENTER, DENVER, CO
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CONTENTS

LIST OF TABLES

1. INTRODUCTION

This appendix documents hydrology tasks for River Spanning Rock Structure Research being accomplished by the Technical Service Center. This appendix contains the following information:

- Description of streamflow data available at rock structure locations
- Flood frequency analysis at rock structure locations
- Estimated recurrence interval and magnitude of the largest flood since construction of each rock structure

2. METHODOLOGY

2.1 DESCRIPTION OF STREAMFLOW DATA:

The nearest USGS stream gages were identified for the structures surveyed as part of the field component of the River Spanning Rock Structure Research. If a USGS stream gage with sufficient data was not located on the same tributary, the nearest gage with the most representative flow patterns was assumed to be applicable to the tributary of interest. Table B-1 contains a list of all the rock structure locations and USGS gages used in the analysis. The annual peak discharge data and mean daily discharge data for each gage listed below was downloaded from the USGS National Water Information System (NWIS): Web Interface [1].

Table B-1. River Spanning Rock Structure locations and USGS gages used for flood frequency analysis.

Structure Location	Drainage Area above Structure (mi^2)	USGS Gage Number(s) used for Flood Frequency Analysis	Drainage Area of Gage(s) (mi^2 ,respectively)
Bear Creek	72	13330500	68
Beaver Creek	108	12449600	62
Catherine Creek	111	13320000	105
Chewuch River	466	12447600	466
East Fork of the Salmon River	286	13298000	532
Salmon River	235	13295500, 13295000	501, 147
Entiat River	395	12452990	419
Grande Ronde	461	13319000	678
Lemhi River	1231	13305500, 13305310	1270, 1216
Middle Fork of the Little Snake River	115	09253000	285
North Fork of the Little Snake River	46	09253000	285
Middle and North Forks of the Little Snake River	161	09253000	285
South Fork of the Little Snake River	45	09253000	285
Rio Blanco	104	09343000	58
San Juan River	281	09342500	298

2.2 FLOOD FREQUENCY ANALYSIS:

The structures on Beaver Creek and the Chewuch River are in the Methow River drainage basin. A basin-wide flood frequency analysis (FFA) for the Methow River was completed in 2006 [2]. The 2-, 5-, 10-, 25-, 50-, and 100-year floods computed in the 2006 study were applied to the structures on Beaver Creek and the Chewuch River. A similar FFA was also completed for the Entiat River in 2009 [3]. This study computed the 2-, 5-, 10-, 25-, 50-, and 100-year floods for Entiat River miles 0.2 to 32. The flood estimates computed for river mile 4 were applied to the structures on the Entiat River. The frequency discharge estimates for these three tributaries are listed in Table B-2.

Table B-2. River Spanning Rock Structure frequency discharges applied from previous studies.

		Frequency Discharge (ft^3/s)					
Structure Location	Drainage Area (mi^2)	2-year	5-year	10-year	25-year	50-year	100-year
Beaver Creek, [2]	108	230	460	630	870	1050	1250
Chewuch River, [2]	466	2890	4440	5440	6660	7530	8370
Entiat River, [3]	395	3100	4460	5390	6590	7510	8450

The annual peak discharge data from NWIS for the USGS stream gages in Table B-1 were used to estimate the frequency discharges at the remaining structure locations. First a Log-Pearson III distribution was fit to each gaged record of peak discharges using the method of moments to develop the 2-, 5-, 10-, 25-, 50-, and 100-year flood frequency values. This process is consistent with the procedure described in the Guidelines for Determining Flood Flow Frequency, *Bulletin 17B* [4]. A Regional skew value was included in the calculations for each gage using the guidelines and figures contained in *Bulletin 17B*.

Using the state specific guidelines in the National Streamflow Statistics (NSS) Program [5], the frequency discharges computed for the USGS gages were used to estimate the frequency discharges for the appropriate structure location. The following expression was used to estimate the frequency discharges at the ungaged structure locations,

$$Q_u = Q_g \left(\frac{A_u}{A_g} \right)^b ,$$

where Q_u is the frequency discharge, in (ft^3/s), at the structure location for a specific recurrence interval, Q_g is the peak discharge, in (ft^3/s), at the gaged site for a specific recurrence interval, A_u is the contributing drainage area, in mi^2, at the structure location, A_g is the contributing drainage area, in mi^2, at the gaged site, and, *b* is the regional exponent specified by the NSS documentation for the state of interest. Table B-3 contains the frequency discharge estimates for the 2-, 5-, 10-, 25-, 50-, and 100-year floods.

Table B-3. River Spanning Rock Structure frequency discharges.

Structure Location	Drainage Area (mi^2)	Frequency Discharge (ft^3/s)					
		2-year	5-year	10-year	25-year	50-year	100-year
Bear Creek	72	960	1280	1500	1770	1970	2170
Catherine Creek	111	780	1050	1220	1430	1580	1730
East Fork of the Salmon River	286	970	1500	1880	2400	2800	3220
Salmon River	235	1630	2120	2440	2820	3100	3370
Grande Ronde	461	2370	3650	4550	5760	6710	7660
Lemhi River	1231	940	1640	2180	2960	3600	4280
Little Snake River at USGS 09253000	285	2130	2990	3530	4170	4620	5040
Rio Blanco	104	1280	1820	2300	2720	3130	3560
San Juan River	281	2580	4300	5700	7780	9580	11600

2.3 RECURRENCE INTERVAL AND MAGNITUDE SINCE CONSTRUCTION:

The River Spanning Rock Structure Research required the estimation of recurrence interval and magnitude of the largest flood since construction at each rock structure. In order to accomplish this, the flood frequency results were compared with peak discharges experienced by each structure since its construction year. First, the date of the largest discharge was determined; second, the magnitude of the largest flood was estimated from available mean daily discharges for the corresponding date; finally, the flood frequency analysis at each location was used to estimate the recurrence interval in years of the flood magnitude. Table B-4 contains the estimated recurrence interval and magnitude of the largest flood since construction of each rock structure.

Table B-4. River Spanning Rock Structure recurrence interval and magnitude since construction.

Structure Location	Earliest Potential Construction Year	Date of Largest Discharge between Contruction Year and Site Visit	Magnitude of Discharge (ft^3/s)	Estimated Recurrence Interval of Discharge (years)
Bear Creek	1999	5/30/2003	2200	100
Beaver Creek	2000	5/19/2006	690	25
Catherine Creek	1998	5/30/2003	1900	>100
Chewuch River	2007	5/17/2007	2800	2
East Fork of the Salmon River	1998	5/21/2006	2500	30
Salmon River	2006	5/21/2008	1800	25
Entiat River, Structures 3.1, 3.2, and 4.6	2001	5/19/2006	4700	5
Entiat River, Structure 3.4	2006	6/4/2007	3600	~2
Entiat River, Structure 5.1	2007	5/19/2008	3400	~2
Grande Ronde	1998	6/16/1999	3200	3.5
Lemhi	2002	5/31/2003	1300	3
Middle Fork of the Little Snake River	2001	5/31/2003	nodata	3
North Fork of the Little Snake River	2001	5/31/2003	nodata	3
Middle and North Forks of the Little Snake River	2001	5/31/2003	nodata	3
South Fork of the Little Snake River	2001	5/31/2003	nodata	3
Rio Blanco	1999	5/23/2005	2300	3
San Juan River	1995	5/23/2005	4700	6.5

The structures on the upper forks of the Snake River do not have discharge estimates of the largest flood since construction. The ‘nodata’ entry in Table B-4 implies that there was not sufficient mean daily discharge data at the Little Snake River structure locations to provide a reasonable estimate of flood magnitude. The date of the largest flood for the Little Snake River structures following construction but prior to the 2005 field visit was associated with the 2003 annual peak discharge (2,490 ft^3/s) from USGS gage 09253000. According to the FFA for this gage, the 5/31/2003 discharge has a recurrence interval of approximately 3 years (Table B-4). Because USGS gage 09253000 is downstream of the confluence of the middle, north, and south forks of the Little Snake River, there is no way to reasonably determine how much of the 2008 discharge is contributed by each fork.

3. DATA LIMITATIONS

The following documents the limitations associated with the data and results presented in this appendix. The level of detail required for this study did not warrant extensive data quality checking for the annual peak discharge and mean daily discharge data downloaded from the NWIS website. Recurrence intervals beyond 100 years were not computed at USGS gages and were not estimated at structure locations because the uncertainty of the results would be too large. The frequency discharges computed at the USGS gages are available upon request. The frequency discharges estimated at the structure locations shall not be used for the design of new structures or modification of existing structures. The purpose of the FFA was only to provide an estimate of recurrence interval for the largest flood since construction. As additional peak discharge data is available from NWIS, the FFA should be updated in order to provide the best possible frequency discharges at the structure locations.

4. REFERENCES

In Text	Complete Citation
[1]	USGS National Water Information System (NWIS): Web Interface; http://waterdata.usgs.gov/usa/nwis/sw
[2]	Sutley, David E. July 2006. *Methow River Drainage Basin, Hydrology Data and GIS for the Methow River, In-Stream Habitat Restoration Project*, Bureau of Reclamation, Technical Service Center, Denver, CO.
[3]	Sutley, David E. January 2009. *Appendix B-Hydrology Data and GIS, Entiat River Tributary Assessment*, Bureau of Reclamation, Technical Service Center, Denver, CO.
[4]	United States Water Resources Council. 1981. *Guidelines for Determining Flood Flow Frequency, Bulletin #17B of the Hydrology Committee*, U.S. Department of the Interior.
[5]	USGS National Streamflow Statistics (NSS) Program: Regional Regression Equation Publications by State; http://water.usgs.gov/osw/programs/nss/pubs.html#wa

Appendix C: Ranges of Parameter Values

Table 1. Mean, min, and maximum values for parameters of particular interest categorized by degree of failure.

Structure type	Category		2yr Discharge (cfs)	General Structure Spacing (river widths)	Max arm Length / Structure Width	Max Plan Angle	Max Scour (ft)	Max Structure Arm Profile Slope (ft/ft)	Open angle A (deg)	Recurrence Interval of High Flow	RSRS Spacing (river widths)	Scour Depth / Scour Offset	Scour Depth A (ft)	Structure Opening (ft)	Structure Width (ft)	Thalweg Slope (ft/ft)	Throat width (ft)	Tie in length (ft)	Tie in length / Structure Width
U,V-weir	All Structures	max	3100.0	12.3	1.7	61.9	4.7	0.2	99.1	100.0	12.3	1.0	4.7	103.4	140.4	0.0	39.5	34.8	0.6
		min	230.0	0.5	0.2	13.8	1.0	0.0	20.7	2.0	0.5	0.1	1.0	19.1	13.1	0.0	0.0	0.0	0.0
		mean	1671.9	3.1	0.8	35.7	2.8	0.1	56.7	18.5	3.4	0.4	2.8	41.8	52.9	0.0	10.5	4.1	0.1
	No Failure	max	2130.0	4.0	1.2	31.9	3.9	0.2	59.1	3.0	5.6	0.6	3.9	45.5	69.1	0.0	15.3	34.8	0.6
		min	2130.0	0.9	0.5	13.8	2.1	0.0	20.7	3.0	1.5	0.2	2.1	19.7	15.2	0.0	7.4	0.0	0.0
		mean	2130.0	2.3	0.9	24.2	2.9	0.1	39.3	3.0	3.2	0.4	2.9	30.4	43.4	0.0	10.6	14.5	0.2
	Partial Failure	max	3100.0	12.3	1.7	60.3	4.5	0.2	99.1	25.0	12.3	1.0	4.5	92.7	104.7	0.0	20.7	31.7	0.5
		min	230.0	0.5	0.3	13.9	1.0	0.0	28.0	2.0	0.5	0.1	1.0	19.8	13.1	0.0	0.0	0.0	0.0
		mean	1456.7	3.8	0.9	38.9	2.5	0.1	57.1	9.4	3.9	0.4	2.5	35.0	41.9	0.0	8.4	2.6	0.0
	Failure	max	3100.0	7.0	1.6	61.9	4.7	0.2	94.1	100.0	7.0	0.9	4.7	103.4	140.4	0.0	39.5	17.8	0.3
		min	230.0	0.5	0.2	18.8	1.4	0.0	33.8	3.0	0.5	0.2	1.4	19.1	14.2	0.0	0.0	0.0	0.0
		mean	1637.2	2.9	0.8	38.6	3.1	0.1	63.3	31.1	3.1	0.4	3.1	50.7	63.9	0.0	11.9	1.0	0.0
Asymmetric U-weir	All Structures	max	2130.0	11.4	1.4	52.6	4.3	0.2	79.4	3.0	11.4	1.3	4.3	42.9	107.9	0.0	9.5	22.1	0.5
		min	2130.0	0.7	0.2	13.9	1.3	0.0	29.9	3.0	0.7	0.2	1.3	18.4	13.0	0.0	3.1	0.0	0.0
		mean	2130.0	4.0	0.9	35.9	2.5	0.1	57.7	3.0	4.6	0.5	2.5	27.4	37.8	0.0	7.1	4.2	0.1
	No Failure	max	2130.0	11.4	1.2	36.4	2.3	0.1	65.8	3.0	11.4	0.7	2.3	31.3	107.9	0.0	7.3	4.2	0.0
		min	2130.0	0.7	0.3	32.9	1.4	0.1	45.8	3.0	0.7	0.4	1.4	20.4	15.0	0.0	5.6	0.0	0.0
		mean	2130.0	6.4	0.8	34.4	1.9	0.1	56.6	3.0	6.4	0.5	1.9	24.8	46.1	0.0	6.5	1.4	0.0
	Partial Failure	max	2130.0	8.3	1.4	49.7	4.3	0.2	72.5	3.0	9.4	1.3	4.3	42.9	80.3	0.0	9.5	22.1	0.5
		min	2130.0	1.1	0.5	13.9	1.3	0.0	29.9	3.0	1.3	0.2	1.3	19.6	13.0	0.0	6.0	0.0	0.0
		mean	2130.0	3.9	1.0	32.9	3.0	0.1	53.9	3.0	4.8	0.5	3.0	29.5	30.8	0.0	7.8	5.0	0.1
	Failure	max	2130.0	6.3	1.2	52.6	3.2	0.1	79.4	3.0	6.3	1.3	3.2	34.6	95.1	0.0	8.6	20.0	0.2
		min	2130.0	0.9	0.2	29.1	1.4	0.0	52.0	3.0	0.9	0.2	1.4	18.4	15.8	0.0	3.1	0.0	0.0
		mean	2130.0	2.6	0.6	39.4	2.2	0.1	62.2	3.0	3.1	0.6	2.2	26.6	47.5	0.0	6.3	5.1	0.1
A-weir	All Structures	max	3400.0	6.4	2.6	49.4	5.5	0.1	71.1	30.0	6.4	1.3	4.6	134.5	136.8	0.0	24.3	29.7	0.4
		min	940.0	0.7	0.6	8.7	2.0	0.0	29.2	2.0	0.7	0.1	1.3	28.7	26.2	0.0	5.2	0.0	0.0
		mean	1694.0	3.5	1.2	27.6	3.3	0.0	43.0	9.2	3.8	0.5	2.5	62.9	70.3	0.0	12.9	8.8	0.1
	Partial Failure	max	2130.0	6.4	2.6	49.4	4.6	0.1	68.3	10.0	6.4	1.3	4.6	73.8	69.1	0.0	21.8	29.2	0.4
		min	1280.0	1.2	0.6	14.2	2.4	0.0	29.2	3.0	2.2	0.2	1.6	28.7	26.2	0.0	6.6	0.0	0.0
		mean	1421.7	3.5	1.3	23.3	3.1	0.0	38.1	8.8	3.8	0.7	2.8	41.5	42.2	0.0	11.2	6.1	0.1
	Failure	max	3100.0	0.9	0.9	45.0	5.5	0.0	48.2	30.0	0.9	0.3	2.2	92.4	136.8	0.0	24.3	29.7	0.3
		min	940.0	0.9	0.8	8.7	2.0	0.0	38.0	2.0	0.9	0.1	1.3	66.5	85.8	0.0	15.7	0.0	0.0
		mean	1670.0	#DIV/0!	0.8	31.5	3.5	0.0	43.5	12.3	#DIV/0!	0.2	1.9	81.6	112.7	0.0	19.0	9.9	0.1
Red Hill U-weir	All Structures	max	N/A	4.0	1.5	28.4	3.7	442.2	48.5	#N/A	5.8	0.9	2.8	35.8	32.6	0.0	17.2	20.1	0.8
		min	N/A	2.2	0.8	13.5	1.8	0.1	0.0	#N/A	2.6	0.0	1.2	31.2	22.8	0.0	9.6	0.0	0.0
		mean	N/A	3.1	1.2	20.2	3.0	269.6	34.8	#N/A	3.5	0.4	2.1	33.3	27.9	0.0	13.6	13.0	0.5

Black = All Structures, Blue = No Failure, Green = Partial Failure, Red = Failure