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Managing Water in the West

Rock Weir Design Guidance

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Rock Weir Design Guidance

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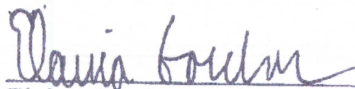Michael Sixta, Hydraulic Engineer

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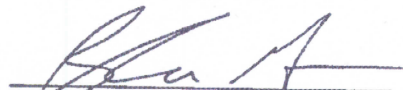

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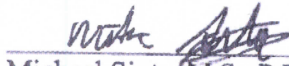

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1 Introduction

River spanning loose-rock structures provide increased hydraulic head for irrigation diversion, permit fish passage over barriers, protect banks, stabilize degrading channels, activate side channels, reconnect floodplains, and create in-channel habitat. These structures are called by a variety of names including rock weirs, alphabet (U-, A-, V-, W-) weirs, J-hooks, and rock ramps. Rock ramps are not incorporated into this guideline as a separate guideline was completed for these structures in Reclamation, 2009a. The structures included in this guideline are termed rock weirs and share these common characteristics:

- loose rock construction materials (individually placed or dumped rocks with little or no concrete),

- span the width of the river channel, and

- cause an abrupt change in the water surface elevation at low flows.

River spanning loose-rock structures also share common performance objectives. Performance objectives include the ability to withstand high flow events and preserve functionality over a range of flow conditions. Functionality is often measured by a structure's ability to maintain upstream water surface elevation and/or downstream pool depths. In meeting specific performance objectives, common design criteria must also be met, often related to vertical drop height, lateral constriction, size of rock material, or construction methods.

This report documents numerous years of research and data collection findings and offers guidance for future designs of rock weirs. The primary purpose of this guideline is to provide quantitative techniques for investigating the conditions of a system and selecting parameters needed for the design of a rock weir. Many sections of this report were tailored to address specific questions of Reclamation designers. The organization of this guideline is as follows:

Chapter 1: Introduction

Chapter 2: Background

This chapter covers background information on the ongoing research initiated by Reclamation in 2005. The chapter summarizes three components of the research, including the field, physical modeling, and numerical modeling efforts.

Chapter 3: Hydrology

Within this chapter, several considerations for determining the flows of importance in the design of a rock weir are presented. In addition, a brief investigation of gage records and flows typically responsible for structure failures is presented.

Chapter 4: Geomorphology

This chapter discusses the importance of a geomorphic evaluation of a system prior to selection of a rock weir as an appropriate tool in river rehabilitation efforts. In addition, the chapter covers key data collection and analysis techniques where rock weirs are being considered as part of a river design effort. The final part of this chapter presents a detailed example of where a geomorphic investigation prior to rock weir installation would have been useful.

Chapter 5: Hydraulics of Rock Weirs

This chapter describes the hydraulic properties of rock weirs using results from physical and numerical models of rock weirs in the laboratory and field.

Chapter 6: Sedimentation and Scour

The types of questions, data, and analyses that might be necessary to develop a sustainable design are presented in Chapter 6. Details of the research conducted for development of scour prediction equations are also introduced in this chapter.

Chapter 7: Design Guidance

Recommendations related to design of rock weirs are offered in Chapter 7. These include guidance related to back water modeling, scour depth prediction approach, structure geometry, rock sizing, structure spacing, footer design, and notches in weirs. The chapter also discusses long-term maintenance issues and expectations.

Chapter 8: Additional Considerations

Within Chapter 8, additional considerations for the design of rock weirs are briefly described, including habitat for key fish species, incorporation of rootwads into structure design, and uncertainty and risk analysis.

Chapter 9: References

2 Background

The use of in-stream structures for habitat and stream restoration dates back to the early 1900's; however, the design, effectiveness, and performance of these types of structures have not been well documented. A review of international literature on grade control structure design by Nagato (1998) found that no official standard guidelines for designing low-head drop structures exist. He found that design guidelines were relatively tentative or provisional and site specific in nature. While recently there has been a large amount of laboratory data and empirical relationships developed, efforts to link these relationships with field engineering practices are lacking. Roni et al. (2002) reported that the lack of design guidance stems from limited information on the effectiveness of various habitat restoration techniques.

Monitoring of in-stream restoration projects has focused primarily on whether structures produce the desired physical response. Understanding the physical hydraulic and sediment processes that underlie the desired outcomes has not been a priority. Cox (2005) found that available guidelines and literature related to rock weirs were scarce and consistently lacked investigation of hydraulic effects and/or performance. Restoration projects that have been thoroughly evaluated and provide some insight into structure effectiveness, or lack thereof, have been highly debated within the scientific community (Frissell and Nawa 1992; Kondolf 1995; Kauffman et al. 1997; Reeves et al. 1991; Schmetterling and Pierce 1999; Wohl et al. 2005). Roni et al. (2002) found that reported failure rates for various types of boulder structures were highly variable, ranging from 0 to 76 percent. These researchers state that the conflicting results are probably due to differences in definitions of "failure" and/or "function," structure age and type, and design and placement methods. While general monitoring of in-stream restoration projects provides some information pertaining to success and failure rates, they usually do not provide sufficiently detailed information to determine the physical processes associated with the success or failure of a given structure geometry. The lack of detailed analysis of failure mechanisms is part of the reason why current design methods are based upon anecdotal information applicable to narrow ranges of channel conditions. Methods and standards based upon predictable engineering and hydraulic performance criteria currently do not exist.

In 2005, the Bureau of Reclamation initiated a program to evaluate the performance of these structures and develop design guidelines using a multi-faceted approach that consists of field reconnaissance, physical modeling, and computer modeling (Figure 2.1).

Field reconnaissance provides long term performance data under actual conditions, including how different river processes affect the structures and how the structures in turn affect river processes. Physical laboratory modeling provides

information under carefully controlled conditions that isolate one or more variables to test the impact of specific changes on structure performance. Computer (numerical) models provide a cost effective method for evaluation of a range of structure geometries and channel conditions to develop a more complete understanding of structure performance and optimize structure design. Integration of field, lab, and numerical data sets provides a scientific basis for predicting structure performance under various river conditions and for developing the most-effective design criteria. A brief description of each component of the research conducted toward the development of this guideline is summarized below and discussed in more detail in subsequent chapters. Results from the field analysis, laboratory testing, and numerical modeling are combined to provide specific design guidance related to river spanning rock structures.
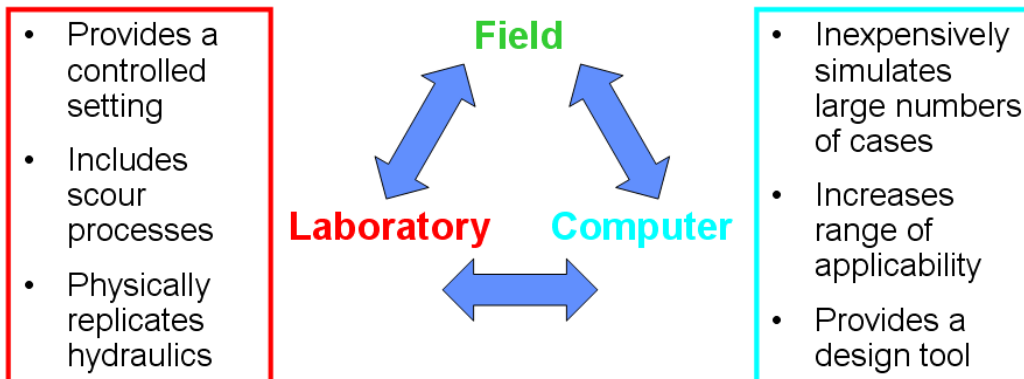
Figure 2.1. – Multifaceted approach of rock weir research incorporating mutually supporting field, laboratory, and numerical studies.

Rock weir configurations are highly variable and depend on site characteristics and project goals. This study focuses primarily on A-weirs, U-weirs, and W-weirs. The letter of the weir type indicates the general shape of each weir. An A-weir typically has a double drop to dissipate energy longitudinally through the structure into two scour pools and is shaped in the form of the letter A. A U-weir, often referred to as a V-weir when the throat width is small, has a single drop through the structure, converges flow to the center of the channel, and is shaped in the form of the letter U. A W-weir is often used in wider channels or where energy dissipation across and longitudinally through the channel is necessary and is shaped in the form of the letter W. Detailed descriptions and diagrams of each weir type are presented in Section 5.1.

2.1 Field Component

Two evaluations were conducted for the field component of the research. The first included a qualitative evaluation of rock weir field performance (Reclamation, 2007) and the second consisted of a quantitative investigation of factors leading to the success or failure of rock weirs (Reclamation, 2009b). Within this section, major findings from the field component are summarized.

2.1.1 Qualitative Investigations

The qualitative evaluation of rock weir field performance included a literature review of existing rock weir guidelines and identification of common failure mechanisms for sites visited across the Western United States. Site visits to existing installations provided critical information for identifying processes that influence the structure's ability to meet management goals. At each site, attempts were made to identify processes influencing each structure's performance, thereby permitting a scientific evaluation of the controlling parameters and potential techniques for modifying structure designs. A major focus of the research was on each structure's ability to maintain upstream water surface elevation and/or downstream pool depths. Therefore, degrees of failure were assigned to each structure visited based upon departures from original designs and shifting of rocks within the structure.

127 structures were evaluated as part of the initial investigation. At each site, survey data was collected and each structure's condition was documented. Structures were assigned a degree of failure (no failure, partial failure, or full failure) and a hypothesized primary and secondary failure mechanism based upon field observations (Table 2.1). Over 70 percent of the structures were determined to have at least partially failed. The most common failure mechanism was the growth of the scour pool and subsequent slumping of the footer rocks. Sliding and rolling of the surface (header) rocks was observed as the primary failure mechanism for only 5 percent of the sites where complete or partial failures were identified. Sliding and rolling had a greater impact as a secondary failure mechanism, most often following growth of the scour pool and slumping of the foundation rocks (footers). Field observations suggest that most structures were comprised of rocks adequately sized to prevent failure by sliding and rolling caused by hydraulic forces for flows they have experienced to date.

The qualitative investigations identified system effects, such as filling of the structures from sediment deposition and lateral channel migration, as potential contributors to structure failures. Impacts of these processes on structure function identify the need for a greater understanding of sediment transport, geomorphology, and physical processes encountered by these types of structures. Observations also indicated that structures tied-in (keyed-in) to the bank showed greater structure stability. Structures that were tied into point bars rather than into the bank were more susceptible to flanking by downstream migration of the gravel bar.

Table 2.1 – 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 undersized rock material due to hydraulic forces.
Filling and Burying	Substantial filling downstream of the scour pool resulting in no defined scour pool downstream of the structure.
General Bank Migration/Flanking	Channel 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 difference between the upstream and downstream water surface elevations.

Several installation and design methods were identified to enhance structure performance. The most successful technique observed was the presence of a deep foundation for the structure, which prevented scour from undermining the footer rocks. Structures in series may increase the likelihood of maintaining project objectives over longer periods of time. In general, interlocking blocky rocks and grout increased resistance to failure and allowed the structure to sustain more damage before losing function. Implementation of these techniques may increase the longevity of the structures, but do not guarantee permanent structure function.

2.1.2 Quantitative Investigations

Field measurements and topographic surveys collected during qualitative field investigations were applied to 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 may be linked with the qualitative findings to 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 as part of the qualitative investigations, topographic surveys were performed on 76 river-spanning loose rock structures between June 2005, and October 2008. Sixty-nine of these structures had sufficiently consistent data to be included in the quantitative analysis. Structures surveyed were defined as A-weirs, U-weirs, Asymmetrical U-weirs (one arm substantially longer than the other), W-weirs, and VW-weirs (U-weir having a small throat combined with a W-weir across one channel width). 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 the highest flows the structure experienced, throat width, planform angles, and scour offset from structure. In addition, structure spacing, planform location and scour depth are potentially important variables that relate to structure performance, but clear relationships were difficult to discern during 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 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 mid-way along the structure arms, which is where the hydraulic head drop over the structure is greater than near the throat. This is also the location where the footer protection is less than at the throat, and where 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 less along the structure arms, where the greatest head drop over the structure occurs, than it is closer to the structure throat, where the head drop is smallest.

Structure spacing was also found to be an important parameter to consider for structure design. Structures that were more closely spaced tended to have greater success rates than those spaced farther apart. However, structures spaced too closely may result in increased potential for failure. Multiple structures in sequence tended to outperform individual structures, likely because the energy dissipation across a single structure was generally less. Asymmetrical U-weirs surveyed in one river system (Little Snake River, CO) failed more frequently as the structure spacing decreased because the development of a scour pool may have influenced the stability of the subsequent downstream structure.

The foundation depth of the structure rocks is a critical component in protecting against failure by scour. However, this study could not detect a strong statistical relationship between scour depth and failure for the sites included. This is likely due to the inability to accurately measure 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 was another key variable observed in the field that must be considered 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 and river bank conditions 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 is coupled with the results from the laboratory and numerical modeling components to better 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.

2.2 Physical Modeling Component

Rock weirs can provide energy dissipation, create and enhance aquatic habitat, allow fish passage, relieve pressure on failing banks, establish grade control, and function as water diversions. Information on the design and performance of these structures is largely anecdotal and based on empirical professional experience without the engineering rigor required for transferability. Site specific design requires a substantial design effort that in some cases can consume more resources than construction. Over the course of the last 3 decades, the use of rock weir structures has spread from use as habitat restoration and fish passage as part of salmon recovery projects in the Pacific Northwest, to examination of alternatives to bank stabilization as part of the Middle Rio Grande River Maintenance Project in New Mexico. Little systematic information is available on how well these structures meet project goals. Realizing the need for more qualitative assessment of the function of various structures, a laboratory test program was designed and implemented. The in-stream rock structures research

was intended to streamline the design process, increase understanding of the performance of river spanning rock structures, and improve the chances for successfully meeting management objectives. The laboratory physical modeling formed part of a multifaceted approach by incorporating a quantitative evaluation of field data to provide empirical validation, laboratory physical modeling for controlled testing of complicated processes, and numerical simulations to expand the range of applicability and analytic capability. The end products of the interstitial flow, local hydraulics, and scour quantification process related structure performance to parameters identifiable through field measurements and numerical hydraulic modeling. Results from the laboratory tests assisted in the development of more robust design or retrofits of river spanning rock structures based upon predictable engineering and hydraulic performance criteria.

Physical testing facilities in the hydraulic laboratories at Colorado State University (CSU) were utilized to investigate downstream scour development on the design, installation, function, and maintenance of rock weirs. The Hydraulics Laboratory at CSU's Engineering Research Center (ERC) in Fort Collins examined the downstream bed scour and structural stability in various rock weir structures. The physical modeling component provided information for the development of sustainable construction and retrofit techniques, minimizing the structure lifecycle costs and maximizing their ability to reliably meet management objectives.

2.2.1 Rock Weir Physical Modeling

Previous testing and field investigations identified development of a downstream scour pool as the dominant failure mechanism for rock weir structures. Existing methods do not accurately predict scour or capture the hydraulic processes controlling growth of scour holes and undermining of rock weirs. Physical modeling facilities at CSU's ERC Hydraulics Laboratory were used to test a near prototype range of weir configurations and bed material sizes to determine scour magnitudes and trends, measure hydraulic conditions associated with structure type, and assess the stability of each weir configuration.

Physical modeling was conducted on three types of in-stream rock structures: U-weirs (similar to V-weirs but with a throat), W-weirs, and A- weirs (also called a double drop). The physical testing program was developed by CSU, in conjunction with Reclamation, to investigate the hydraulics specific to the three weir types and the associated bed scour development. For all three rock weir types, physical tests were performed using consistent methodology to document and improve understanding of how the structures function up to a state of instability, to add to the existing data set and to expand the range of applicability of rock weirs.

Meneghetti (2009) presented stage/discharge curves for estimating the water surface elevation upstream of U-, W-, and A-weirs based on measured results from the CSU physical modeling effort. Thornton et al. (2011) proposed a new set of stage/discharge equations based on the same physical model data that may be

more applicable over a wider range of discharges and weir geometries. Both sets of equations give similar results over the range of flows and types of structures tested in the hydraulics lab at CSU.

Starting from the D'Agostino and Ferro (2004) approach for predicting local scour downstream of grade control structures, Scurlock (2009) used the physical test data to calibrate the coefficients in the D'Agostino and Ferro equations for rock weirs. The range of applicability of the equations developed by Scurlock may be limited to the flows and weir geometries tested at CSU. Application of the Scurlock equations to field data over predicted the measured and observed scour depths downstream of rock weirs. In their 2009 report, Reclamation achieved greater success applying the Scurlock (2009) form of the scour equations with the original D'Agostino and Ferro (2004) coefficients to a limited number of field scour measurements. A re-evaluation of the laboratory data performed by Reclamation developed simplified equations that may be more applicable to a broader range of field conditions than those evaluated in the laboratory (Section 6.4.2.2).

2.3 Numerical Modeling Component

The complex flow patterns and resulting performance of rock weirs are not well understood. Methods and standards based upon predictable engineering and hydraulic performance criteria are very limited. Without accurate hydraulic performance criteria, designers cannot address the failure mechanisms of structures. There are no specific hydraulic guidelines for rock weirs; field work alone cannot quantify and capture detailed processes, and physical modeling is expensive and time intensive.

Collecting enough detailed field and laboratory data to include a wide range of design parameters (structure geometry, grain sizes, channel characteristics, etc.) and performing an analysis of structure performance would be costly and take decades to accomplish. To address the paucity of design guidelines and logistical challenges of empirical modeling, the numerical modeling research examines the applicability of One-dimensional (1D), Two-dimensional (2D), and Three-dimensional (3D) models to simulate the complex flow patterns associated with numerous river spanning rock structure configurations. Numerical modeling provides a design tool for analyzing how hydraulics in a channel are affected by changes in channel geometry, flow rate, and the presence of structures (weirs, culverts, bridges). The type of numerical model used in an analysis must capture significant flow patterns and replicate the important processes.

Due to the lack of reliable design guidance for river spanning rock weirs, a numerical model testing matrix was developed to investigate the physical processes associated with river spanning rock weirs and how changes in structure geometry affect the local hydraulics within and around the structures. The testing matrix includes a U-weir with varying structure geometries and channel

characteristics (bed slope, discharge, and grain size) to investigate how local flow patterns are influenced by variations in structure geometry associated with river spanning rock weirs. In order to understand how these types of structures impact the local hydraulics, an analysis of a wide range of structure geometries were tested. Chapter 4 describes in more detail the numerical model testing matrix design used in the research project.

The results from the numerical modeling provide assistance to designers in selecting the appropriate numerical modeling method based on its applicability, limitations, and ability to meet project goals. Each of the numerical models (1D, 2D, and 3D) provides varying degrees of information related to the hydraulics and local flow patterns associated with various river spanning rock structure designs. Results from the numerical modeling provides designers various methods for estimating changes in water surface elevations, local flow patterns (e.g., high/low velocity zones, eddies, bed shear stress), and scour associated with varying structure geometries. Depending on the complexity of the site and overall project goals, designers can use the numerical modeling results to compare and select the structure geometry that provides the best hydraulics and local flow conditions for meeting their specific project needs.

3 Hydrology

3.1 Flows of Importance

The intended purpose of an in-stream structure will help determine the type of hydrologic analysis necessary for design. Typically, the ability of a structure to perform its intended function and meet management objectives over an entire range of flows should be evaluated. If a structure is intended for diversion, structural component stability is paramount, and a high flow analysis will be most important in the stability assessment. However, it is also important to maintain upstream pools at or above the minimum level required for diversion at low flows. If a structure is to be used primarily for fish habitat, stability may be less important with the low flow analysis taking precedence. This is to ensure useable habitat exists at low flows along with sufficient depth and velocity for fish passage. Low flow analyses may also be important to ensure habitat features (e.g., pools, notches, and chutes) are not filled or buried during the receding limb of a flow hydrograph. The effects of high flows should still be considered in the design to minimize damage occurring to habitat features during flood events. If fish passage is the primary objective, low flow and high flow analyses should be performed to ensure structural stability to maintain passable hydraulics at all flows.

Design of any river spanning rock structure must consider its performance under both high and low flow conditions. While high flow designs focus on structural stability and upstream flooding impacts, low flow designs are constrained by fish passage criteria and maintenance of diversion capability.

3.1.1 Low Flow Analysis

Determining the low flows of greatest importance to a structure design again depends on the purpose of the structure and the indigenous fish species present in the subject stream. When used as diversions, the required upstream water surface elevation providing the necessary hydraulic head at all flows is a critical design component. For ramps with a continuous crest spanning the entire channel and common weir shapes (U-Weir, A-Weir, W-Weir), rating curves and head loss equations have been developed to ensure upstream water diversion over the range of design flows.

Successful design for low flows requires careful consideration of physical and political constraints. Meeting velocity fish passage criteria for high flows presents a challenge. For low flows, these velocities at small depths can be particularly complicated to incorporate into a rock weir design, as flows are concentrated toward the center of the channel. Conflicting guidance for fish passage by species, organization, and by State can further confound compliant design.

3.1.2 High Flow Analysis

Since most structures observed have failed after a large flow event has occurred, it seems rather intuitive that a high flow analysis is an imperative component of a structure's design. However, determining which high flows are of greatest importance could be subject to debate, depending on the structure's purpose (fish passage, irrigation diversion, pool habitat, etc.) and on the goals of the design (no tolerance for mobilization of structure versus dynamic constituent rocks). For most structures intended to be stable over the long-term, designs typically need to consider two critical pieces of information in the hydrologic analysis. First, these structures must be designed using the discharge that is responsible for the maximum scour immediately downstream from the structure. Second, most structures are required, through regulatory means, to be designed such that the added feature does not impact the water surface elevations at a specific discharge (e.g. 50-year or 100-year floodplain). Section 7.4.3 presents a method for determining the flow responsible for the maximum scour using a one-dimensional model.

3.2 Spacing of Structures

Structure spacing is an important parameter to consider for structure design; however, meeting the high flow and low flow design criteria for structures in sequence may be challenging. The spacing of structures is highly dependent upon the goal of the project. Structures in sequence reduce the energy dissipation experienced by a single structure and create redundancy in the design. Multiple structures seem most appropriate when the objective of the project is irrigation diversion or fish passage. However, if the project objective is to create a large pool volume for holding habitat, structures closely spaced in sequence may limit the maximum pool volume attainable.

Early field evidence suggests that structures were most stable when placed in series rather than individually. Field observations suggest multiple hypotheses for why this is true. First, structures in series provide redundancy for meeting management objectives. For example, the probability of success for structures used to prevent channel incision may increase through the placement of multiple structures. If the downstream-most structure fails, upstream structures will continue to provide grade control. A second hypothesis for why several structures in sequence are more stable than independent structures is that the difference in water surface elevations on the upstream and downstream sides of one independent, large structure may create sufficient backwater pressure (potential energy) to instigate structure failure. To produce the same head, several structures in series distribute the energy dissipation and may increase the potential for structure success and longevity.

Structure spacing design varies according to the purpose/intent of the structures and typically requires both low flow and high flow analysis as described in the previous sections. Structure spacing depends on channel slope, length of

backwater effects created by downstream structures and associated depth, and length of the scour pool created downstream. Low flow analysis takes into account the effects that multiple structures in series might have on diversion and fish passage criteria. Structures placed too far apart can result in failure to meet the required tailwater depths for fish passage. Structures placed too close together can result in downstream structures being influenced by the hydraulics of the upstream structure, potentially limiting pool/scour development and structure failures due to flow impingement on the downstream structure crest. Additionally, at high flows, structures spaced too close together can result in an increase in the water surface elevation above allowed Federal Emergency Management Agency (FEMA) flood flow levels due to an overall increase in the channel roughness compared to the natural channel conditions.

3.3 Correlation between Peak Flow and Degree of Structure Failure

Structure failure is influenced by the magnitude of flows experienced since construction. A hydrologic analysis was conducted as part of a quantitative investigation (Reclamation, 2009b) 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 with topographic surveys.

First, the nearest USGS stream gages were identified for the structures surveyed during field investigations. Estimates of discharges with 2-, 5-, 10-, 25-, 50-, and 100-year recurrence intervals were calculated at 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 construction was developed for each structure location.

Table 3.1 indicates the recurrence interval experienced and the degree of failure documented at each site visited as part of the field research conducted (Reclamation, 2009b). Determination of structure success or failure was 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 Reclamation's field study, 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.

The recurrence interval of the highest flows that each structure has been subject to since construction was found to be a good indication of the degree of structure failure. Based on the hydrologic analysis, 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. From the qualitative analysis (Reclamation, 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 the perhaps more common assumption 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.

Table 3.1 – River spanning rock structure recurrence interval and magnitude since construction.

Structure Location	Earliest Potential Construction Year	Date of Largest Discharge between Construction Year and Site Visit	Magnitude of Discharge (ft^3/s)	Estimated Recurrence Interval of Discharge (years)	Number of Structures Surveyed	Degree of Failure Experienced
Bear Creek	1999	5/30/2003	2200	100	6	6 full failures
Beaver Creek	2000	5/19/2006	690	25	3	2 partial failures and 1 full failure
Catherine Creek	1998	5/30/2003	1900	>100	5	3 partial failures and 2 full failures
Chewuch River	2007	5/17/2007	2800	2	2	1 no failure* and 1 partial failure*
East Fork of the Salmon River	1998	5/21/2006	2500	30	3	1 partial failure* and 2 full failure
Salmon River	2006	5/21/2008	1800	25	2	1 no failure* and 1 partial failure*
Entiat River, Structures 3.1, 3.2, and 4.6	2001	5/19/2006	4700	5	3	3 full failures
Entiat River, Structure 3.4	2006	6/4/2007	3600	~2	1	1 partial failure
Entiat River, Structure 5.1	2007	5/19/2008	3400	~2	1	1 no failure
Grande Ronde	1998	6/16/1999	3200	3.5	2	2 full failures
Lemhi	2002	5/31/2003	1300	3	4	3 partial failures and 1 full failure
Middle Fork of the Little Snake River	2001	5/31/2003	No data	3	14	5 partial failures, 4 full failures, and 5 no failures
North Fork of the Little Snake River	2001	5/31/2003	No data	3	7	3 partial failures and 4 full failures
South Fork of the Little Snake River	2001	5/31/2003	No data	3	25	9 partial failures, 11 full failures, and 5 no failures
Rio Blanco	1999	5/23/2005	2300	3	9	8 partial failures, 1 full failure
San Juan River	1995	5/23/2005	4700	6.5	4	3 full failures, 1 no failure

4 Importance of Geomorphology in Design of Rock Weirs

River spanning rock structures, including rock weirs, are often installed in rivers to address a localized problem within a system on the order of tens to thousands of feet in length. They may also be installed for diversion purposes. As a result of the small scale implementation of these structures to "fix" a problem or improve the condition of a specific reach of river, the processes acting at larger scales within the system that may be impacting the reach are commonly not evaluated. This may lead to a structure not functioning as originally intended or to a complete failure of the structure and a worsening of the condition meant to be improved upon. Understanding the geomorphology of a river reach and processes controlling it will increase the likelihood for successful design and implementation of rock weirs at meeting their intended objective. The purpose of this section is to (1) describe the importance of a basic geomorphic understanding of a river landscape prior to the selection of a rock weir as the tool of choice for a river rehabilitation project and (2) define the types of geomorphic data and analyses that may be required to reduce potential for structure failure and negative impacts to the river system.

Geomorphology is the scientific study of the formation, alteration, and configuration of landforms, including the depositional and erosional processes that affect these landforms. Through these studies, geomorphologists are able to understand more about the physical environment and the processes responsible for its formation. For stream restoration efforts, a detailed understanding of the geologic evolution of the river system is essential to augment the hydrology, hydraulic conditions, and sediment movement used in design of specific structures.

The purpose of a geomorphic investigation is generally to (1) determine the areal distribution and physical characteristics of the various surficial deposits, (2) reconstruct the general fluvial history of the area, (3) conduct stratigraphic studies for the purpose of correlating various environmental settings and understanding the geomorphic processes responsible for their formation, and (4) provide a scientific basis in support of proposed channel modifications and design.

Recent research of rock weir success and failure rates and mechanisms (Reclamation, 2009b) has suggested the importance of involvement of a trained geomorphologist in every river rehabilitation effort to identify key processes important to a given system, what may have instigated the issue that needs resolve, how installation of a rock weir may influence the river processes, and how those processes may impact the structure's success and sustainability. An experienced geomorphologist with an earth science background can be an

important asset to advise the design team to reasonable assumptions and estimates about conditions at a particular site. This is particularly critical in many alluvial settings where lateral migration, aggradation and/or channel incision may be considered an issue. This expertise is also essential for identifying geomorphic processes or underlying geologic conditions that may be exerting otherwise unrecognized control on the river system and associated environments.

As a first step in determining if a rock weir is appropriate for a specific reach of river, a field visit and consultation with a trained geomorphologist is necessary. The geomorphologist may be able to use some background data on the geological history of the region in combination with a field visit to better shape the needed analyses for each river and for each rehabilitation objective (i.e. increased pools, improve incised channel condition, improved fish passage). For most investigations, geomorphic mapping of fluvial landforms and their associated stratigraphic units, including the soils, should be an initial component to help guide subsequent studies. These geomorphic maps are most useful in combination with hydraulics and sediment data to better understand the evolution of the system and dominant processes controlling the present form of the river. The extent of the mapping efforts vary widely and often include an initial evaluation of the areal distribution of surficial deposits and related landforms determined from topographic maps, aerial photography, and other similar types of data. Comparisons of historical and recent aerial photography combined with LiDAR surveys provide insight on natural channel change and are useful for documenting ground disturbance from anthropogenic modifications. Initial geomorphic studies are typically reconnaissance in nature and require a thorough review of available information to cross check the accuracy of geologic interpretations. More detailed studies may be justified once a site is evaluated and better understood.

Geomorphic mapping can often be supplemented with pertinent data from geological publications, water supply papers, bulletins, technical reports and historical data from a variety of Federal and State agencies, including Departments of Transportation, the U.S. Geological Survey and state geological surveys, and numerous private geotechnical engineering firms. Each geomorphic feature or environment represented by particular deposits (e.g., point bars, terraces, floodplains, alluvial fans) within a particular project area can be mapped. Thin deposits on the surface such as alluvial fans/colluvial aprons and natural levees should also be identified. These landforms and the deposits can provide important insight into processes acting in a specific reach. Lastly, the underlying geology and its structural control that might influence the river form or development can be identified and mapped.

Major swales or point bars should be identified and mapped to show the trends in the migration of meanders. Analyses of alluvial fans can aid in identifying the sequence of deposition and impacts on channel slope, form, and position. Similarly, within any given river valley, a sequence of terraces of multiple ages may represent different periods of incision or aggradation during the stream's evolution. Understanding the timing of the incision and deposition and how such

features relate to perceptions of instability of the river system is important. The basic geomorphic model can be vital for developing a better understanding of potential change and to help guide restoration efforts. Many failures of projects intended to resolve real or perceived problems can often be directly related to poor understanding of the system and local conditions at project sites.

Unfortunately, reviews of existing records and readily available literature describing conditions at a site may be among the most limited methods for evaluating and understanding the geomorphology and processes at a specific location. In a complex fluvial environment, this type of background study cannot be substituted for adequate field investigations and data collection. Change in a fluvial system is often characterized by very rapid and complex adjustments over short distances, both vertically and laterally. The combination of a reconnaissance field inspection of a site with a limited number of observations (both visual and systematic) separated by a period of time may lead to misinterpretations and are subject to any other analysis based on a limited sample set.

Data needs that can lead to a useful understanding of continuity and conditions at any particular site may include:

- Soil pits or other exposures (road cuts, quarries, borrow pits, bank exposures, animal burrows, etc.) that provide information on the stratigraphy that comprise the deposits and landforms

- Geomorphic maps of surficial deposits, geomorphic features, and their relationships to the bedrock and structural elements of the bedrock that may control geomorphic processes

- Aerial photographs (from the earliest available to the most recent in 5-10 year increments if available)

- Topographic Maps (e.g., 7-1/2 USGS topographic quadrangles)

- Geologic maps of surficial geology or soil surveys (USGS, NRCS)

- Academic reports, theses, and guidebooks from field trips

- Historical photographs and maps (often found in local museums or historical societies)

- LiDAR imagery (allows for better evaluations and interpretations at heavily vegetated sites)

- Bed material grain-size distribution data and any sediment data or reports

- Information regarding locations of bank erosion and bank erosion rates (aerial photography can often be used to make estimates)

- Recent and historical hydrology information, including documentation through observations and/or photographs of significant floods or droughts

- Information on the major tributaries and their contributions of flow and sediment

- Documentation or evidence of artificial channel modification and other anthropogenic impacts (i.e., roads, bridges, railroads, dams, diversions, and other irrigation infrastructure).

The character and evolution of floodplain deposits can provide essential clues useful for interpreting material properties and continuity. Floodplains are formed by a complex interaction of processes governed by the hydrology and the character of the sediment as well as other influences such as vegetation and hydraulic geometry. The deposits can range from coarse-grained to fine-grained environments, each with unique geomorphological features. Understanding and defining the range of expected environments for a particular site helps form the basis of important interpretations and judgment that are not possible without field studies and data collection. In a strictly stratigraphic sense, just being able to map the different landforms allows the geomorphologist to predict subsurface stratigraphic relationships and gain a better understanding of erosional and deposition processes throughout the sequence of events responsible for landform development.

The required geomorphic data collection and analyses will vary from one project to another. For river spanning rock structure designs, the scope of the geomorphic analysis is typically based upon the risk of failure to nearby property and infrastructure, desired longevity and level of maintenance, and available budget. River rehabilitation projects typically have a limited budget for analysis, design, and implementation. Managers are often dissatisfied when the analysis and design cost is greater than the cost of project implementation. This is often the case with rock weirs due to the affordability of materials and the relatively low level of construction effort to install the structures. Understanding reach and system-scale geomorphic characteristics for a channel may not seem imperative given a limited budget. Unfortunately, the cost of failure of one of these structures may be much greater than the analysis, design, and implementation if the necessary steps are not taken to maximize the potential for structure success, particularly those related to understanding channel processes. For example, mapping out historical channel position over a 2 mile channel stretch for 5 different sets of aerial photo sets requires time and budget to obtain the photos and labor costs to georectify and delineate channel positions. However, this effort on a channel that has historically migrated across the floodplain may provide insight as to the locations where the greatest potential for lateral channel stability is to be expected and offer guidance for locations of rock structure placement to avoid failure from channel cutoffs and/or bank erosion. Research presented in a quantitative analysis of structure failures (Reclamation, 2009b) found that several of the failure modes were related to geomorphic processes, such as lateral migration or burial by sediment, rather than related to structure design. Geomorphic processes play a large role in defining the success of a river spanning rock structure in meeting its intended objective, and understanding those processes must therefore be considered an integral part of a successful design.

4.1 Example of the Need for a Geomorphic Assessment

One field study that exemplifies where a geomorphic evaluation of the river and surrounding landscape would have been beneficial prior to design and construction of a rock weir is on the East Fork Salmon River in Idaho, where an A-weir was installed to provide sufficient head for irrigation while maintaining fish passage and eliminating the need for the land owners to seasonally construct a berm using gravel from the bed of the river to direct flow to the irrigation diversion. This structure is located in a system with a high sediment load. The structure is located just downstream from a geological constriction in the channel where bedrock was noted in the bed of the channel. Just upstream from the structure, the valley changes from a more confined configuration upstream to a slightly wider valley with a low surrounding surface across which the channel likely migrated historically (in geologic time).

In July 2008, just 5 years after construction, the structure was not meeting fish passage criteria due to jump height and velocities. Five years post-installation of the rock weir, the channel downstream from the structure eroded at least one foot vertically and widened substantially (up to 10 feet) due to erosion along the right bank. The irrigation canal filled with sediment, and the land owners have returned to using a push-up berm to meet diversion requirements. Figure 4.1 through Figure 4.4 provide evidence of structure changes from construction in August of 2003 to October 2008. Although the structure was still intact during the 2008 site visit, a metal plate was exposed in several locations and some of the headers had moved into the deep scour pools (Figure 4.5). The metal plate provided a hard point in the river and strongly influenced the structure persistence. The drop over the throat was greater than 2.5 feet, and the drop over the cross bar had become greater than 4 feet.

The geomorphology of this reach of the river controls the sediment transport through the location of the rock weir. The relatively narrow reach upstream transports sediment to the wider reach in the vicinity of the irrigation diversion, where velocities are reduced and gravels and cobbles may settle. This process was evidenced through a review of ground photos at the site prior to installation of the A-weir. Prior to installation, a mid-channel gravel bar appeared to have been forming upstream from the current location of the structure between the channel and the push-up berm. During high flows, material from this gravel bar may have been mobilized and conveyed downstream along with the smaller material in the push-up berm. Detailed evaluation of historical aerial photos in this reach could help determine if the wider channel and depositional zone present today is part of the natural river process or if it was initiated by the anthropogenic activities associated with the irrigation diversion.

The hydraulics associated with the A-weir inherently induced low velocity zones along the structure arms. The installation of the structure supported the continued

development of the gravel bar along the left arm of the structure, which gained substantial rock and organic material during a 2006 high flow (Figure 4.6). This resulted in complete burial of the left arm of the structure, forcing the flow through the confined throat (with high velocities) and against the downstream opposite bank. Material settling in the gravel bar resulted in a sediment deprived condition just downstream from the structure (similar to a dam) and caused local erosion of the channel bed and banks (Figure 4.7 through Figure 4.8). Several small rills and channels have formed through the gravel bar, none of which conveys sufficient flow to mobilize the deposited gravels and cobbles.

A geomorphic evaluation of the site prior to selection of the structure type and location could have identified potential issues with this site, including a historically active channel with a high sediment load and high rates of lateral channel migration. In addition, the location of the structure was in an area of active deposition based on photographs illustrating gravel bar development in a low velocity zone.

Figure 4.1 – Looking upstream at the berm that was pushed up on a regular basis to guide flow to the irrigation diversion prior to installation of the A-weir.

Figure 4.2 – Site post construction in August 2003, looking upstream.

Figure 4.3 – July 2005. Note depositional zone upstream from structure on right side of the photo (left bank).

Figure 4.4 – October 2008. Note significant change in drop height since construction, mobilization of larger boulders, and excessive deposition upstream of structure.

Figure 4.5 – Exposed metal support along structure cross bar (October 2008).

Figure 4.6 – Looking upstream from where the left arm of the structure lies buried under large gravel bar and large woody material (October 2008).

Figure 4.7 – Looking downstream of structure in May 2005. Note initial bank erosion along right bank.

Figure 4.8 – Looking downstream of structure in October 2008. Bank erosion has resulted in destabilization of structure tie-in.

5 Hydraulics of Rock Weirs

Multiple investigations were undertaken between 2005 and 2010 to better understand hydraulics through river spanning rock structures. Field investigations included qualitative observations of hydraulics processes and quantitative measurements of water surface elevations, velocities, and physical traits of rivers and structures across the western U.S. Physical modeling of rock weirs was performed at Colorado State University, in which hydraulic properties were measured across multiple structure configurations. Numerical modeling using one-dimensional, two-dimensional and three-dimensional models was also conducted to evaluate the ability of each to replicate measured hydraulics. This chapter describes the design of the physical and numerical modeling investigations and summarizes results of comparisons of the numerical models with measured hydraulics from the laboratory and field. Limitations of each dimensional model are detailed, and recommendations are provided as to how the results can be utilized in designing river spanning rock structures.

5.1 Physical Model Design

Field investigations identified growth of the downstream scour hole and the resulting geotechnical slump as a primary failure mechanism for loose rock weir structures. Existing methods do not accurately predict scour or capture the hydraulic processes controlling growth of scour holes and undermining in rock weir structures. As a result, physical modeling facilities at Colorado State University were used to test a range of weir configurations and bed material sizes to investigate the hydraulics and scour development associated with three types of rock weirs (U-, A-, and W-weirs). The objectives for physical modeling of structures included:

- Replicate the 3D flow patterns around weir structures.

- Replicate the shape and dimensions of the scour hole downstream of the structures.

- Measure the 1D water surface profiles through the structures to develop stage-discharge relationships.

- Quantify dimensions of the bed scour downstream of rock weirs and develop scour prediction equations.

Testing included live bed scour simulations of single and double drop style weirs. Each structure was built to pre-determined design specifications with a unique geometry, bed material size, bed slope, and weir rock size (e.g. Figure 5.1). Designs were based on three different representative bed material sizes. Grain sizes were selected to match field conditions as well as minimize scaling required for model testing. Three size classes were selected using the geometric mean of the AGU classification system, a log base 2 scale:

- Small Cobble: D_{50} = 90.51 mm, D_{84} = 181.0 mm, D_{16} = 45.25 mm

- Very Coarse Gravel: D_{50} = 45.25 mm, D_{84} = 90.51 mm, D_{16} = 22.63 mm

- Coarse Gravel: D_{50} = 22.63 mm, D_{84} = 45.25 mm, D_{16} = 11.31 mm

For each bed material class, a unique bed slope and weir rock size was used. A discharge sequence of one-third bankfull, two-thirds bankfull, and bankfull was prescribed for each configuration. This sequence of three bed materials, bed slopes and weir rock sizes was completed for U-, A- and W-weirs. Data collection included bed and water-surface elevations and three-dimensional (3-D) velocities.

Figure 5.1. Example of a W-weir constructed at the CSU Hydraulics Laboratory, showing Test 50.

Weirs were designed such that the structure parameters for throat width, arm angle and arm slope were near the median of the range of values recommended by Rosgen (2001). The U-weir consists of a horizontal sill constructed perpendicular to the flow, centered in the lateral dimension and spanning one-third of the total channel width. Arms extend from each side of the sill at a 20-30 degree angle with the bank and rising upwards. Rosgen (2001) recommends that the structure intersect with the side of the channel at the overbank elevation. The contact point with the bank is higher than the sill elevation at the structure throat and should result in an arm slope between 2-7 percent. Figure 5.2 shows a conceptual design.
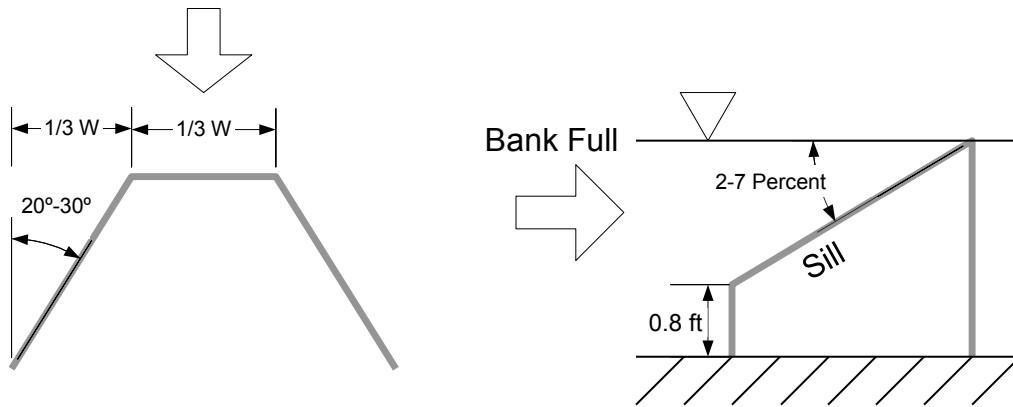
Figure 5.2– U-weir conceptual design plan view (left) and side view (right).

A W-weir consists of four sill segments with the center point facing downstream. All downstream points are higher than the upstream points. The middle point is lower than the points intersecting the bank. Figure 5.3 shows the conceptual design of a W-weir.
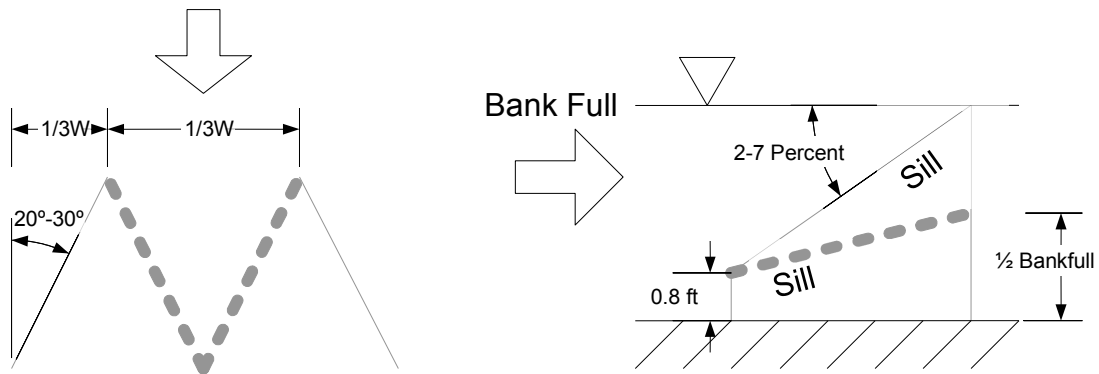
Figure 5.3 – W-weir conceptual design plan view (left) and side view (right).

An A-weir combines the U-weir design with a second horizontal sill spanning the area between the arms approximately half way through the structure. Figure 5.4 shows the conceptual design.
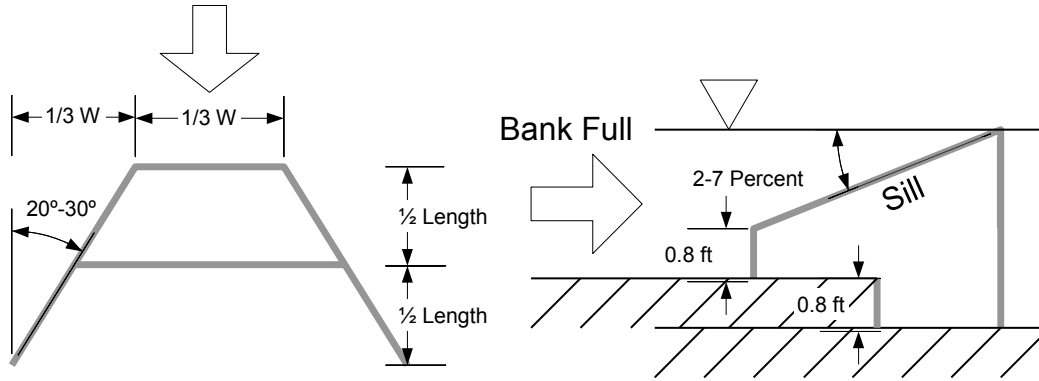
Figure 5.4 – A-weir conceptual design plan view (left) and side view (right).

Bed material grain size and the dimensions of the flume were used in an iterative process to determine the dimensions of the natural channel (prototype) and associated model scaling. Froude and Shields scaling were used to determine model scaling ratios.

A priori independent variables for physical modeling included bankfull discharge; grain size; bed slope; and bankfull width.

Fixed variables included:

- Drop Height: Fixed at a prototype drop of 0.8 feet according to criteria from the Pacific Northwest Regional Office.

- Width Partitions: equal partitioning as per Rosgen (2001) guidelines and standard field practice.

- Planform Arm Angle: Constrained by flume dimensions and Rosgen (2001) guidelines centered on 25 degrees.

- Profile Crest Angle: Constrained by flume dimensions and Rosgen (2001) guidelines centered on 4.5 percent.

- Bank Intersection Point: Set to bankfull as per Rosgen (2001) guidelines and standard field practice.

Flume limitations constrained testing to a maximum of 40 ft^3/s, 4 feet of depth, and 16 feet of width. Table 5.1 presents the design criteria of scaling ratios, grain size, discharge, weir rock size, drop height, bed slope, and channel width used in the laboratory testing.

Table 5.1 – Design parameter summary.

Parameter	Scenario 1	Scenario 2	Scenario 3
Length Scaling Ratio	1:5.75	1:4.61	1:4.36
Prototype Bed Material D_{50} (mm)	91	45	23
Model Bed Material D_{50} (mm)	15.0	9.8	5.0
Prototype Bankfull Discharge (cfs)	3178	1360	795
Model Bankfull Discharge (cfs)	40	30	20
Prototype Weir Rock Size (in)	39.5	40.5	43.6
Model Weir Rock Size (in)	6.87	8.8	10
Prototype Drop Height (ft)	0.8	0.8	0.8
Model Drop Height (ft)	0.14	0.17	0.18
Prototype Bed Slope (ft/ft)	0.0047	0.0033	0.0021
Model Bed Slope (ft/ft)	0.0047	0.0033	0.0021
Prototype Width (ft)	92	74	70
Model Width (ft)	16	16	16
Arm Angle (degrees)	25	28	30
Arm Slope (percent)	4.08	4.38	3
Prototype Throat Width (ft)	30.66	24.66	23.33
Model Throat Width (ft)	5.33	5.33	5.33

Detailed descriptions of the testing facility, model design and construction, testing procedures, and results are documented in the Mercure (2006), Meneghetti (2009) and Scurlock (2009).

5.2 Numerical Modeling

Numerical modeling provides a design tool for analyzing how hydraulics in a channel are affected by changes in channel geometry, flow rate, and the presence of structures (weirs, culverts, bridges). The type of numerical model used in an analysis must capture significant flow patterns and replicate the important processes. One-dimensional (1D) numerical simulations model downstream changes in hydraulics while neglecting vertical and lateral variation. Two-dimensional (2D) models incorporate lateral differences in velocity and water surface elevation, but neglect flow non-parallel to the stream bed. Three-dimensional (3D) modeling simulates the motion of water in all directions and most accurately captures complex flow patterns. Estimating channel hydraulics with lower dimensional methods requires understanding the impact of representing a feature with simplified methods. Flow characteristics that are not captured in 1D or 2D models such as vertical jets, recirculation, and plunging

flow associated with river spanning rock structures govern scour pool development and overall structure performance.

Current methodologies for the modeling of rock weirs utilizing 1D and 2D models revolve around manipulation of cross-section geometry, contraction and expansion loss coefficients, Manning roughness values, cross-section spacing, and cross-section survey point resolution (Cox, 2005; Scurlock, 2009; personal communication Humbles, 2009). Flow through the structures is rapidly varied and therefore violates the one-dimensional, cross-section averaged parameter assumption necessary for the direct application of a standard step methodology in determining water surface and energy profiles. Additionally, vertical velocity components in the scour hole downstream of the structure contain plunging flow which violates 2D modeling assumptions that velocity vectors are parallel to the bed. 3D numerical models capture these patterns without requiring the prior and possibly incorrect assumptions of lower order models The limited understanding of the complex flow patterns around rock weirs currently requires three-dimensional simulations. However, potential uses of and recommendations for 1D, 2D, and 3D modeling of rock weirs and the limitations of each is described in the following sections. Measured data from a field reconnaissance and laboratory experiments of a U-weir are used to compare numerical modeling methods of free surface flows associated with river spanning rock weirs.

5.2.1 Qualitative Comparison of 1D, 2D, and 3D Numerical Modeling Methods for Rock Weirs

A 3D numerical model simulation using data collected during a field reconnaissance trip to an existing U-weir on the South Fork Little Snake River near Steamboat Springs, Colorado was conducted to better understand the limitations of representing the complex flows associated with river spanning rock weirs using lower order models. Results from the 3D numerical model are presented below and used to illustrate flow patterns associated with river spanning rock weirs and describe the limitations of lower order models.

Figure 5.5 shows a U-Weir measured in the field and the corresponding water surface and velocity obtained from the 3D numerical model. In the photograph, entrained air reveals areas of high energy dissipation and the water surface draw down along the crest of the structure exhibits an area of high velocity. The 3D model captured flow features including the draw down curve, hydraulic jump, and variations in velocity. Dry areas in the photograph, such as the protruding rocks in grey in the upper left corner, match the dry areas shown with the 3D model water surface. Figure 5.5 demonstrates the capability of 3D numerical modeling to simulate field conditions and is described in more detail in Section 5.5. Three-dimensional modeling directly captures much of the physics of the flow hydraulics and can provide critical information in understanding how flow patterns are influenced by structure geometry.
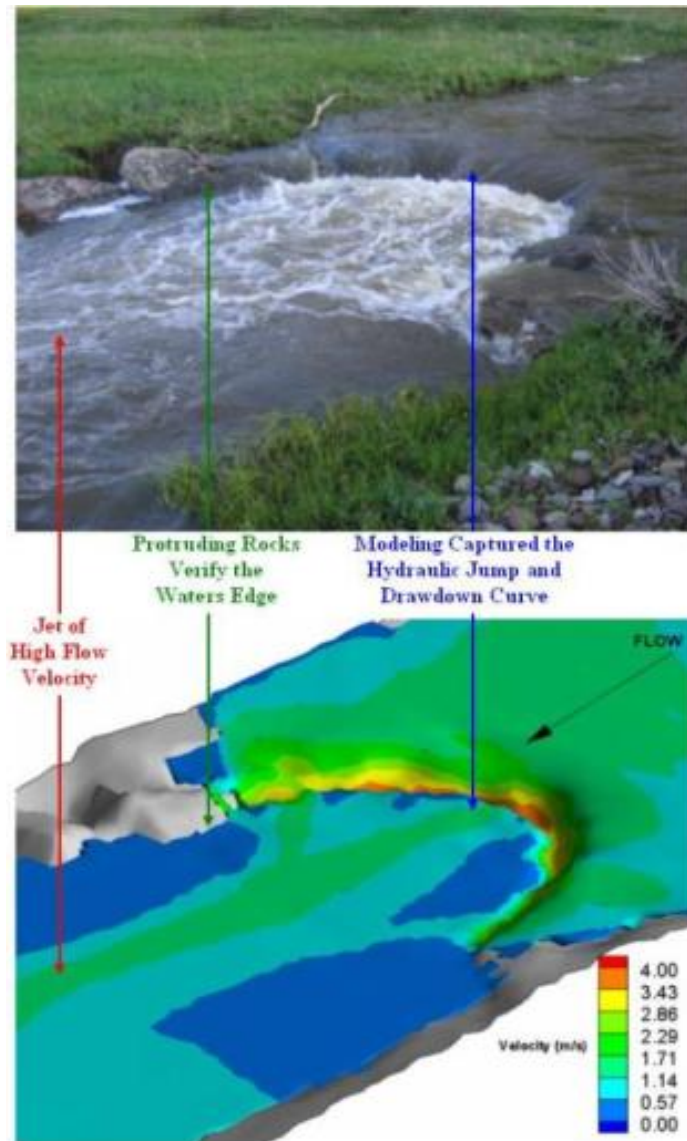
Figure 5.5 – Field photo and corresponding 3D numerical modeling results.

Figure 5.6 shows a plan view with water surface elevation contours obtained from the 3D model. Output from the 3D numerical model is used to illustrate why lower order models, 1D and 2D, are not able to properly represent the flow patterns associated with river spanning rock weirs. The areas upstream and downstream of the structure show little lateral variation. The water surface drops rapidly over the structure and follows the weir crest topology. As a result, a transect located within the structure results in multiple water surface elevations along the transect, violating 1D model assumptions of gradually varied flow and constant water surface elevation across a transect. Methods to meet 1D water surface requirements include constructing cross sections tracing water surface elevation contours or creating multiple cross sections perpendicular to the flow (Figure 5.9). Figure 5.7 shows surface velocity vectors obtained from the 3D numerical model. In the channel upstream and downstream of the structure water

flows parallel to the banks. Over the weir, the flow paths rapidly converge and then slowly expand. A jet through the center of the channel creates abrupt lateral changes in velocity. As a result, a transect located within the structure results in the velocity vectors not being perpendicular to the transect, violating 1D model assumptions for velocity. Methods to meet 1D velocity requirements include bending the cross section perpendicular to anticipated velocity vectors in order to accommodate lateral variability (Figure 5.9).
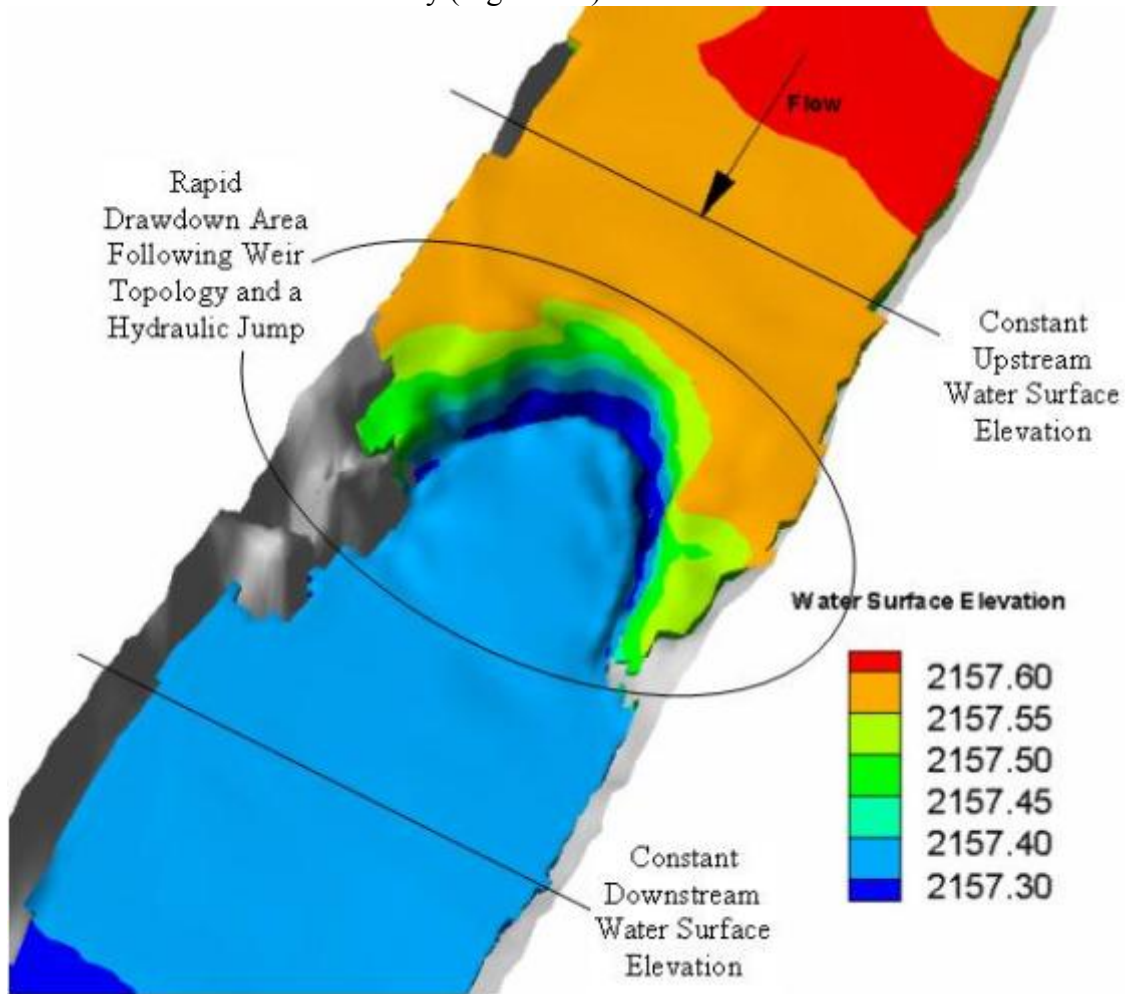
Figure 5.6 – Water surface elevation obtained from 3D model.

Velocity
Vectors Aligned
Parallel to the
Primary Flow
Direction.

Jet of Higher Velocity
with Abrupt Lateral
Velocity Changes

Area of Rapid Flow
Convergence and
Redirection

Velocity Vectors Gradually
Return to Parallel Flow.
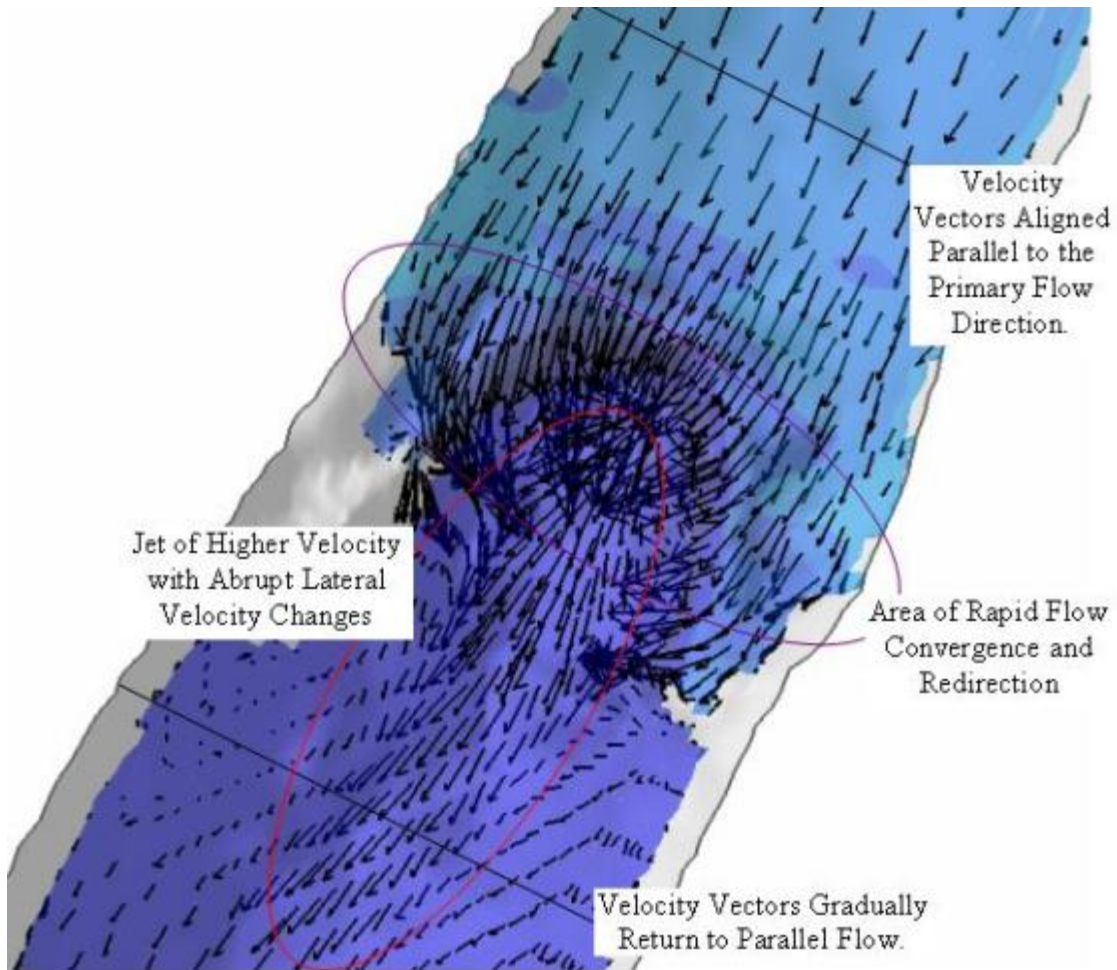
Figure 5.7 – Plan view velocity vectors and wetted area.

Figure 5.8 shows a profile view for velocities in a slice cut along the thalweg. Water flows parallel to the bed upstream and downstream of the structure. The stream lines rapidly converge and diverge vertically through the structure near the structure crest. The velocity profile contains a jet midway through the water column rather than the logarithmic profile of a typical river section. Vertical velocity components downstream of the structure crest show plunging flow. The vertical velocity components found downstream of the structure violate both 1D and 2D modeling assumptions that require velocity vectors perpendicular to the vertical plane. Additionally, while the 3D model calculates velocity vectors along the vertical, 1D and 2D models compute an average cross section and vertically depth-averaged velocity, respectively.
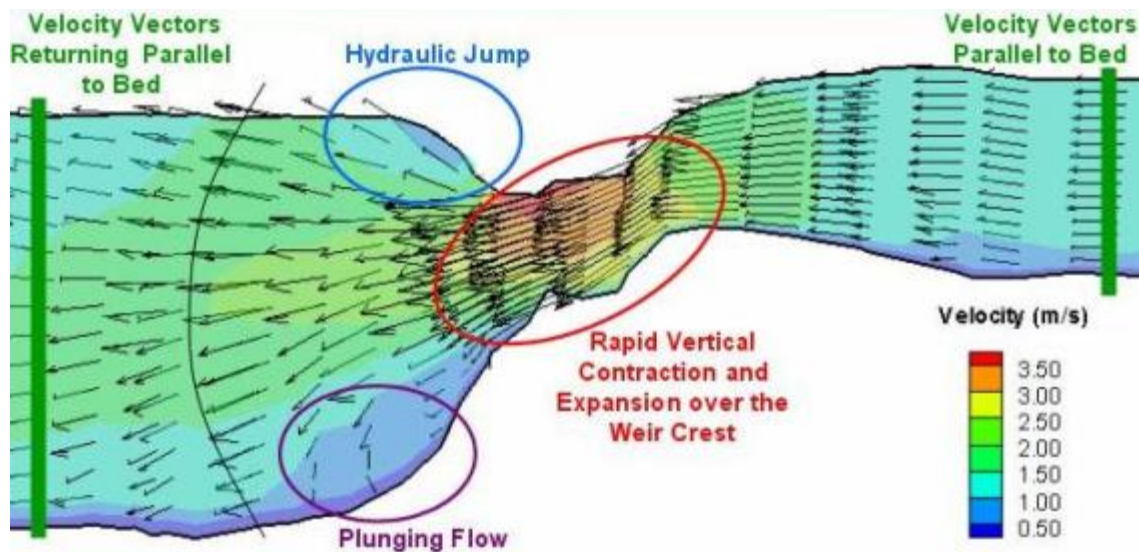
Figure 5.8 – Thalweg profile view and velocity magnitude. The location of the hydraulic jump and plunging flow may vary depending upon the flow magnitude and weir geometry.

Figure 5.9 shows attempts to reconcile 1D modeling requirements with the observed field conditions and results obtained from the 3D numerical model. A 1D cross section model for rock weirs can meet either water surface requirements or velocity requirements, but not both. Figure 5.9 demonstrates the need for a 1D model to incorporate adjustments for multi-dimensional effects. HEC-RAS contains placeholders to account for multi-dimensional effects (e.g. expansion/contractions coefficients, weir equations, roughness, ineffective flow, etc.), but the magnitudes of the adjustments are unknown for rock weirs. The adjustments will depend on the throat width, profile and plan arm angle, structure length, drop height, bed material, and more. After understanding the 3D processes associated with rock weirs, 1D adjustment parameters can be tested and developed where required. The degree of the 3D effects are flow dependent. At very high flows where the structures are completely submerged, 1D effects become dominant. Outside of the plunge pool, where there is less of a vertical flow component, 2D modeling estimates the lateral changes in flow fairly well and is described in more detail in Section 5.4 and Section 5.6.
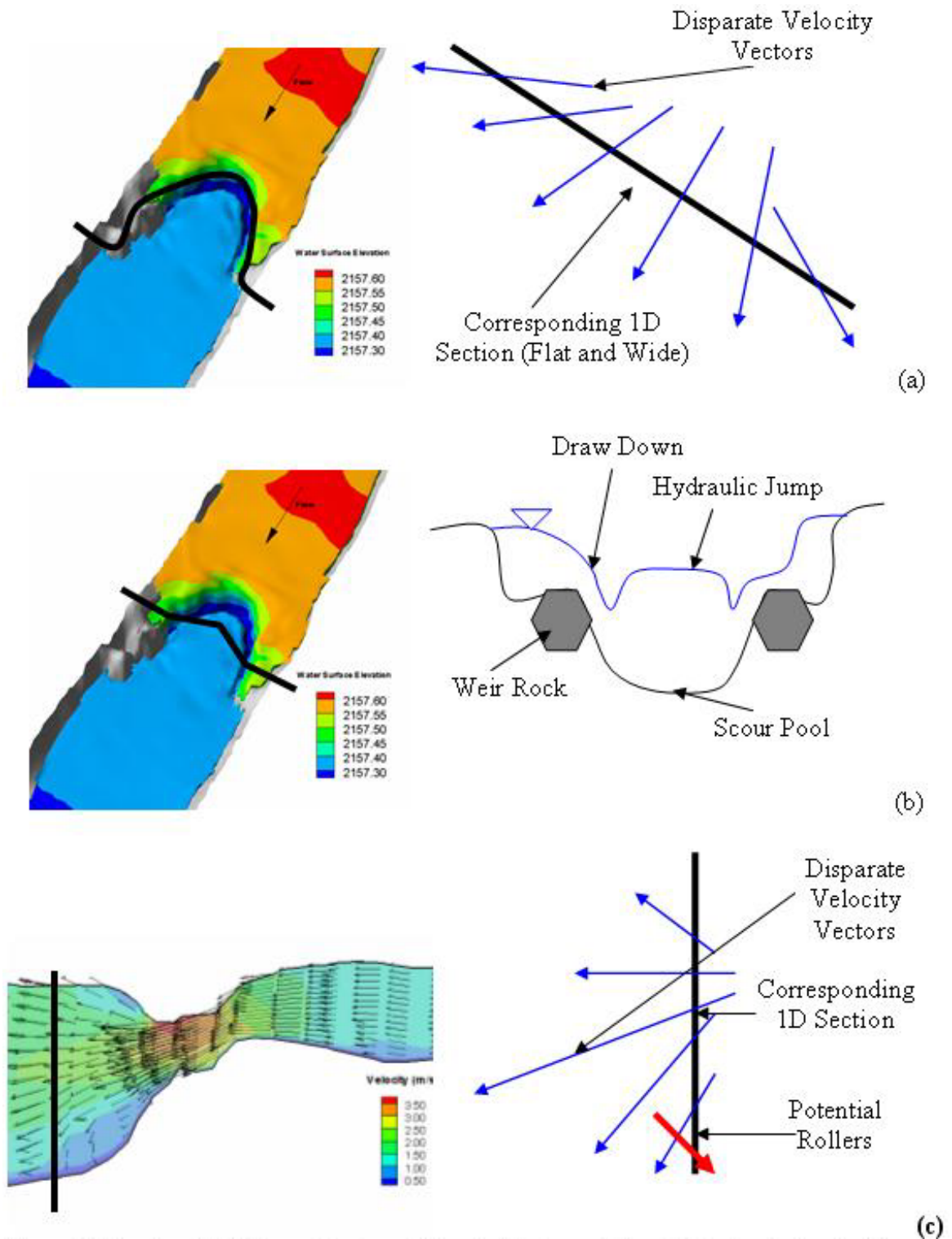
Figure 5.9 – (a) Meeting 1D water surface criteria fails to meet velocity criteria (b) and vice versa; (c) no method of manipulating 1D transects captures jumps or plunging flow.

5.3 One-Dimensional Modeling

As mentioned above, one-dimensional models operate on the fundamental assumption that velocity vectors are aligned perpendicular to a channel cross section, or parallel to the channel thalweg. Transverse and vertical velocity components are disregarded entirely and all hydraulic properties and parameters are cross-section averaged and described at the point defined by the cross section location. For non-prismatic channels, such as a channel with an installed rock weir, an iterative method is used for solving the one-dimensional, shallow water momentum equation along a user defined 1D grid demarcated with channel cross sections using a standard-step method. The following two equations are solved by an iterative procedure (the standard step method) to calculate the water surface elevation at a cross section:

$$\frac{V_2^2}{2g} + z_2 + \frac{P_2}{\rho g} = \frac{V_1^2}{2g} + z_1 + \frac{P_1}{\rho g} + h_e \qquad (5.1)$$

$$h_e = LS_f + C\left|\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right| \qquad (5.2)$$

Where V = mean cross section velocity;
g = acceleration of gravity;
z = bed elevation;
P = pressure;
ρ = fluid density;
L = reach length;
S_f = friction slope; and
C = expansion or contraction loss coefficient.

Commonly, the Manning equation is used to approximate the friction slope, which, with known channel geometry, is a function of discharge and flow depth The Manning roughness coefficient is a parameter empirically calibrated to account for roughness caused by bed material, vegetation, and channel form.

Channel bends, abrupt changes in channel geometry, hydraulic structures, vegetation, and other in-stream objects can alter velocity vectors within the channel from the thalweg direction significantly. If such features are present, the computation of the next energy head along the 1D grid will result in higher than actual values. To account for this, one-dimensional models commonly incorporate contraction/expansion losses and channel bend losses. These are simply terms added to the equations which give approximate head losses due to the departure from thalweg-oriented velocity vectors. However, such head losses may also be absorbed within the Manning roughness coefficient. Increasing the roughness value increases energy head loss along the channel and emulates features resulting in 3D velocities.

Rapidly varied flow, occurring at hydraulic structures, hydraulic jumps, or any other channel feature inciting a large degree of energy loss due to turbulence, is typically treated with empirically calibrated equations embedded in the model. One-dimensional models can resolve hydraulic jumps, but only if they occur uniformly across the channel. The hydraulic jumps that occur at rock weirs are most often localized or not uniform across the section as they tend to follow the longitudinal shape of the structure.

Applied to rock weirs, one-dimensional models are rudimentary for a variety of reasons. Rock weirs exhibit highly three-dimensional, rapidly-varied flow, not only with a hydraulic jump, but one that occurs along a channel length. The location of the jump is not set, but is dependent upon the geometry of the weir crest and discharge. Furthermore, the cross-sectional averaged assumption of one-dimensional models breaks down at the weir where relatively stagnant water on the outer arms is in contrast to the fast moving, plunging jet entering the scour pool. Thus, using a one-dimensional model to approximate hydraulics through the rock weir structure may be an overly-simplified approach. The nature of the hydraulics generated through a rock weir structure violates every basic assumption that would allow for a one-dimensional model to appropriately approximate the hydraulic conditions through the weir. However, two-dimensional and three-dimensional models are not as ubiquitous as the readily-available HEC-RAS.

Incorporation of rock weirs into HEC-RAS was investigated in two ways in order to determine the potential value and quantify the limitations of 1D modeling for design: 1) alteration of place holders in the program to increase energy loss along the channel reach encompassing the structure to emulate losses due to turbulence; and 2) incorporation of rapidly-varied flow equations tailored to specific rock-weir shapes which may be used as new boundary condition input. The former of these is important to evaluate hydraulics throughout the structure reach as the use of a rapidly-varied flow equation will bypass the structure entirely. Rapidly-varied flow equations in the form of stage-discharge relationships can be imported as a downstream boundary rating curve or internal rating curve to a cross section in HEC-RAS, treating the rock weir as a singular point or cross-section. Details related to rock weir stage-discharge equations developed from the research are presented in Section 7.2.

To evaluate the hydraulic conditions associated with rock weirs using HEC-RAS, adjustments to the program must be performed to account for the three-dimensional flow nature associated with rock weirs. Place holders for increased energy loss in HEC-RAS are in the form of the Manning roughness coefficient, contraction/expansion losses, and cross-section geometry. This method may be useful if a designer is utilizing a one-dimensional model to approximate hydraulics through the structure reach. Additionally, rapidly-varied flow equations may also be incorporated into HEC-RAS directly by employing them as downstream reach boundary conditions on a new river reach or internal rating

curves. This method is substantially more accurate in determining water surface elevations upstream of the weir crest than calculating an upstream condition by performing standard-step backwater calculations through the structure reach.

5.4 Two-Dimensional Modeling

The use of two-dimensional models to evaluate hydraulics through river spanning rock structures has not been extensively documented in available literature. To determine the ability of a two-dimensional model to adequately capture hydraulic processes through a rock weir, studies were undertaken to model a simplified rock weir using data from the CSU physical laboratory tests and through a field experiment on the South Fork Little Snake River near Steamboat Springs, Colorado. Results of how well two-dimensional models were capable of predicting hydraulics through rock weirs are presented within this section.

5.4.1 Comparison with Physical Laboratory Model

As mentioned in Chapter 1, Colorado State University performed physical modeling to examine hydraulics through rock weirs with varying discharge, slope, weir rock size, drop height, bed material grain size distribution, arm length, plan arm angle, and profile arm angle (Meneghetti, 2009). These laboratory data provided a unique opportunity to test whether the current numerical two-dimensional (2D) depth-averaged model may be applicable to predict hydraulics and subsequent scour downstream of such weirs.

The Sedimentation and River Hydraulics Two-Dimensional (SRH-2D) model was selected to model three test laboratory cases, all of which were simple U-weirs with discharge varying between one-third bankfull flow, two-thirds bankfull flow, and bankfull flow. While the 1D model only requires cross sections to represent bed topography, 2D modeling requires detailed bed topography through the entire reach modeled. High resolution LiDAR data of the topography were collected for each laboratory test case.

5.4.1.1 Water Surface Elevations

Water surface elevations derived from a two-dimensional model incorporate lateral variations in the water surface. However, two-dimensional models neglect the effects of vertical velocity, such as recirculating flow caused by a hydraulic jump or plunging flow over a river training feature, and therefore cannot adequately capture the scour and flow depths immediately downstream from a rock weir. To evaluate how well the 2D model is capable of replicating water surface elevations from experiments in the laboratory, measured water surface elevations were compared with the modeled water surface elevations. Thirty-four measurements of water surface were acquired for each model configuration in the laboratory after the majority of scour had occurred and assuming the equilibrium scour condition had been reached.

Results of the modeling (Figure 5.10) illustrate that the water surface elevations derived from the 2D model matched reasonably well with the measured values. The 2D mesh was generated with a spacing that ranged from 1.5 feet to 0.1 feet, and therefore, the bed elevations of each node differ slightly from those directly derived from the LiDAR. In addition, the 2D model was extended 18 feet upstream and 38 feet downstream of the laboratory model with a slope of 0.003 in order to provide entrance and exit boundary conditions that would not affect the models results near the test section around the structure. Despite these slight differences between the model and the laboratory, the predicted water surface elevations are typically within 0.04 feet for bank full discharge at measured depths ranging from 0.4 to 1.4 feet. All differences were within 10 percent of measured water surface elevations relative to normal depth. The greatest differences were predicted downstream from the throat of the structure at a distance approximately equal to the structure's longitudinal extent. Modeled water surface elevations upstream of the structure throat and downstream of the structure more closely matched measured values. Variations in the predicted and measured water surface elevations are likely attributable to fineness of the mesh used in the 2D model to represent the topography, an inability of the 2D model to capture turbulence just downstream from the structure, and possibly errors attributable to measurements in the laboratory.
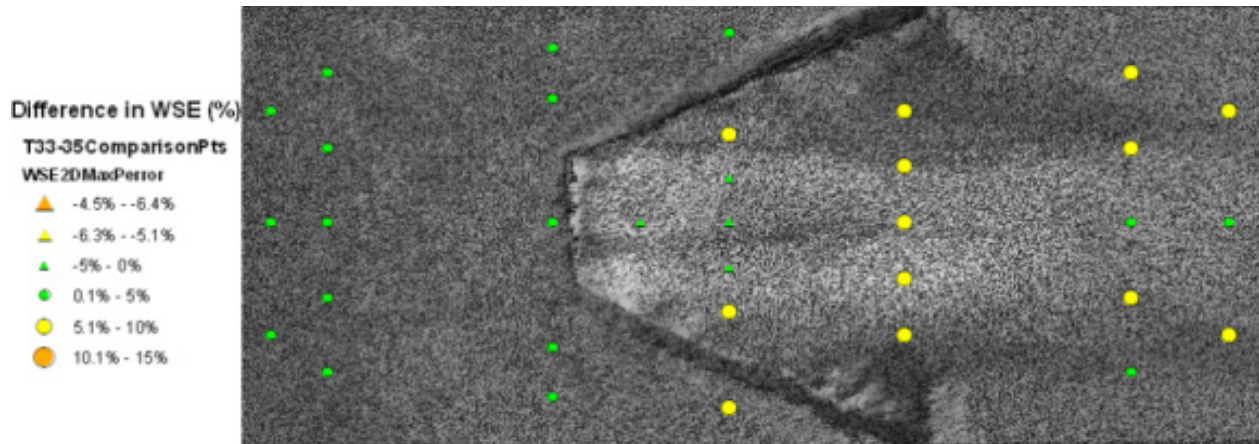
Figure 5.10 – Maximum difference in WSE between the physical lab and the 2D model. Percent differences are relative to normal depth. A negative value indicates that the physical model value was lower than the predicted 2D model result.

5.4.1.2 Depth-Averaged Velocities

Two-dimensional modeling computes depth-averaged velocities, and therefore does not incorporate the vertical component of velocity. While velocities computed using 2D models are useful for analyses of rivers without sudden topographic changes along the channel bed, depth-averaged velocities cannot adequately represent the vertical changes in velocities that occur within a close proximity to instream river features, such as low-head dams, weirs, and large woody material (LWM).

The SRH-2D fixed bed model was used to simulate the laboratory test cases with the measured scour pool topography for each test and Manning's *n* determined through the Manning's-Strickler Equation. Velocities computed with the model were compared with those measured from the point-gage instrument in the laboratory. These results (Figure 5.11) indicate that the magnitudes of the depth-averaged velocities upstream and downstream of the structure were measured within 20 percent (0.5 ft/s) for all modeled discharges. However, velocities through the structure differ by more than 40 percent (2 ft/s) at the lowest discharge modeled and just over 1 ft/s at the highest discharge modeled. The differences likely result from an inability of the 2D model to capture the three-dimensional hydraulic effects of the plunging flow over the structure crest and possibly due to measurement errors in the physical model.
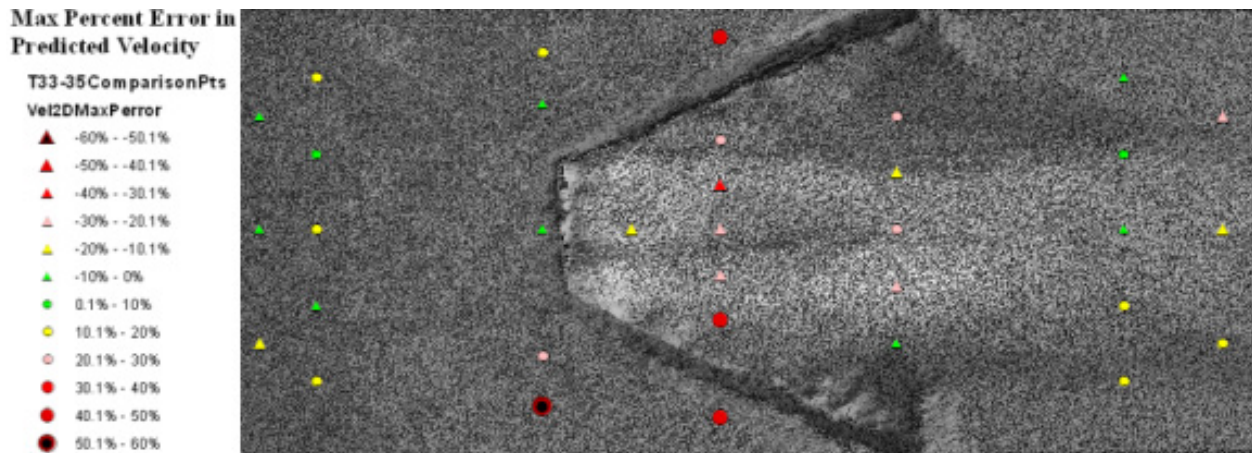
Figure 5.11 – Difference in velocities between the physical lab and the 2D model. A negative value indicates that the physical model value was lower than the predicted 2D model result.

5.4.1.3 Scour Patterns

The SRH-2D mobile bed model was also applied to investigate how closely predicted scour patterns could replicate a physical laboratory test case. SRH-2D mobile bed model was run at one-third and two-thirds bankfull discharges for the simplified U-weir. At one-third bankfull, no sediment mobilization was experienced, and measured velocities were used to calibrate the hydraulic model. Scour modeling was performed for the two-thirds bankfull discharges using the Meyer-Peter & Muller (1948) transport equation. Results from the preliminary modeling effort suggest that the 2D mobile bed model is limited in its ability to predict scour immediately downstream of the weir (Figure 5.12). This may be attributed to the fact that the flow in the immediate downstream zone is highly three-dimensional (3D) in nature. However, comparisons between the laboratory measurements and the predicted scour depths indicate that the 2D model may still be useful in estimating the scour further downstream from the structure crest, e.g., one channel width downstream of the weir. The scour that the 2D mobile bed model can predict is the scour due to the contraction of flow caused by the weir geometry and the scour caused by the deposition of sediment upstream of the

weir. Based on the study findings, it is likely that a fully 3D model is necessary to accurately predict scour patterns downstream from rock weirs.

(a) Prediction

(b) Measurement
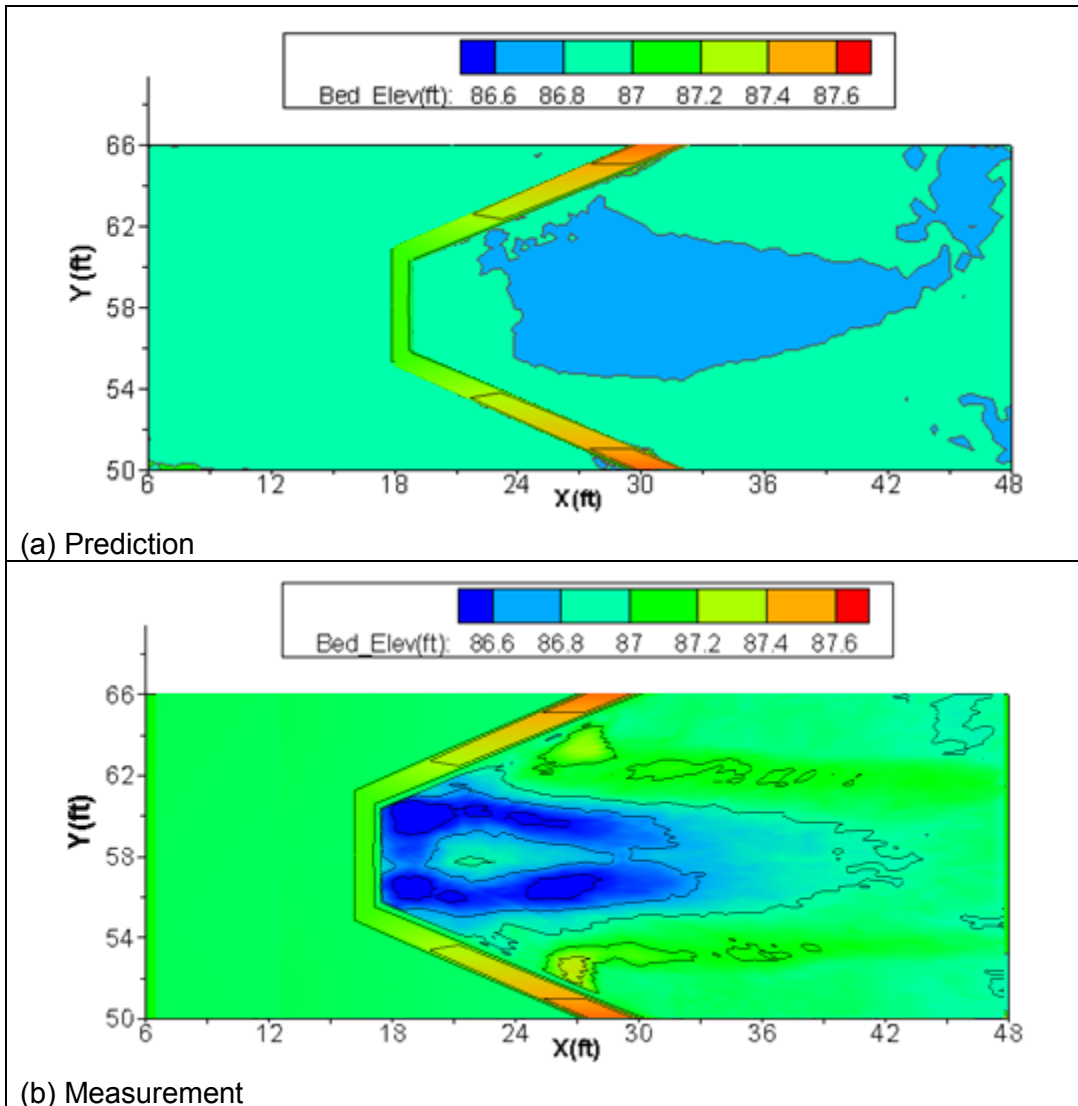
Figure 5.12 – Comparison of predicted and measured equilibrium bed topography for two-thirds bankfull discharge.

5.4.2 Limitations and Recommended Use

Two-dimensional modeling of rock weirs may be used for predicting water surface elevations through the rock weir, but will not accurately represent water surface elevations through complex hydraulic jumps associated with the structure. Computed velocities approximately one channel width upstream of the structure compare relatively well with measured velocities. Two-dimensional velocities can be used to determine the approach velocities to the structure and may be used to help size the structural components of a river spanning rock structure and possibly to predict sediment transport capacity through the structure. However, velocities just downstream of the structure do not adequately capture the vertical component

of velocity and should not be used in predictions of potential scour near the structure footers. The mobile bed modeling illustrated an inability to capture scour patterns just downstream of the structure crest, which is critical for adequately protecting the structure against failure by scour development and subsequent geotechnical slumping of the footer rocks. Two-dimensional mobile bed modeling may be able to adequately predict the longitudinal extent of the scour, but additional model testing is necessary to validate the use of the 2D model for this purpose.

Numerical modeling with any two-dimensional model can serve many purposes in the design of rock weirs, particularly those with intricate features within the reach that need to be evaluated as part of the design. However, the need for a 2D model should be evaluated on a case by case basis, and should be coupled with other design tools. Two-dimensional modeling requires detailed bed topography, which can be time-consuming and expensive to collect, and expertise in understanding the limitations for application of the model results. One area where the 2D model results should not be applied without supplemental analysis is in designing the foundation depths for river spanning rock features due to the three-dimensional nature of plunging flows over the structure crests.

5.5 Three-Dimensional Modeling

The use of three-dimensional models to evaluate hydraulics through river spanning rock structures has not been extensively documented in available literature. To determine the ability of a three-dimensional model to adequately capture hydraulic processes associated with river spanning rock structures, two test cases were modeled utilizing data from the CSU physical laboratory as well as a field experiment conducted on the South Fork Little Snake River near Steamboat Springs, Colorado. Results of how well the three-dimensional model was capable of predicting the hydraulics associated with these two structures are presented in the following sections.

5.5.1 Comparison with Physical Laboratory Model

Colorado State University performed physical modeling to examine hydraulics associated with river spanning rock structures which provides a unique opportunity to test whether a numerical three-dimensional (3D) model is capable of simulating the same hydraulic conditions.

The Bureau of Reclamation's Unsteady-Unstructured Reynolds-Averaged Navier-Stokes (U2RANS) model was selected to model a simple U-weir with discharges varying between one-third bankfull flow, two-thirds bankfull flow, and bankfull flow. Three-dimensional modeling requires detailed bed topography through the entire reach modeled as well as a vertically distributed mesh between the bed and water surface elevation in order to describe flow patterns in three dimensions. High resolution LiDAR data of the topography were collected for each laboratory test case along with measured water surface elevations and three-dimensional

velocity measurements at thirty four pre-determined locations. The 3D mesh was generated using the LiDAR data of the physical model channel bed collected after an equilibrium condition was assumed to have been reached for each test case (Figure 5.13) with a node spacing ranging from 1.5 feet to 0.1 feet in both the x- and y-direction and 0.1ft in the z-direction. The 3D model was extended 18 feet upstream and 38 feet downstream of the laboratory model with a slope of 0.003 to provide boundary conditions that were outside the influence of the structure itself.

Unlike 1D and 2D models, three-dimensional models account for vertical velocity accelerations, such as flows through a hydraulic jump or river training feature, and therefore more adequately capture the depths and associated velocities immediately downstream from a rock weir. Three-dimensional models compute velocity for each grid point control volume within the computational mesh, and therefore incorporate both the horizontal and vertical components of velocity. Verification of the 3D model was performed by comparing results from the 3D model with measured water surface elevations and velocities collected in the laboratory physical model.

Analysis of the numerical model output shows that the water surface elevations derived from the 3D model matched reasonably well with the measured values from the physical model. The predicted water surface elevations were within 10 percent for all three tests (normalized using the normal depth for the reach; n = 102 observations) with measured depths ranging from 0.2 to 1.4 feet. The greatest differences were predicted downstream from the throat of the structure at a distance approximately equal to the length of the structure's longitudinal extent (Figure 5.14).

Figure 5.15 shows the observed (true) water surface elevation measurements plotted against predicted ones from the numerical model for the same locations. The linear correlation coefficient of 0.985 confirms relatively good overall agreement (with no regard to spatial component in the data) between predicted and measured values. The residuals of the water surface elevation predictions have a slight positive bias with a mean = 0.012 ft (n = 102).

Velocities computed with the 3D model were compared with those measured from the laboratory. Figure 5.16 shows the observed (true) velocity magnitude measurements plotted against predicted velocity magnitude from the numerical model for all three tests for the same locations. The linear correlation coefficient of 0.86 confirms relatively good overall agreement (with no regard to spatial component in the data) between predicted and measured values. The residuals of the velocity predictions have a slight negative bias with a mean = -0.036 ft/s (n = 102). Results for test 35 (Figure 5.17) indicate that the velocity magnitudes compare relatively well with a mean difference of -0.42 percent (n = 34) and a maximum difference of 28 percent. The differences likely result from the three-dimensional hydraulic effects of the plunging flow over the structure crest and arms and the high turbulence encountered downstream of the structure crest that are not measured in the laboratory.
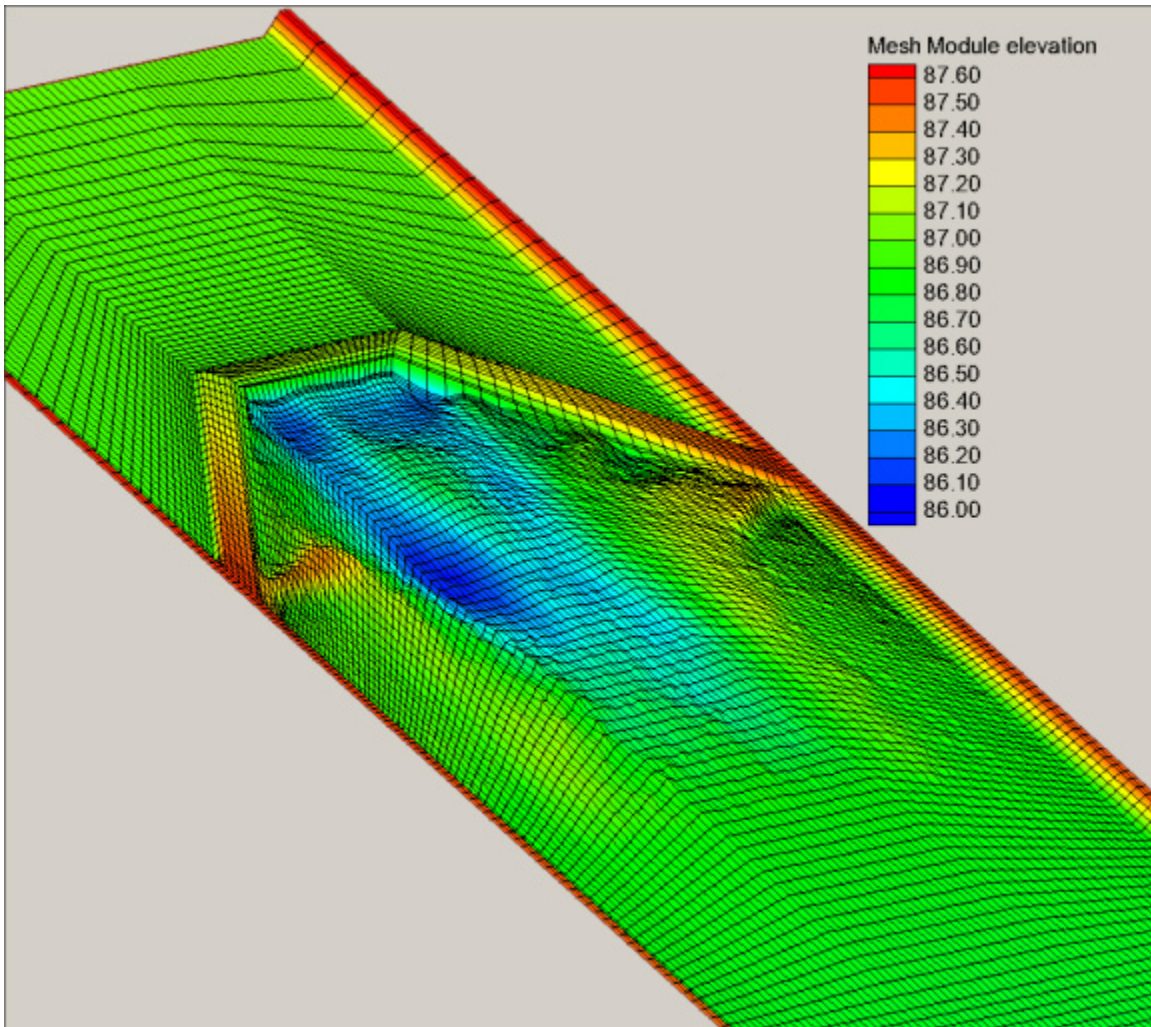
Figure 5.13 – 3D numerical model mesh representing physical model test 35. Elevations are in feet.
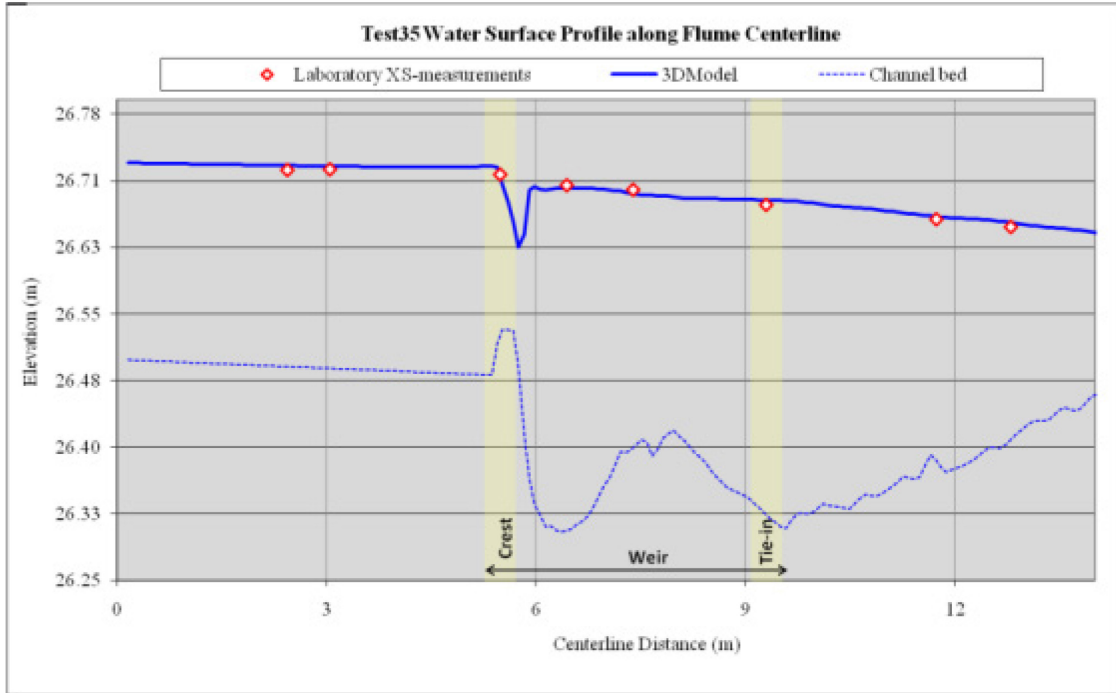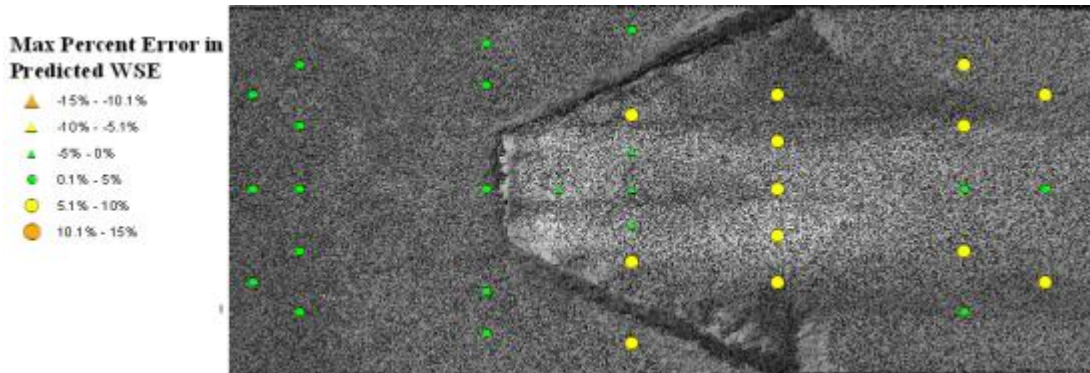
Figure 5.14 – Water surface elevation comparison: a) Test 35 centerline profile, b) Test 35 sample point locations.
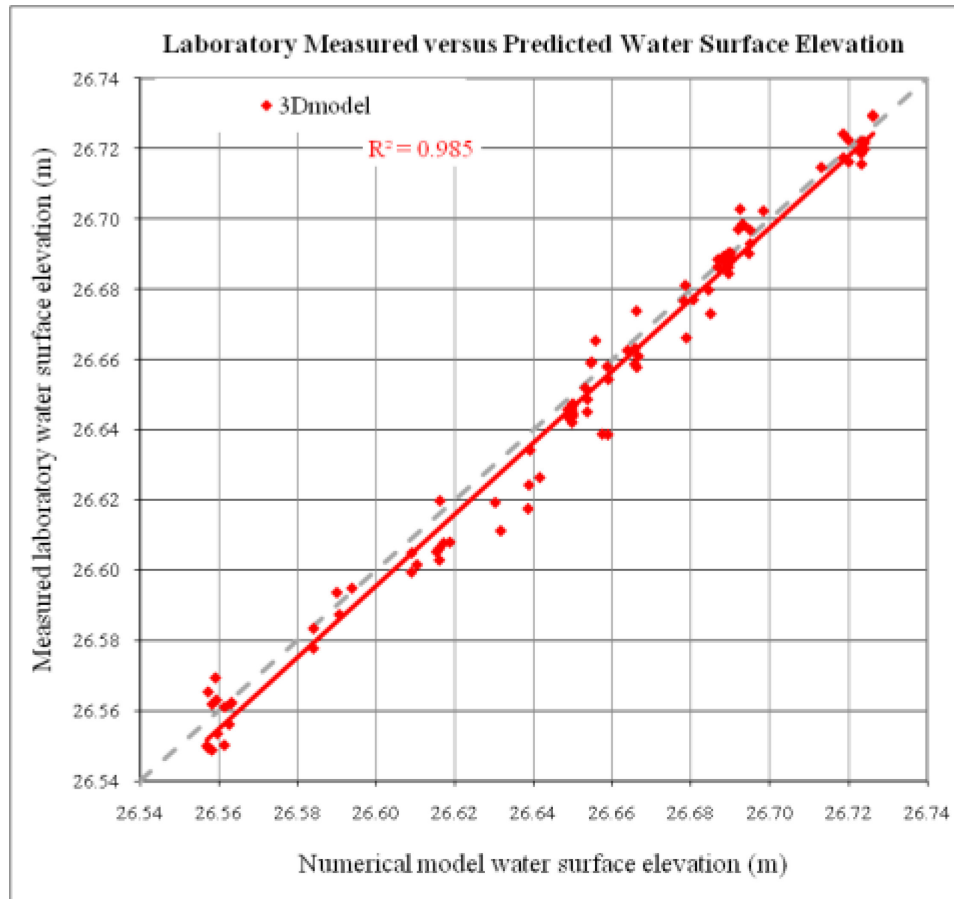
Figure 5.15 – Measured vs. predicted water surface elevations.
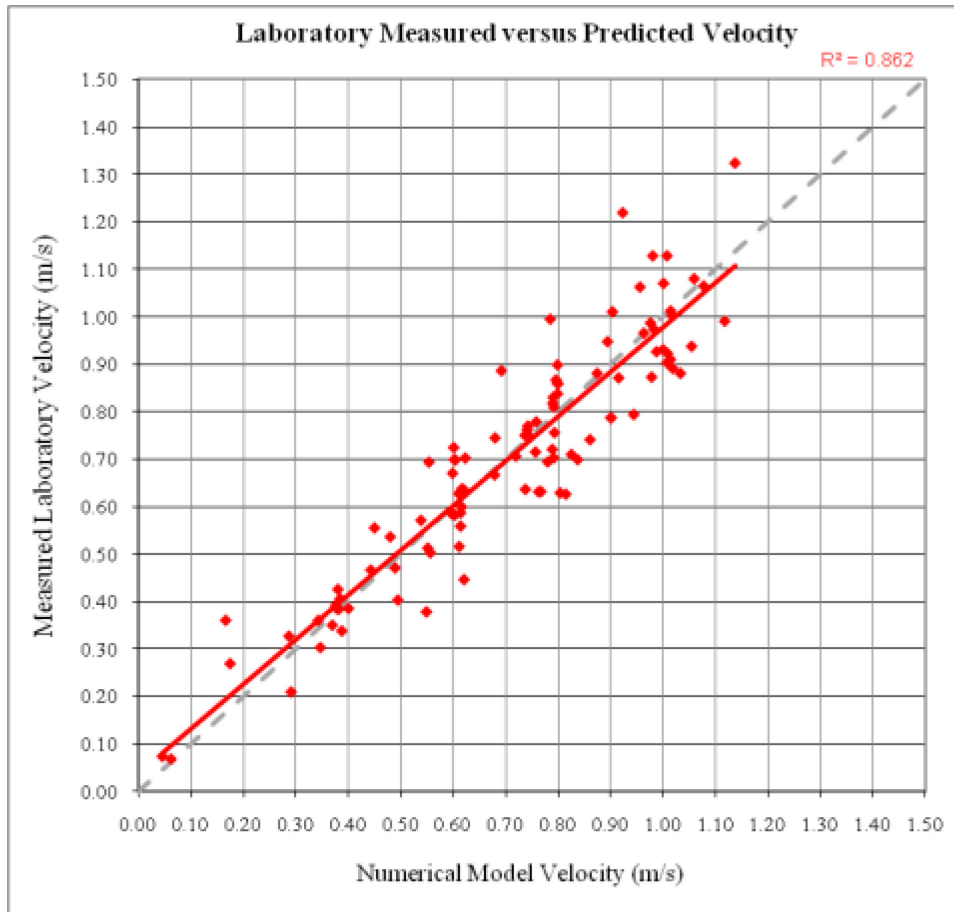
Figure 5.16 – Measured vs. predicted velocity.
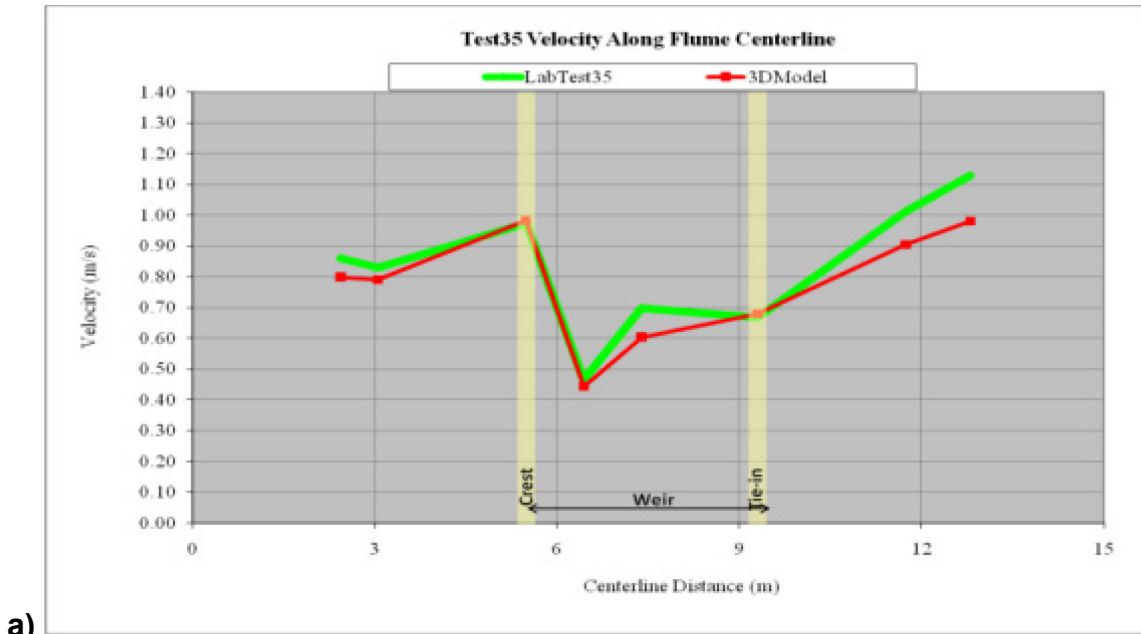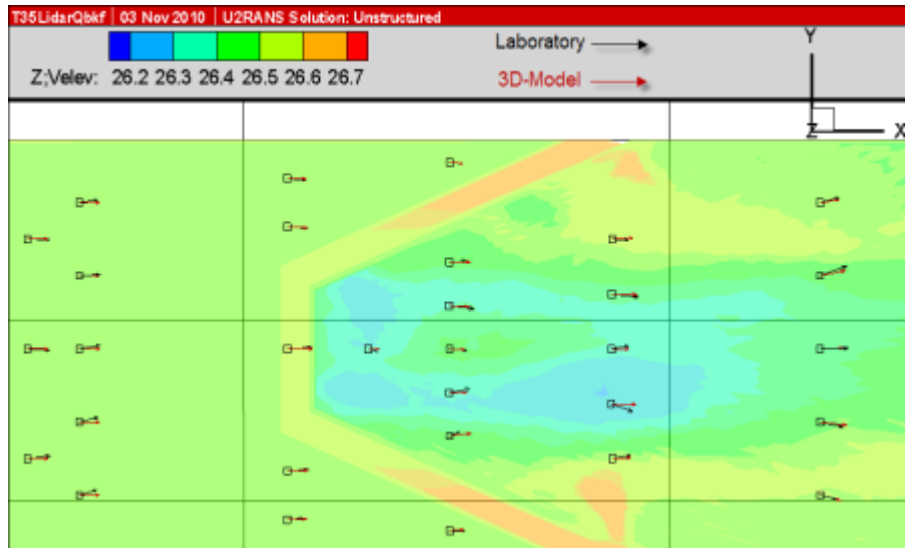
a)

b)
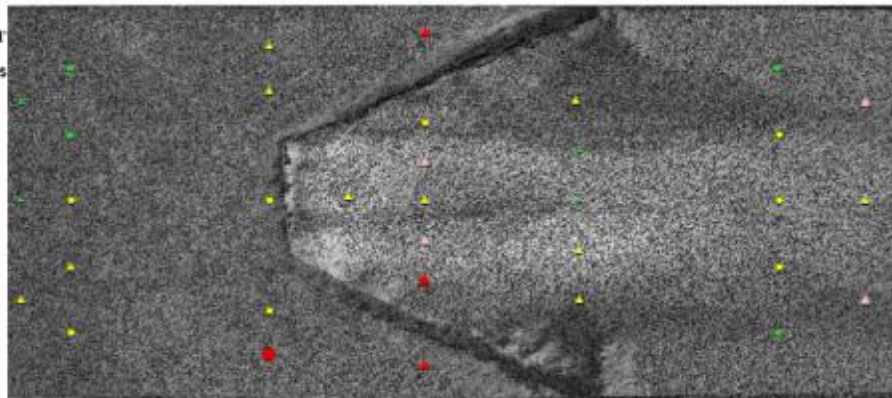
c)

Figure 5.17 – Velocity comparison: a) Test 35 centerline profile, b) Test 35 sample point locations, c) Test 35 sample point maximum percent errors.

5.5.2 Comparison with Field Measurements

Using field data collected on the South Fork of the Little Snake River near Steamboat, Colorado, measured water surface elevations, velocities, and bed topography were analyzed and compared to results from the 3D numerical model. Using the topographic survey data from field visits, a scatter point data set containing Northing (y), Easting (x), and elevation (z) was generated and used in creating the computational mesh for the numerical model. A mesh was generated using quadrilateral and triangular elements to describe the structure and channel bathymetry in the numerical model (Figure 5.18).

Measured water surface elevations and discharge from site visits were used as input boundary conditions to the numerical model. The upstream boundary condition specified a discharge measured in the field and the downstream boundary condition specified the corresponding known water surface elevation also measured in the field.

Comparisons between numerical model results and measured water surface elevations (Figure 5.19) show that the numerical model is able to replicate field measurements by matching measured water surface elevations within 10 percent (Figure 5.20) for bankfull discharge at measured depths ranging from 1 to 4.3 feet.

Figure 5.21 shows the observed (true) water surface elevation measurements plotted against predicted ones from the numerical model for the same locations. The linear correlation coefficient of 0.98 confirms relatively good overall agreement (with no regard to spatial component in the data) between predicted and measured values. The residuals of the water surface elevation predictions have a slight negative bias with a mean = -0.095 ft (n = 27).

Figure 5.22 shows the observed (true) velocity measurements plotted against predicted ones from the numerical model for the same locations. The linear correlation coefficient of 0.93 confirms relatively good overall agreement (with no regard to spatial component in the data) between predicted and measured values. The residuals of the velocity predictions have a slight positive bias with a mean = 0.22 ft/s (n = 27). Comparisons between numerical model results and measured velocities also show that the numerical model is able to replicate the redirection of the stream lines over the weir and match field measurements within 25 percent (Figure 5.23). Variations in the predicted and measured values are likely attributable to minor differences in the modeled topography, high turbulence downstream of the structure crest from the plunging flow over the structure crest and arms, accuracy of the instruments used in the measurements, and measurement error in the field.

The comparison of results from the numerical model for the field site described above provides validation that the numerical model is capable of representing the complex flow patterns associated with river spanning rock weirs.
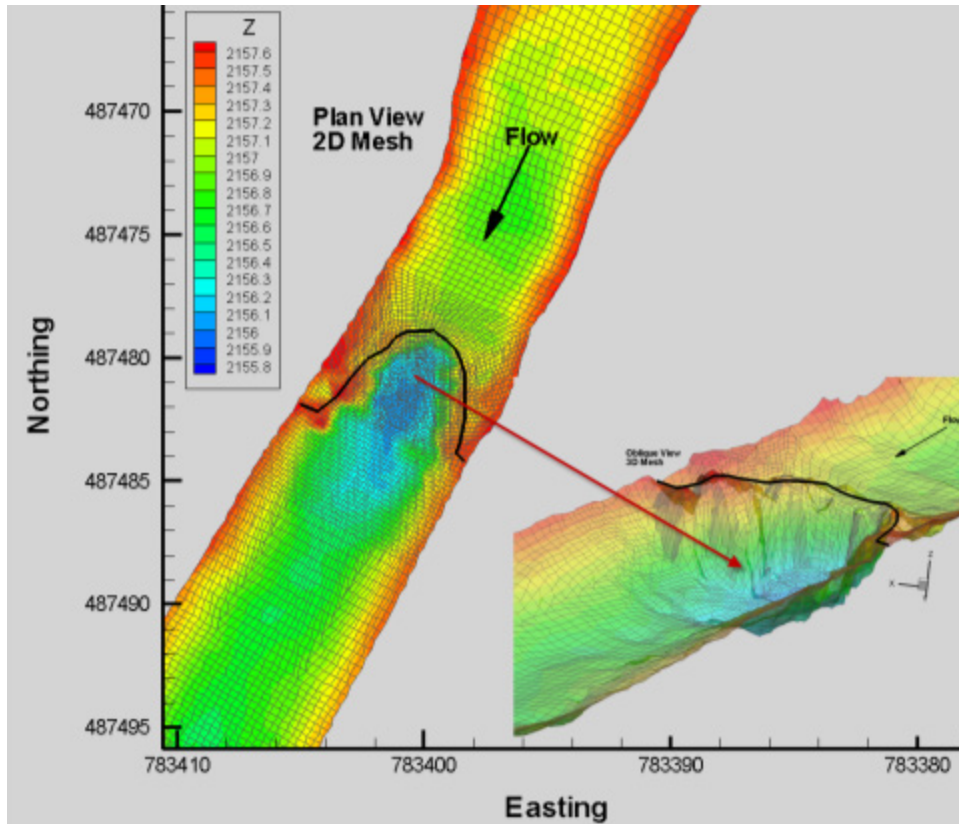
Figure 5.18 – Computational mesh for U-weir on South Fork Little Snake River.
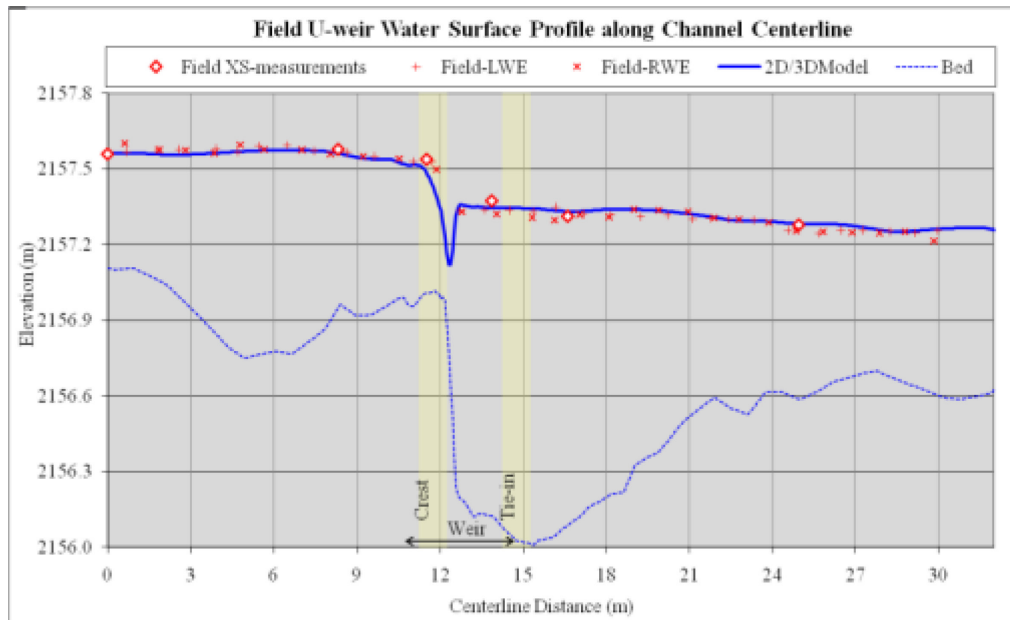
Figure 5.19 – Field U-weir numerical model centerline water surface profile comparison with field measured left water edge (LWE) and right water edge (RWE) elevations.

**Field U-weir
Comparison Points**

▲ <-27.5%

▲ -27.5% - -25.1%

▲ -25% - -15.1%

▲ -15% - -12.6%

▲ -12.5% - -10.1%

▲ -10% - -7.6%

▲ -7.5% - -5.1%

▲ -5% - 0%

▪ 0.1% - 5.5%
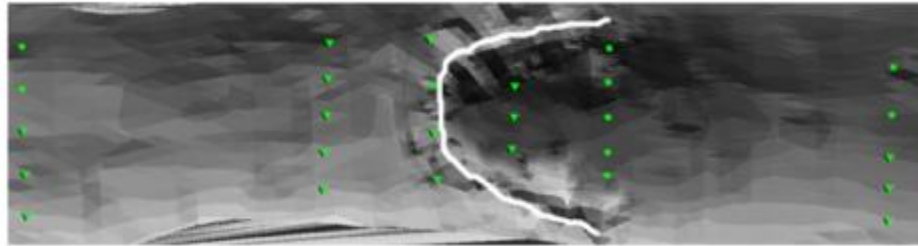
Figure 5.20 – Field U-weir percent error in predicted water surface elevations.
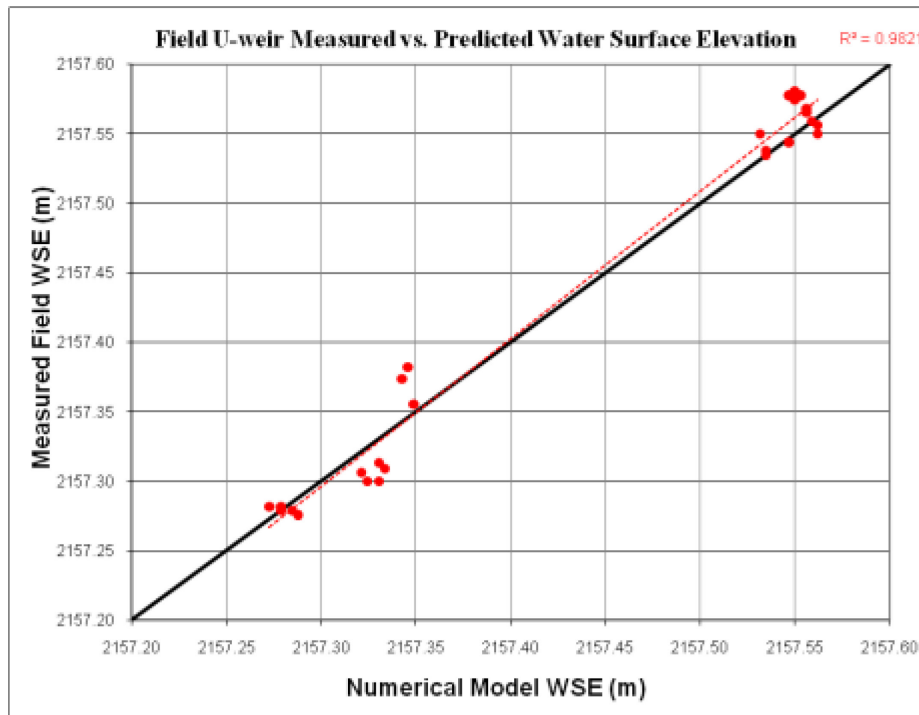
Figure 5.21 – Measured vs. predicted water surface elevation.
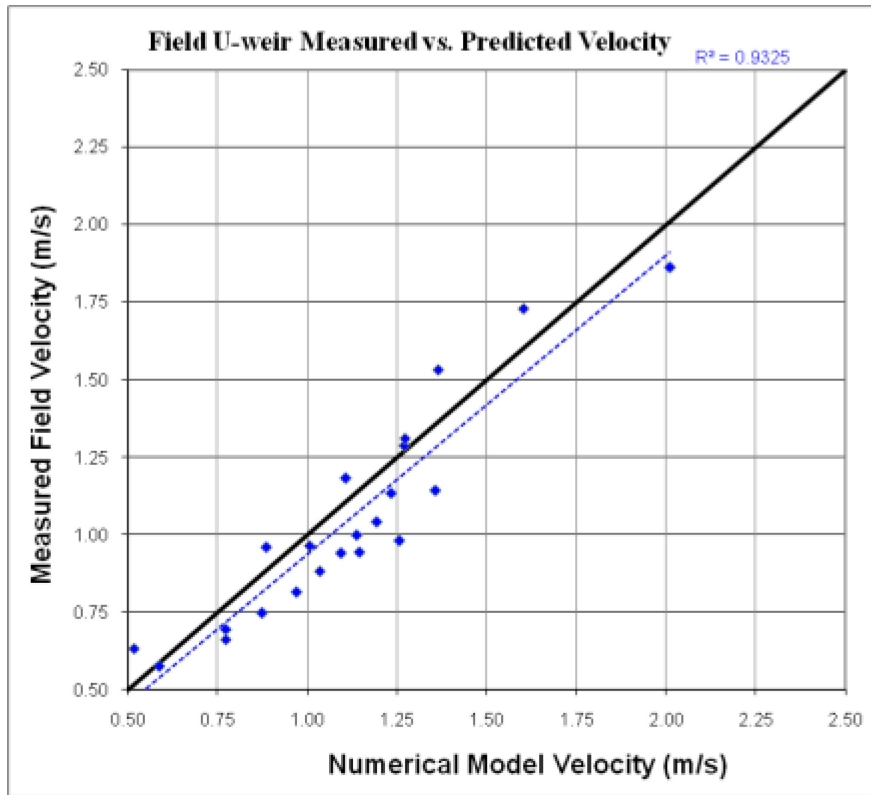
Figure 5.22 – Measured vs. predicted velocity.

a)

b)

c)
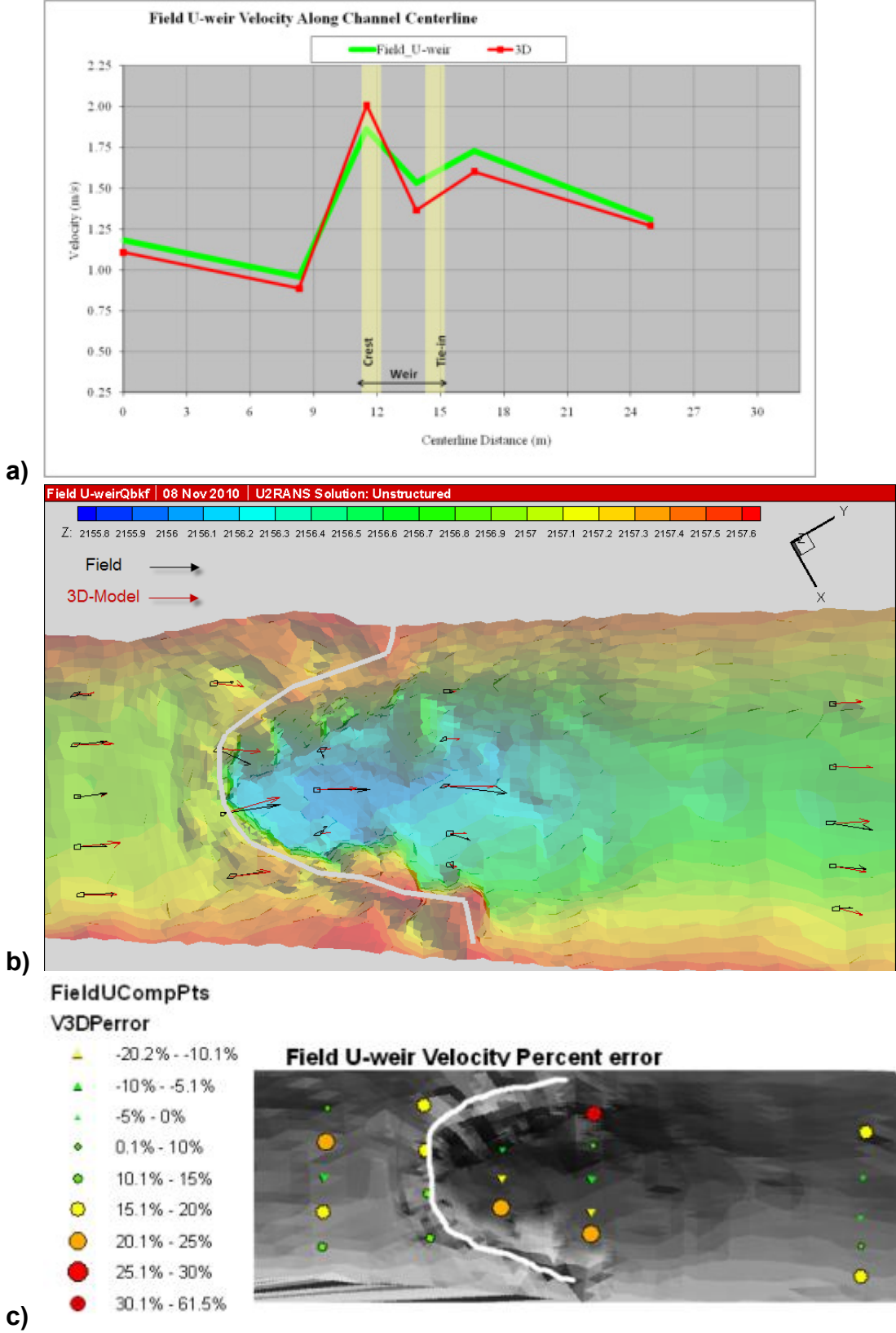
Figure 5.23 – Field U-weir velocity comparison: a) centerline profile, b) sample point locations, c) sample point percent error.

5.5.3 Three-Dimensional Model Design

The results of the numerical model comparison with the laboratory data and the field data show that the three-dimensional model, U2RANS, is capable of simulating the complex hydraulics associated with river spanning rock structures. As a result, the 3D-model was used to investigate how local flow patterns are affected by variations in structure geometry and provide a cost effective method for evaluation of a range of structure geometries and channel conditions to develop a more complete understanding of structure performance and optimize structure design. To understand how these types of structures affect the local hydraulics, an analysis of a wide range of structure geometries needed to be tested. The following section describes the design of the numerical model testing matrix.

The following variables were included in compiling the testing matrix and are described in more detail in the sections below:

1. Bed material

2. Discharge

3. Channel geometry (slope, width, and depth)

4. Structure throat width

5. Structure drop height

6. Structure arm length (incorporates arm angle and slope)

The following variables were considered but not included in the design of the testing matrix for the following reasons:

7. Bank height: Bank height is set by hydraulic geometry equations. All model runs were at bankfull or less; no overbank flows were simulated, and therefore, water depth does not exceed the height of the weir.

8. Top width: Top width was set by hydraulic geometry equations.

9. Non-linear weirs: Asymmetric geometries were not investigated because they were beyond the scope of this research.

10. Meandering channel: Radius of curvature was not part of the study scope. Only straight prismatic channels were investigated.

Grain sizes were selected to match field conditions in which river spanning rock weirs are most commonly used (e.g. gravel bed rivers). The distributions for the D_{84} and D_{16} were set to plus and minus one phi class. The three D_{50} grain diameters were selected using the geometric mean of the AGU classification system, a log base 2 scale:

11. Coarse Gravel: D_{50} = 22.6 mm, D_{84} = 45.3 mm, D_{16} = 11.3 mm

12. Small Cobble: D_{50} = 90.5 mm, D_{84} = 181.0 mm, D_{16} = 45.3 mm

13. Large Cobble: D_{50} = 181 mm, D_{84} = 256 mm, D_{16} = 90.5 mm

5.5.3.1 Bankfull Channel Geometry

Previous research has shown that it is possible to define a ''bankfull channel geometry'' (Leopold and Maddock, 1953; Leopold et al., 1964) in terms of a bankfull width, bankfull depth and down-channel bed slope. More recently, Parker et al. (2007) used a baseline data set consisting of four differing stream reaches from Canada, USA, and Britain to determine bankfull hydraulic relations for alluvial, single-thread gravel bed streams with definable channels and floodplains (Equations 5.3, 5.4, and 5.5). Their results show a considerable degree of universality and the exponents of Q_{bf} in the equations below are similar to those found by other authors (e.g., Millar, 2005).

$$S \cong 0.101 \cdot \left(\frac{Q_{bf}}{\sqrt{gD_{s50}} \, D_{s50}^2} \right)^{-0.344} \tag{5.3}$$

$$W_{bf} \cong \frac{4.63}{g^{1/5}} Q_{bf}^{0.4} \left(\frac{Q_{bf}}{\sqrt{gD_{s50}} \, D_{s50}^2} \right)^{0.0667} \tag{5.4}$$

$$H_{bf} \cong \frac{0.382}{g^{1/5}} Q_{bf}^{2/5} \tag{5.5}$$

where S = bed slope;

     g = gravitational acceleration (m/s^2);

     Q_{bf} = bankfull discharge (m^3/s);

     D_{s50} = median particle diameter, (m);

     W_{bf} = bankfull width (m); and

     H_{bf} = bankfull depth (m).

Additionally, Parker et al. applied the regression relations to three other data sets, one from Maryland, one from Colorado, and one from Britain, confirming this tendency toward universality. The degree of universality and ease of use of the hydraulic geometry equations presented by Parker et al. (2007) were the reasons that they were selected for determining the bankfull hydraulic geometry used in this study. Given the bed material grain sizes listed above and a range of representative bankfull discharges that match field conditions in which river spanning rock weirs are most commonly used, Equations 5.3, 5.4, and 5.5 above were used to compute central width, depth, and channel slope tendency in the design matrix. To analyze each structure configuration and the effects on the local flow patterns, the discharge for each structure configuration was varied to include one-third, two-thirds, and bankfull discharges.

5.5.3.2 Structure Geometry

a. Throat width

Initial structure throat width was set to 1/3 the bankfull width as specified in Rosgen (2006) and following observed field applications (Reclamation, 2007). To

study how flow patterns are affected by changes in structure throat width, the throat width was varied over 1/4, 1/3, and 1/2 the calculated bankfull width for each of the three grain sizes and corresponding channel geometry.

b. Drop height

Initial drop height was set to 0.8 feet as prescribed by WDFW (2004) for the maximum drop height allowed for fish passage. Since the focus of this proposal is related to overall structure performance and not specifically fish passage, additional fish passage criteria were not considered and are outside the scope of this research proposal. To study how the drop height over the structure affects flow patterns, the structure drop height was varied by 0.5 and 1.5 times the value prescribed by WDFW (2004). This results in a range of drops heights of 0.4, 0.8, and 1.2 ft.

c. Arm length

Weir arm length (L_A) was defined as the length of the weir along the channel bank. The plan angle (θ) was the angle between the channel bank and L_A. The profile angle (ϕ) was the angle between the horizontal plane and the weir arm that sloped downward from the tie-in at bankfull elevation to the weir crest. The weir arm length is a function of arm angle, arm slope, channel width, bank height, and throat width.

Initial structure arm lengths were designed such that the arm angle and slope for a given channel geometry approached as close as possible the midpoint of the design ranges specified in Rosgen (2006). Recommended arm angles were between 20 to 30 degrees and arm slopes between 2 to 7 percent. Target angles were 25 degrees for plan angles and 4.5 percent for profile angles. The solver function in Microsoft Excel® was used to calculate the weir arm length that minimized the relative distances on the planform and profile angles of the weir arms. This minimized solution was then used to calculate the arm length ranges that would be used in the numerical modeling in two ways; multiplying the minimized values by 0.5 and 2. This provided a wide range of planform (10.31 to 48.35 degrees) and profile angles (1.47 to 10.11 percent) to be tested (Figure 5.24).

The channel and structure geometry used in the numerical modeling is generated from the mesh generation program described by Holmquist-Johnson (2011). The channel consists of a simplified trapezoidal channel with 0.75H:1V side slopes and the width and depth of the channel calculated from hydraulic geometry regime equations presented in Section 5.5.3.1. Figure 5.25 shows a computational mesh that was generated from the mesh generator and Figure 5.26 shows the numerical representation of the trapezoidal channel looking downstream at the structure crest/throat of a U-weir and how the arms tie-in to the top bank. One should notice how the downstream footer begins to be exposed as the arm elevation increases and the footer elevation becomes greater than the bed elevation.
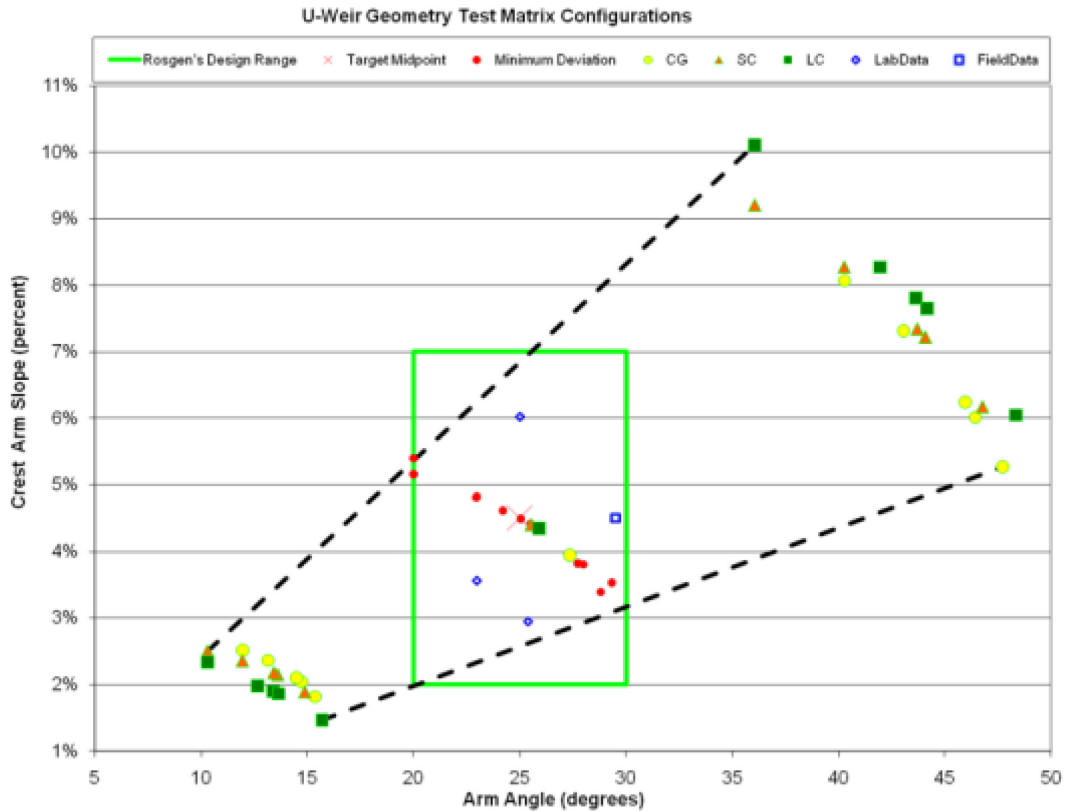
Figure 5.24 – Plot of variation in structure parameters for three grain sizes (CG-coarse gravel, SC-small cobble, LC-large cobble) used in design matrix.

The research conducted by Holmquist-Johnson (2011) included the development and implementation of a rock weir mesh generation program and numerical model testing matrix, comparison and validation of numerical modeling methods for a field site and laboratory data, analysis of the numerical model output and development of stage-discharge relationships for U-weirs. Thirty three unique weir configurations were generated and numerically modeled at five different flow rates ($1/10Q_{bkf}$, $1/5Q_{bkf}$, $1/3Q_{bkf}$, $2/3Q_{bkf}$, and Q_{bkf}) for a total of 165 test cases. Output from the numerical modeling was analyzed to quantify the effects that variations in structure geometry had on local velocity and bed shears stress distributions and develop a stage-discharge relationship for U-weirs.

Detailed descriptions of the three-dimensional modeling, testing matrix, model design and validation, modeling procedures, and results are documented in Holmquist-Johnson (2011).
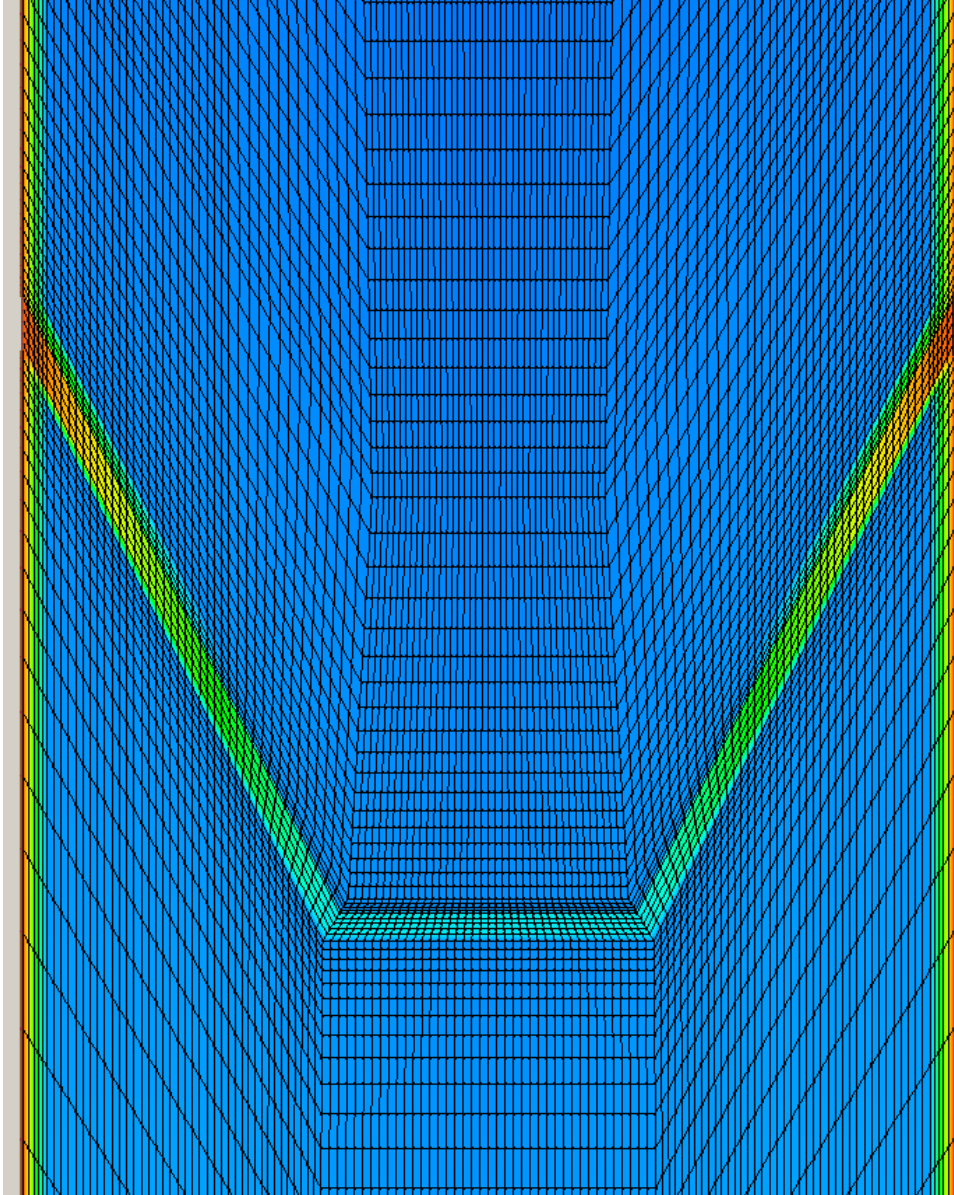
Figure 5.25 – Computational mesh generated from mesh generator for a U-weir.
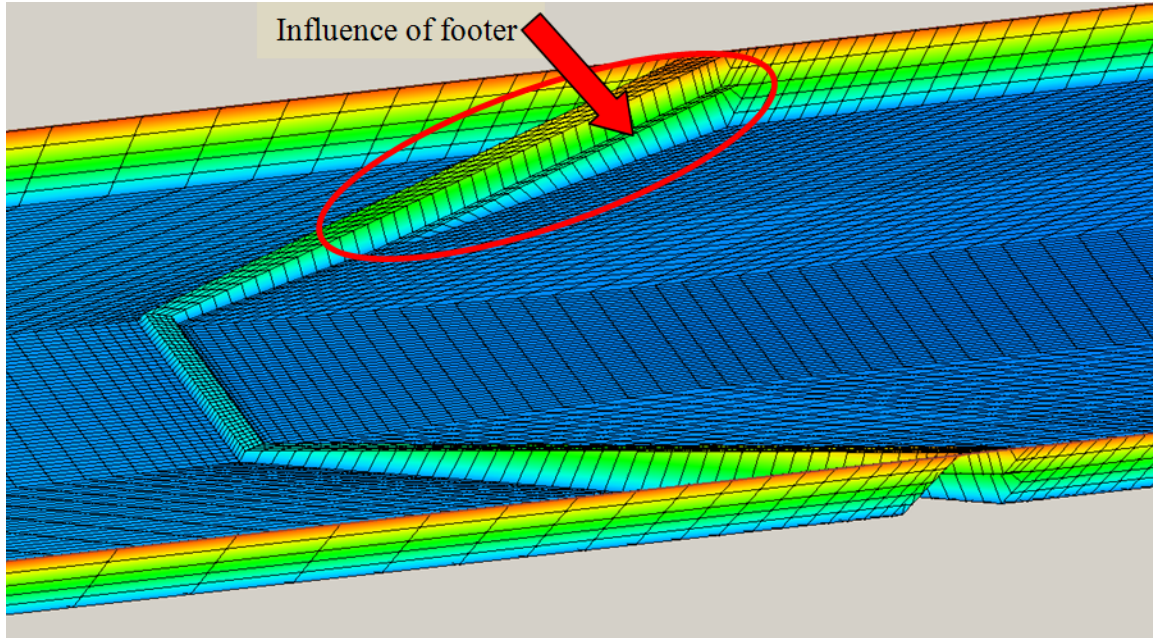
Figure 5.26 – Numerical representation of U-weir header-footer configuration.

5.6 Comparisons between 1D, 2D, and 3D

To better understand the applicability and limitations of each of the numerical modeling methods presented in the previous sections, a comparison of predicted water surface elevations and corresponding velocities from the 1D, 2D, and 3D models was conducted. The results of the numerical model comparisons for the field data set as well as the laboratory tests are presented in the following sections.

5.6.1 Field Case

Using field data collected on the South Fork of the Little Snake River near Steamboat Colorado, measured water surface elevations, velocities, and bed topography were analyzed and compared with results from each of the numerical models (1D, 2D, and 3D) described in the previous sections.

Comparisons between numerical model results and measured water surface elevations (Figure 5.27) show that the 2D and 3D (2D/3D) numerical models were able to replicate field measurements by matching measured water surface elevations within 7.5 percent (Figure 5.28) for bankfull discharge at measured depths ranging from 0.3 meters to 1.3 meters. Figure 5.28 also shows that for the bankfull flow, the 1D model was able to replicate field measurements of water surface elevation within 12.5 percent with the exception of a 25-percent error near the structure crest.
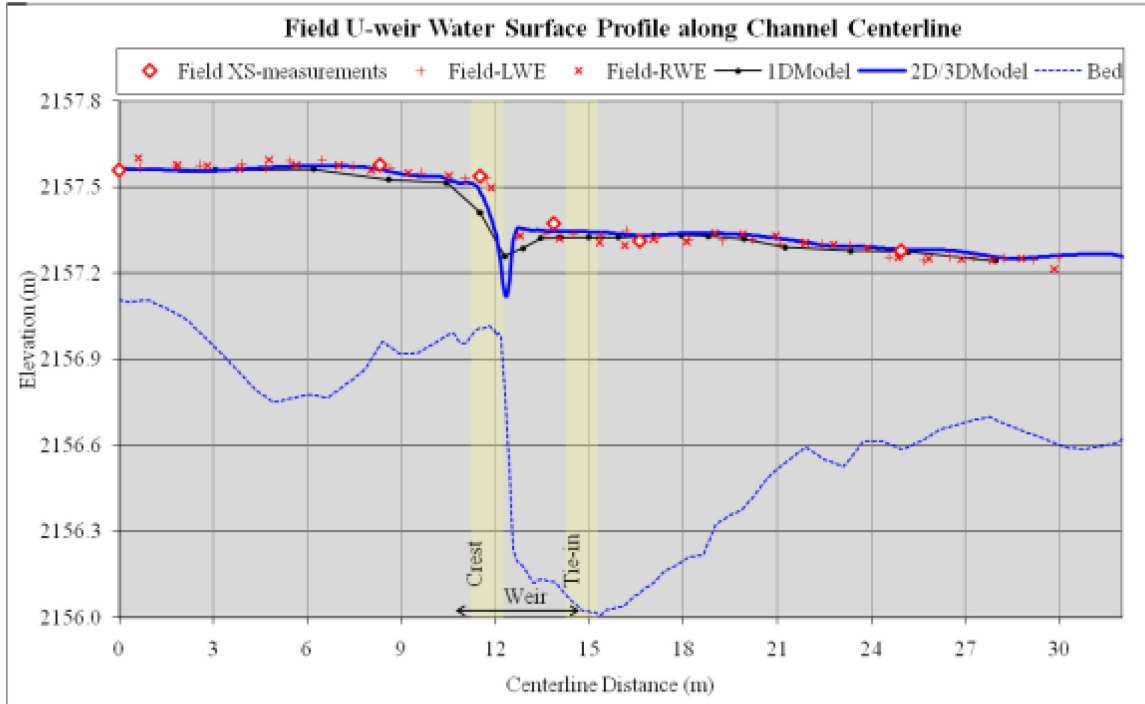
Figure 5.27 – Field and numerical model U-weir centerline water surface profile comparison.
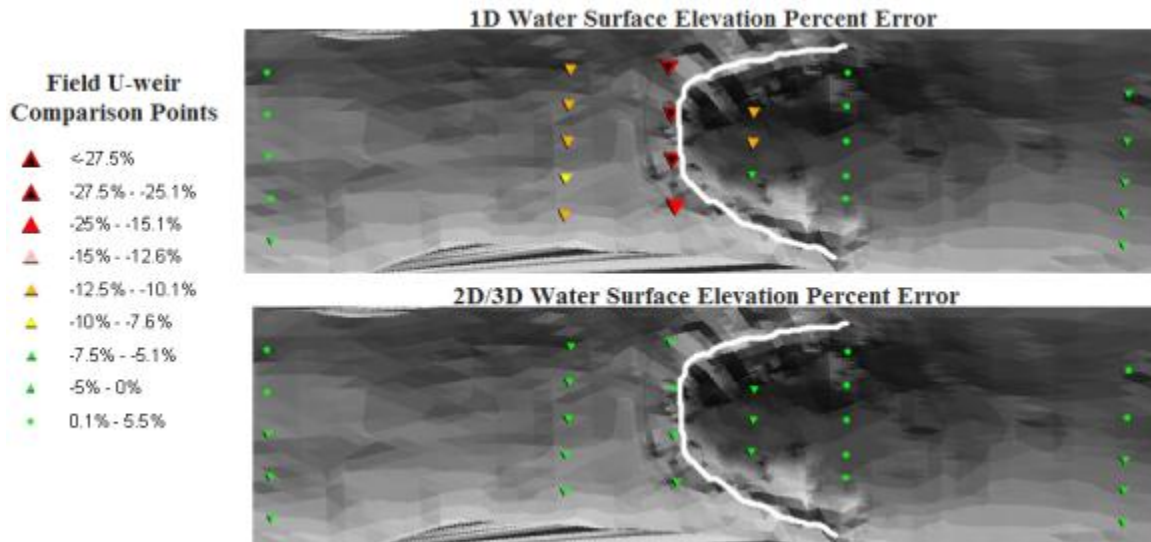
Figure 5.28 – Field U-weir percent error in numerical model water surface elevations.

Figure 5.29 shows the observed (true) water surface elevation measurements plotted against predicted ones from the numerical models for the same locations. The linear correlation coefficients of 0.85 for the 1D model and 0.98 for the 2D/3D model show relatively good overall agreement (with no regard to the spatial component in the data) between predicted and measured values for all models. However, when the spatial component of the data is considered (Figure

5.30), it is evident that the 2D/3D model provides a much better prediction of the water surface elevation near the crest of the structure. While the error in the predicted water surface elevations from the 2D/3D model are all less than 8 percent (0.06 m), the 1D model shows more than a 25 percent error (0.13 m) at the structure crest. The large error in the 1D model is due to the rapidly varied flow condition that exists along the crest of the structure which is not uniform across the channel. The 1D model is not able to properly simulate the rapidly varied flow conditions that occur near the crest of the structure that change both longitudinally and laterally along the structure.

Comparisons between numerical model results and measured velocities along the channel center line show that the numerical models also differ in their ability to replicate field measurements, especially downstream from the structure crest (Figure 5.31). From Figure 5.31, it is evident that the 3D model provides a much better prediction of the velocities downstream from the crest of the structure. While the maximum error in the predicted channel centerline velocity from the 3D model is 11 percent (0.168 m/s), the 1D and 2D models have much greater errors at 56 percent (0.97 m/s) and 41 percent (0.71 m/s), respectively. The 1D and 2D models are not able to properly simulate the vertical components of the velocity vectors that occur downstream of the structure crest. The high variability in the velocity predictions is a result of the plunging flow that occurs along the structure crest causing the flow downstream of the structure to have a strong vertical velocity component which violates 1D and 2D model assumptions that require velocity vectors perpendicular to the vertical plane. Additionally, while the 3D model calculates velocity vectors along the vertical, 1D and 2D models compute average cross section and depth-averaged velocity, respectively. Since the 1D model provides an average cross sectional velocity, predicting changes in flow patterns around the structure both longitudinally and laterally is not feasible. Hence the need for higher order models (2D and 3D) that incorporate lateral changes in flow.
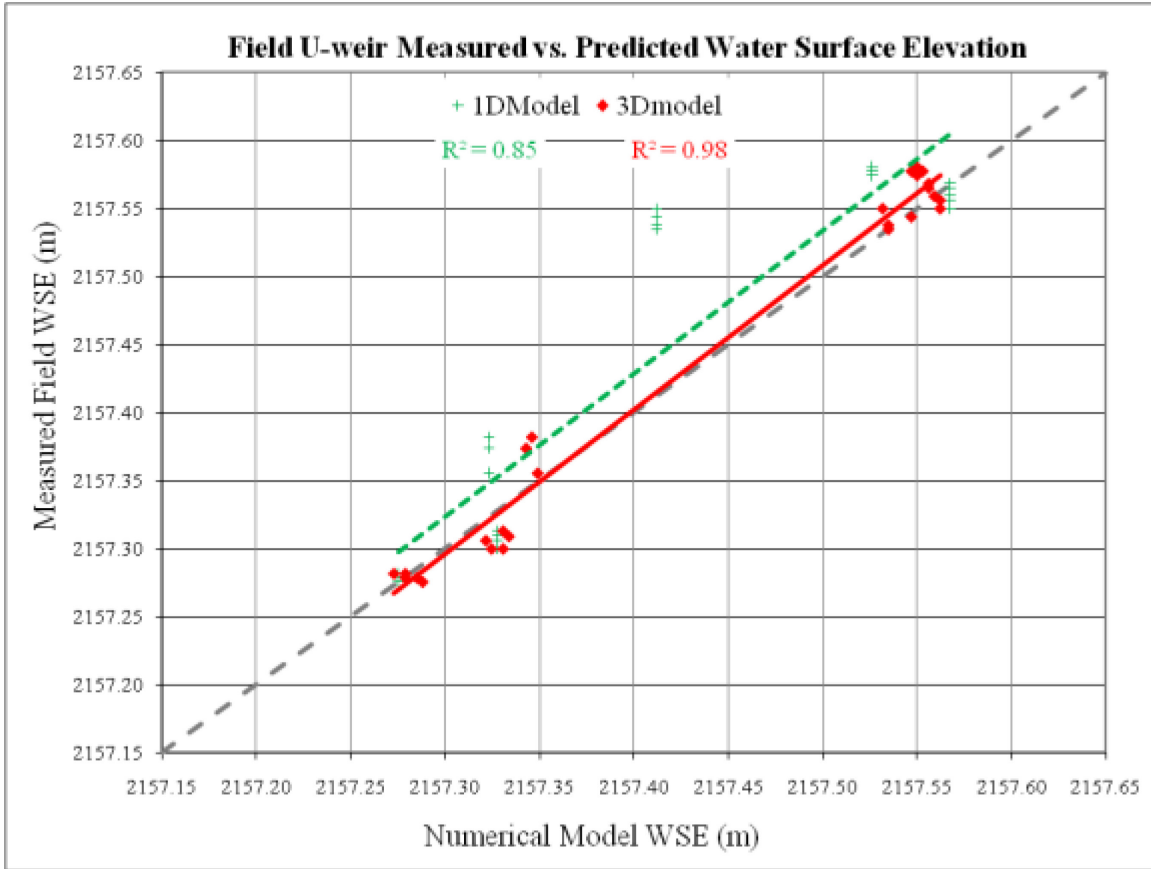
Figure 5.29 – Field U-weir measured vs. predicted water surface elevation.
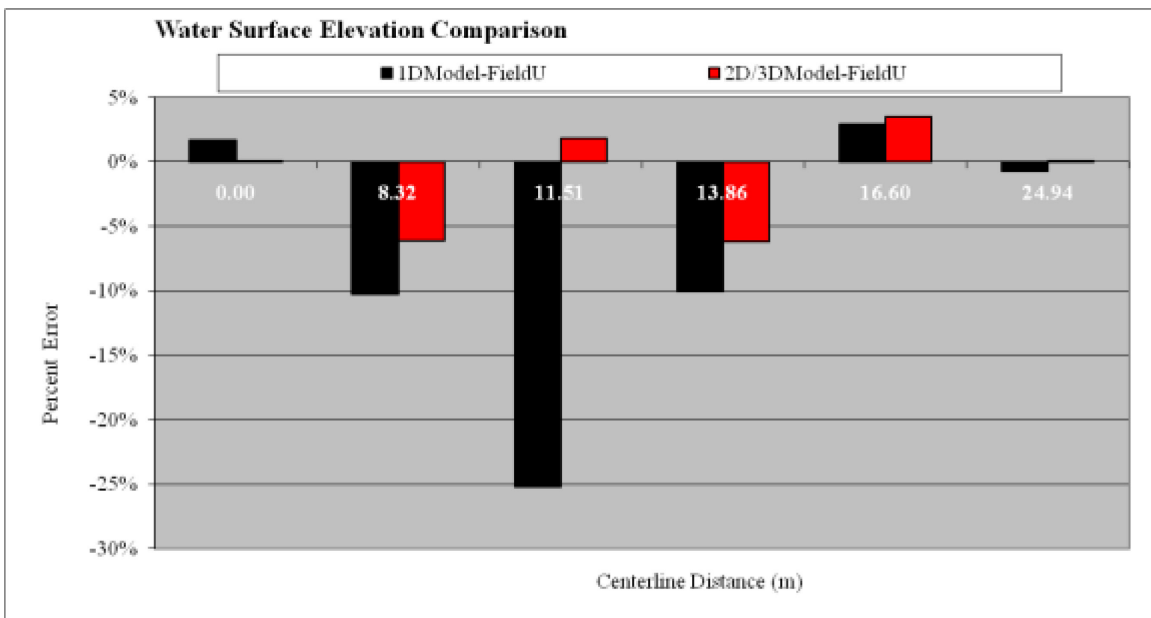
Figure 5.30 – Field U-weir water surface elevation comparison, percent error along centerline profile.

a)

b)
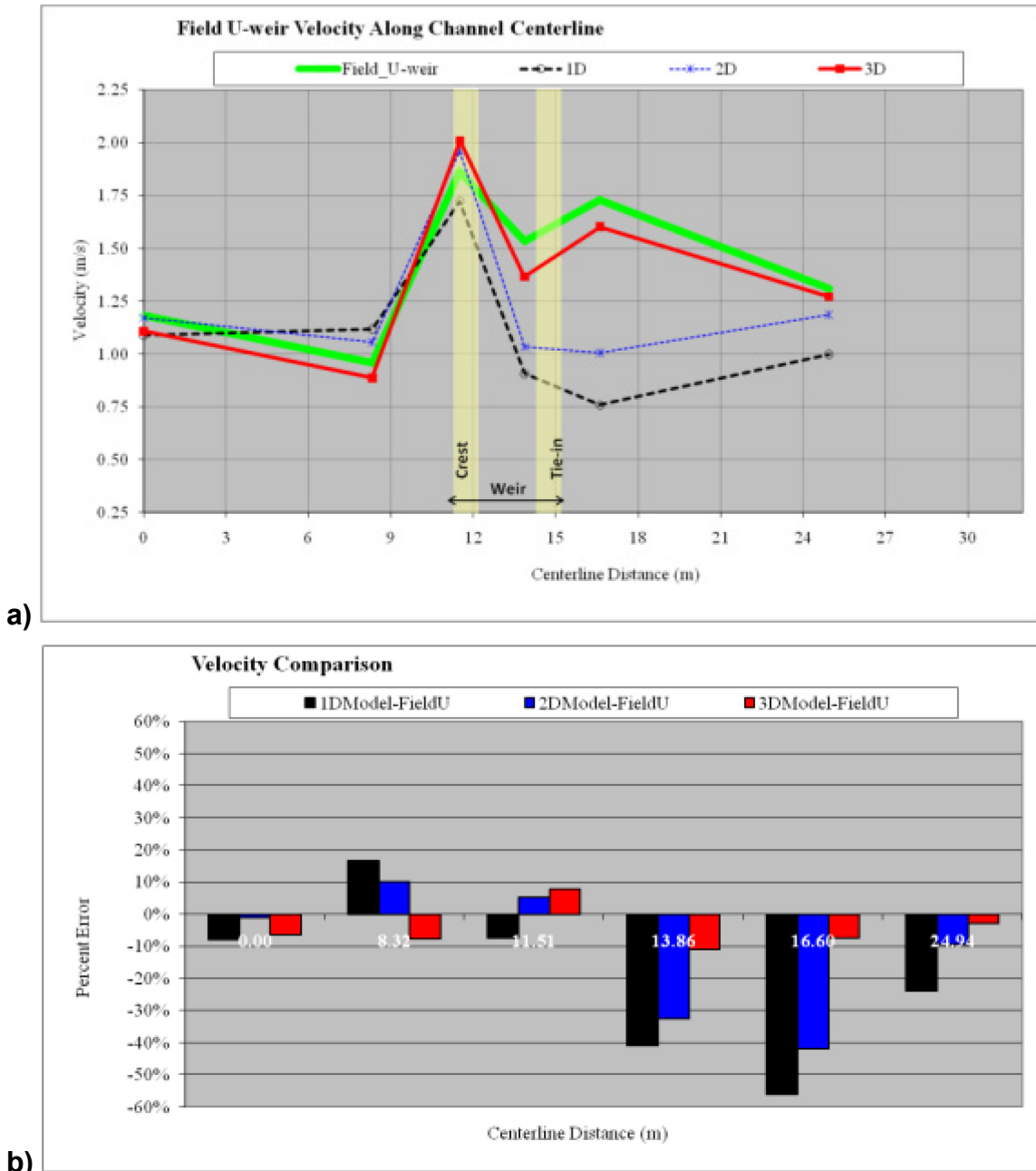
Figure 5.31 – Field and numerical model U-weir velocity comparison along channel centerline: a) Velocity magnitude, b) Percent error.

Figure 5.32 shows the field (true) velocity measurements plotted against predicted ones from the numerical models for the same locations. The linear correlation coefficients of 0.14 for the 1D model, 0.46 for the 2D model, and 0.93 for the 3D model show that only the 3D model provides relatively good overall agreement (with no regard to the spatial component in the data) between predicted and measured values. When the spatial component of the data is considered (Figure 5.33), it is evident that the 3D model provides a much better prediction of the velocities downstream from the crest of the structure. While the maximum error in the predicted velocity from the 3D model is 28 percent (0.3 m/s), the 1D and

2D models have much greater errors at 130 percent (0.976 m/s) and 73 percent (0.83 m/s), respectively. This further elucidates the inability of 1D and 2D models to properly simulate the vertical velocity vectors that occur downstream of the structure crest.
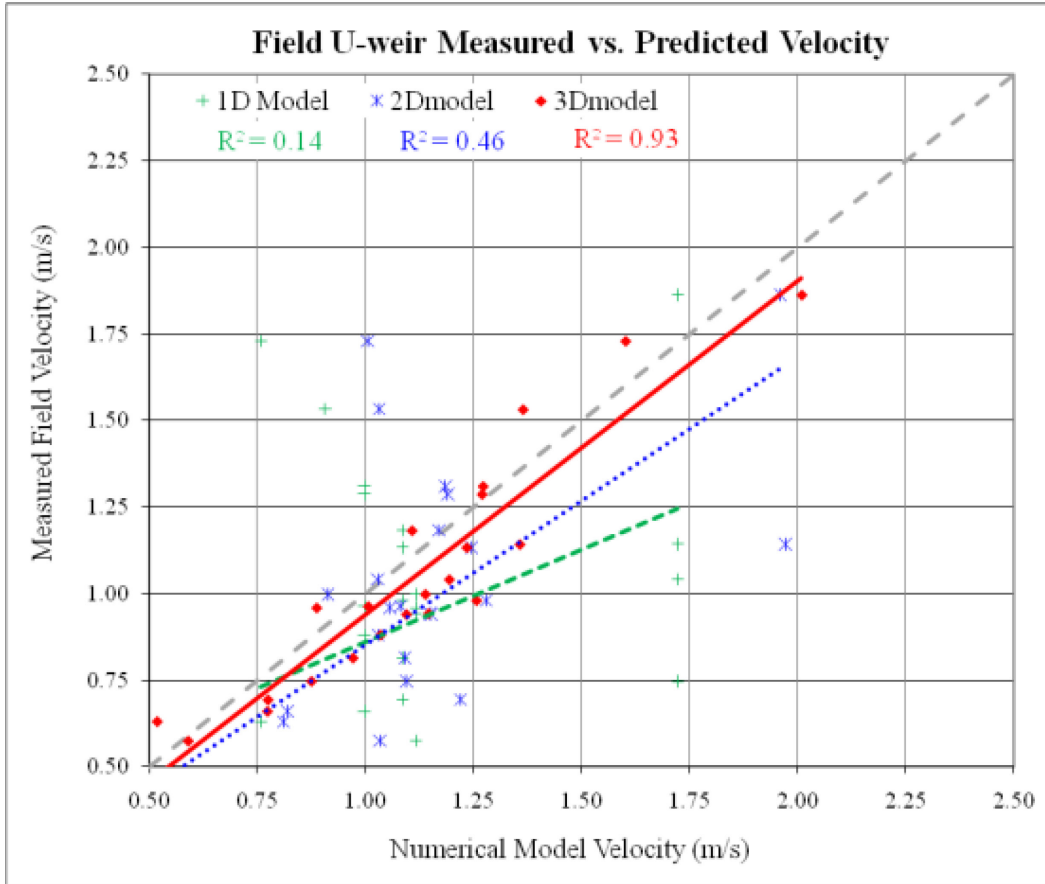
Figure 5.32 – Field U-weir measured vs. predicted velocity.
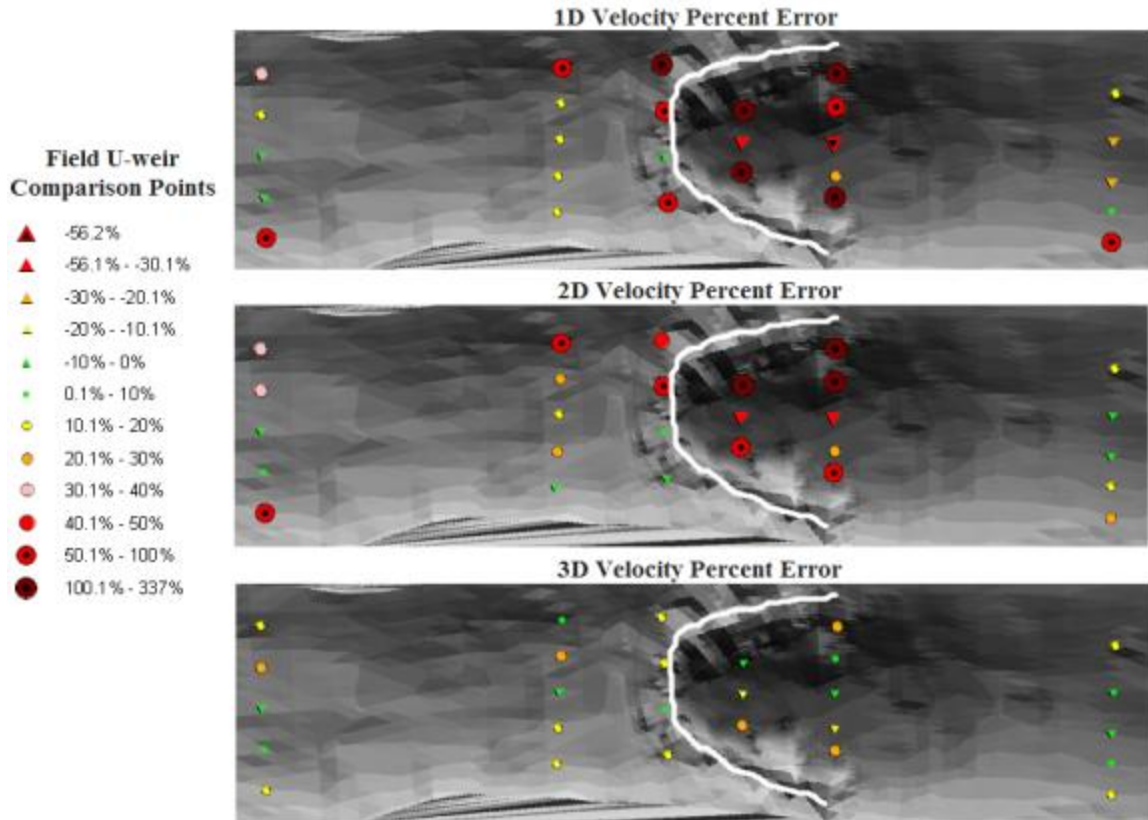
Figure 5.33 – Field U-weir and numerical model velocity percent error comparison for 1D, 2D, and 3D models.

A box and whisker plot showing the variation between the 1D, 2D, and 3D model percent error magnitude in velocity is presented in Figure 5.34. It is evident from the comparison that of the three numerical models, the 3D model provides a better overall prediction of the velocities throughout the study reach with a 95-percent confidence interval of 7.5–18 percent compared to 17–66 percent and 10–54 percent for the 1D and 2D models, respectively.
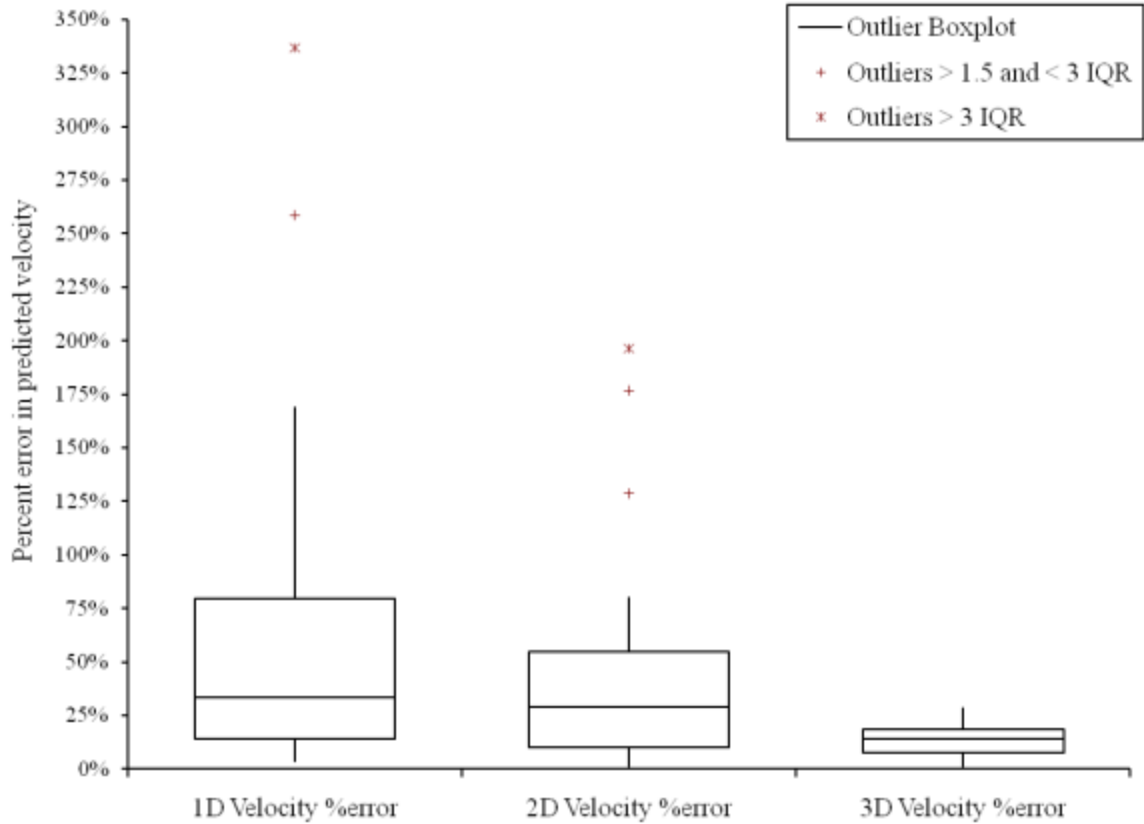
	n	Min	1st Quartile	Median	95% CI		3rd Quartile	Max	IQR
1D Velocity %error	27	3.61%	13.84%	33.50%	16.57%	65.52%	79.64%	336.66%	65.81%
2D Velocity %error	27	1.02%	9.96%	28.69%	10.18%	53.64%	54.64%	196.49%	44.68%
3D Velocity %error	27	1.33%	7.41%	14.00%	7.48%	17.96%	18.64%	28.18%	11.23%

Figure 5.34 – Field U-weir velocity comparison, percent error magnitude box-plot with maximum 1.5 interquartile range (IQR) and 95% confidence interval (CI) for 1D, 2D, and 3D models.

5.6.2 Laboratory Case

Using data collected during the laboratory testing conducted by Colorado State University (Meneghetti, 2009; Scurlock, 2009), measured water surface elevations, velocities, and bed topography were analyzed and compared with results from each of the numerical models (1D, 2D, and 3D) described in the previous sections.

Analysis of the numerical model output shows that the water surface elevations along the channel centerline from each of the models matched reasonably well with the measured values from the physical model (Figure 5.35). While the percent error in the predicted water surface elevations from the 2D/3D model is less than 5 percent (0.011 m), the 1D model shows a 15 percent error (0.031 m) near the structure crest (Figure 5.36). The large error in the 1D model is due to the rapidly varied flow conditions that exist along the crest of the structure that are

not uniform across the channel. The 1D model is not able to properly simulate such conditions.

Figure 5.37 shows the laboratory water surface elevation measurements plotted against predicted ones from the numerical models for the same locations. The linear correlation coefficients of 0.95 for the 1D model and 0.98 for the 2D/3D model show good overall agreement (with no regard to the spatial component in the data) between predicted and measured values for all models.

However, when the spatial component of the data is considered (Figure 5.38), it is evident that the 2D/3D model provides a better prediction of the water surface elevations near the structure. Figure 5.38 shows that the predicted water surface elevations from the 2D/3D model were within 10 percent for all three tests (n=102 observations) with measured depths ranging from 0.061 meters to 0.427 meters. The 1D model was able to replicate field measurements within 23 percent with the exception of a 40-percent error in the middle of the structure where multiple water surface elevations were present along the transect due to the structure arm.

Comparisons between numerical model results and measured laboratory velocities (Figure 5.39) show that the numerical models differ in their ability to replicate laboratory measurements. The high variability in the velocity predictions is a result of the plunging flow that occurs along the structure crest causing the flow downstream of the structure to have a strong vertical velocity component which provides energy dissipation. From Figure 5.40, it is evident that the 3D model provides a much better prediction of the velocities downstream from the crest of the structure compared with the 1D and 2D models. While the maximum error in the predicted channel centerline velocity from the 2D and 3D model is 0.133 m/s (19%) and 0.094 m/s (13%) respectively, the 1D model has a much greater error at 0.336 m/s (72%). The 1D model is not able to properly simulate the flow convergence and vertical component of the velocity vectors that occur downstream of the structure crest. Since the 1D model provides an average cross sectional velocity, predicting changes in flow patterns around the structure both longitudinally and laterally is not feasible.
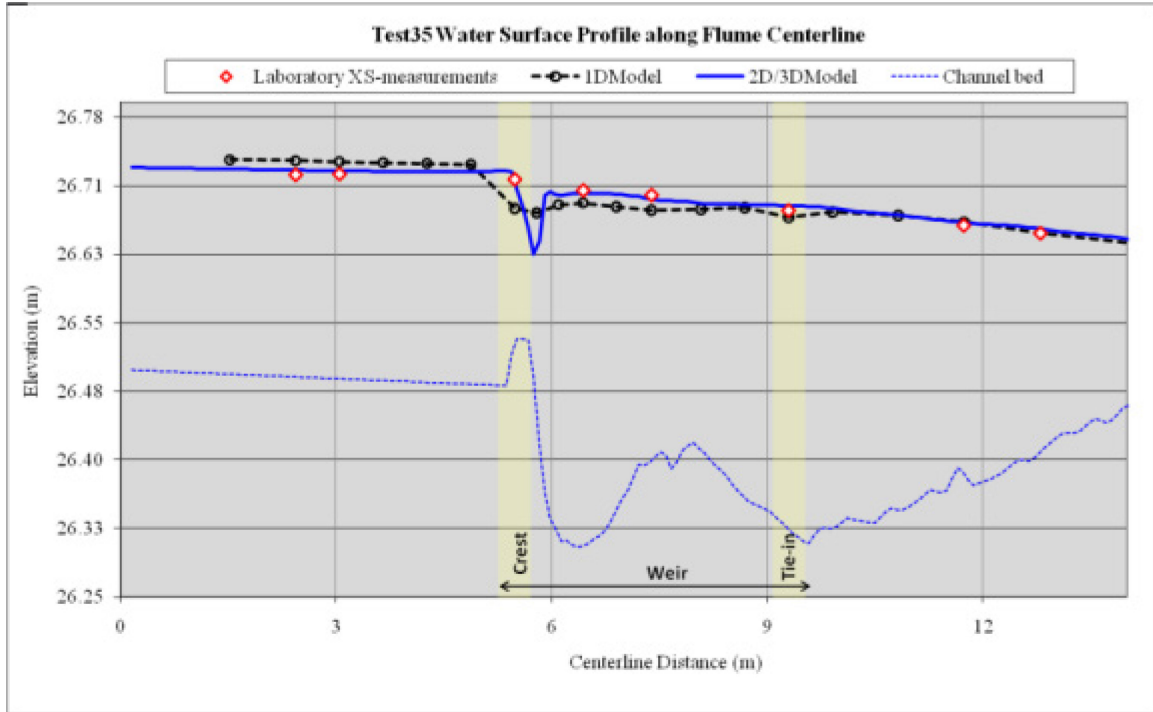
Figure 5.35 – Test 35 water surface profile along flume centerline.
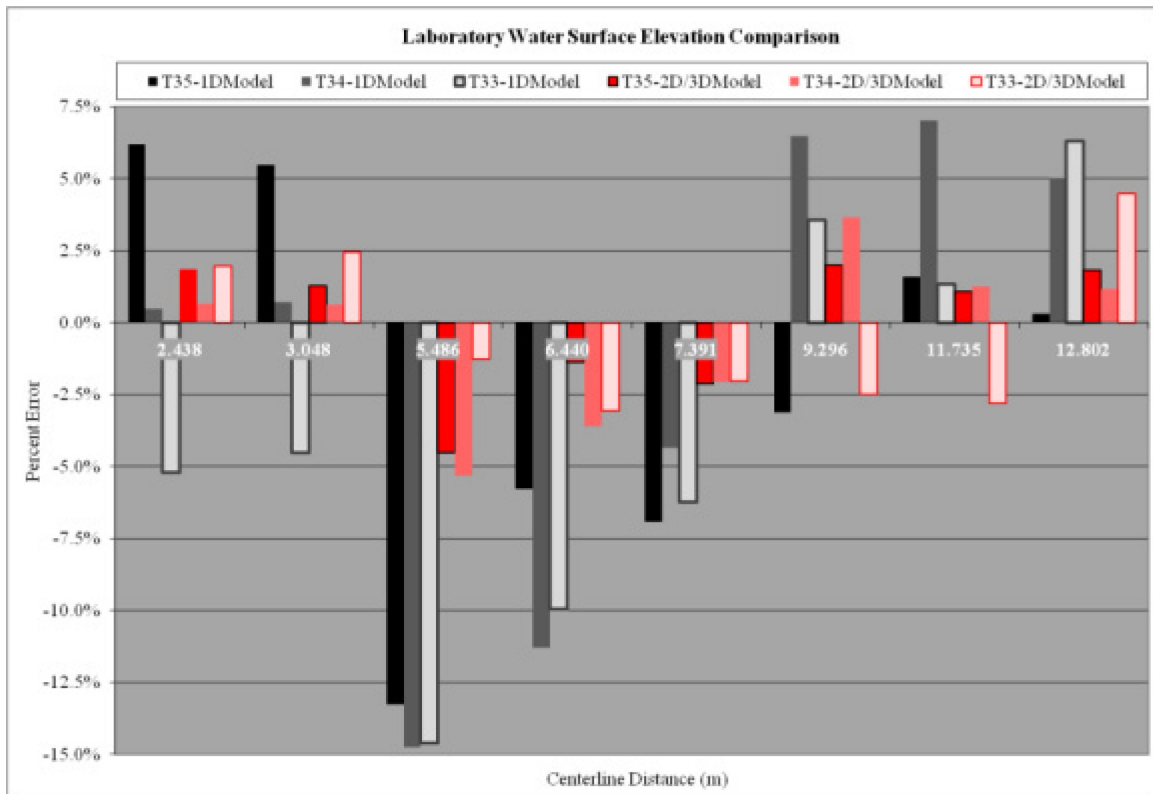
Figure 5.36 – Laboratory and numerical model water surface elevation comparison, percent error along channel centerline for tests 33-35.
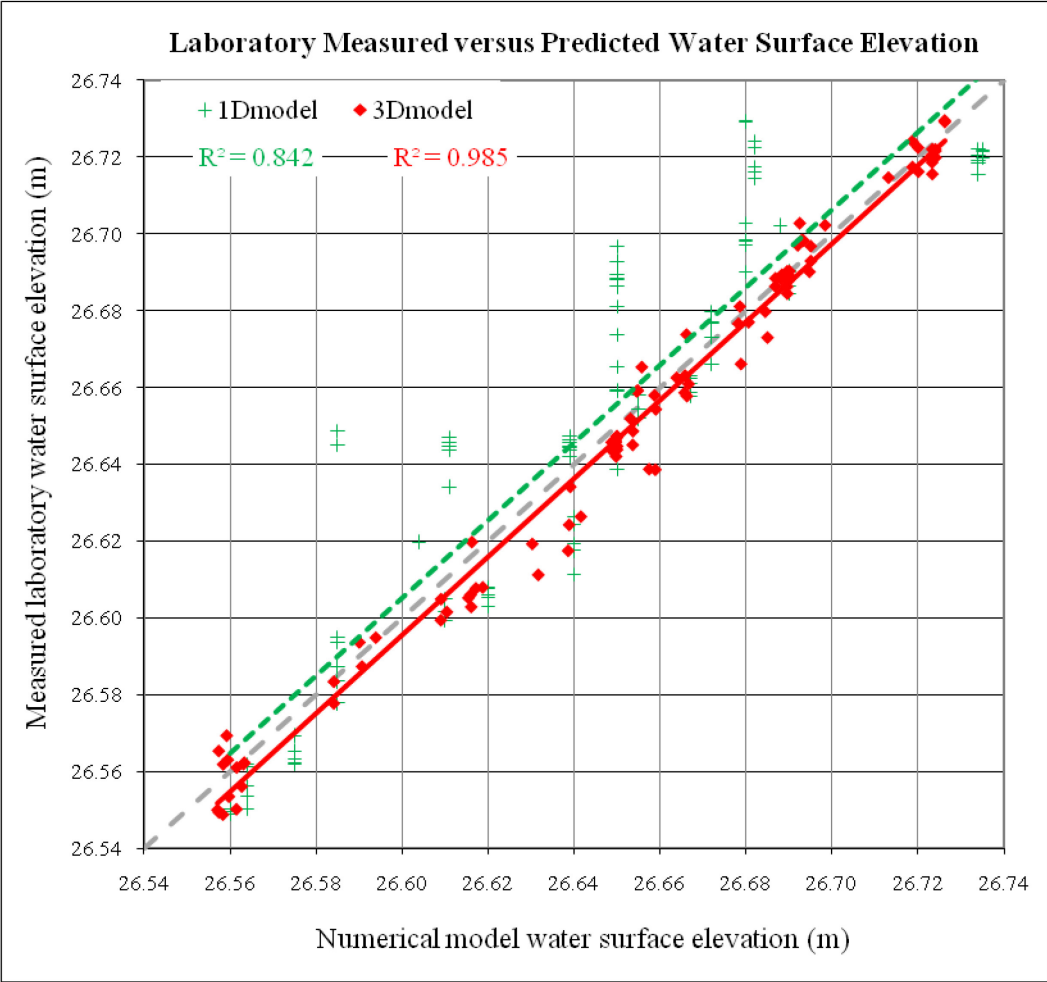
Figure 5.37 – Laboratory measured vs. predicted water surface elevation.
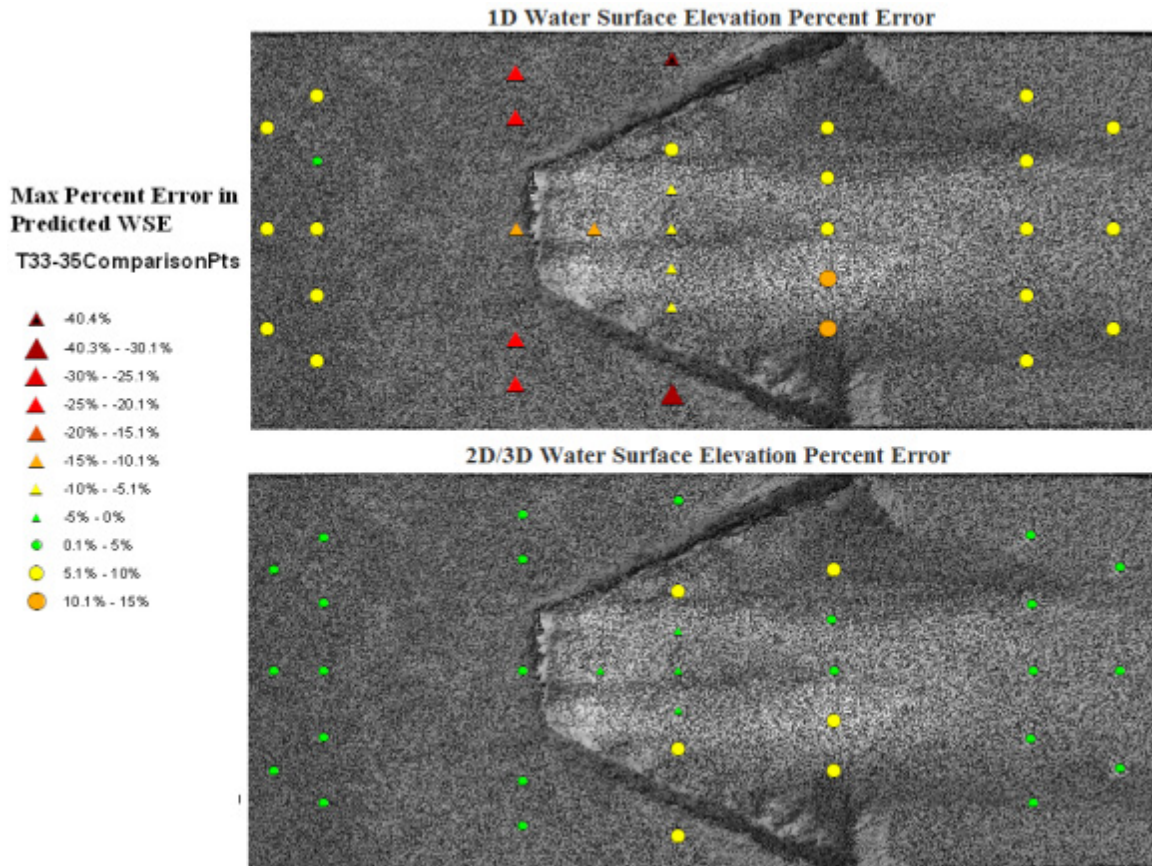
Figure 5.38 – Numerical model water surface elevation maximum percent error comparison for laboratory tests 33-35.
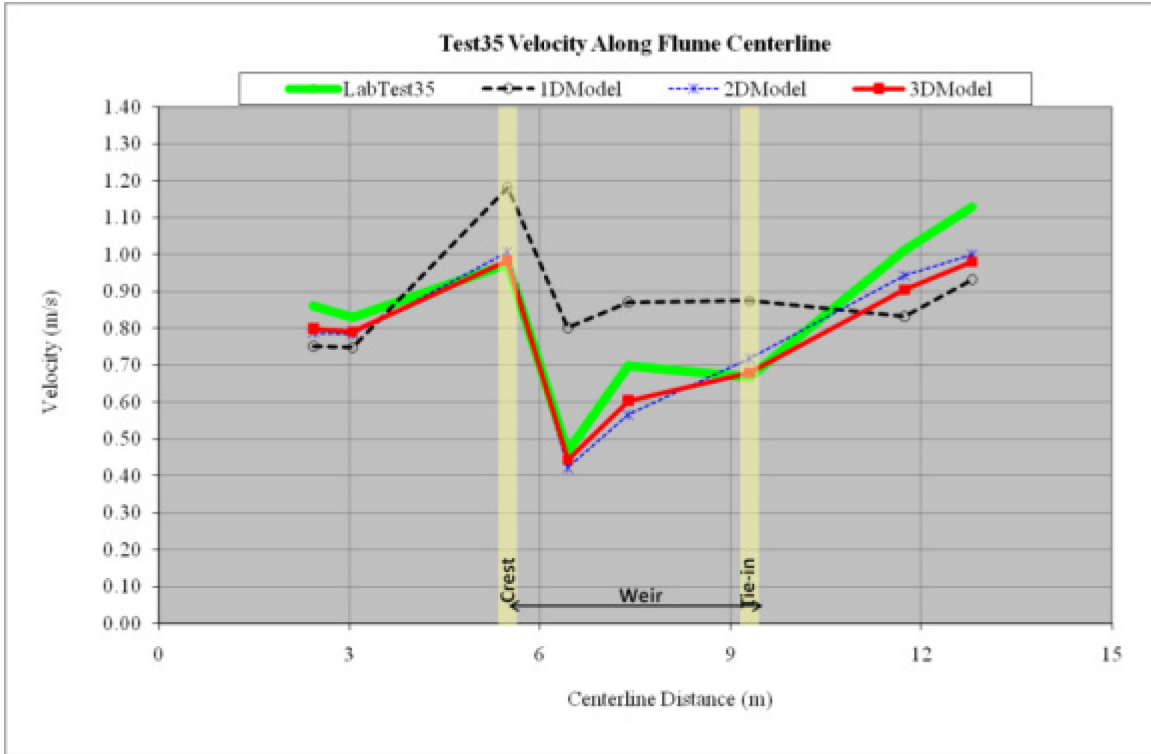
Figure 5.39 – Test 35 velocity magnitude profile along flume centerline.
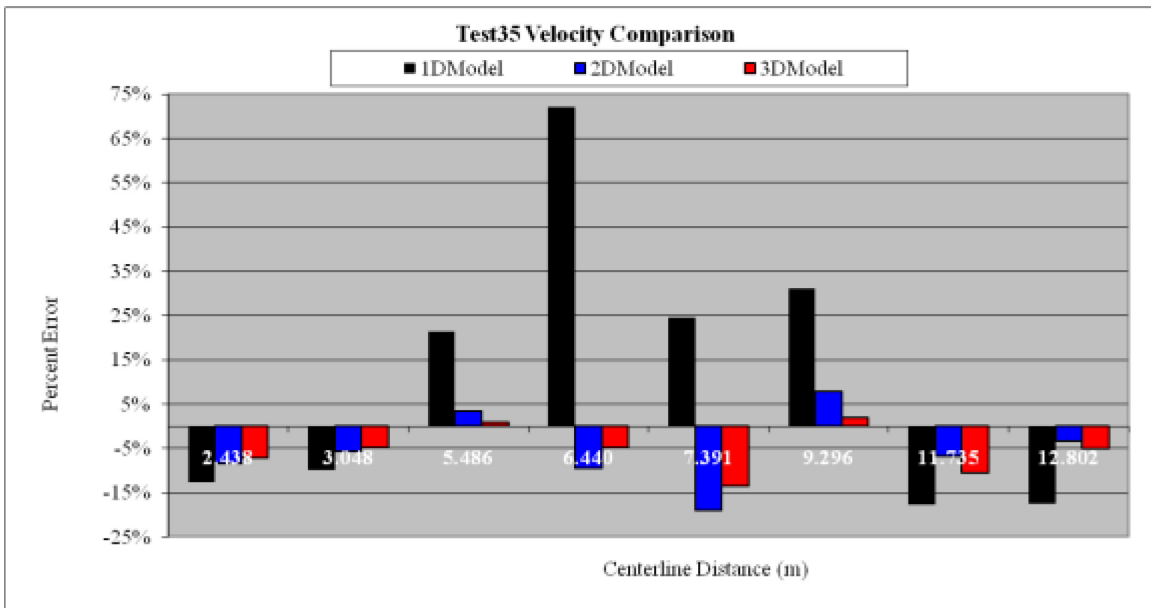
Figure 5.40 – Laboratory and numerical model velocity comparison, percent error along centerline for test 35.

Figure 5.41 shows the laboratory (true) velocity measurements plotted against predicted ones from the numerical models for the same locations. The linear correlation coefficients of 0.06 for the 1D model, 0.79 for the 2D model, and 0.78 for the 3D model show that both the 2D and 3D model provide relatively good

overall agreement (with no regard to the spatial component in the data) between predicted and measured values.
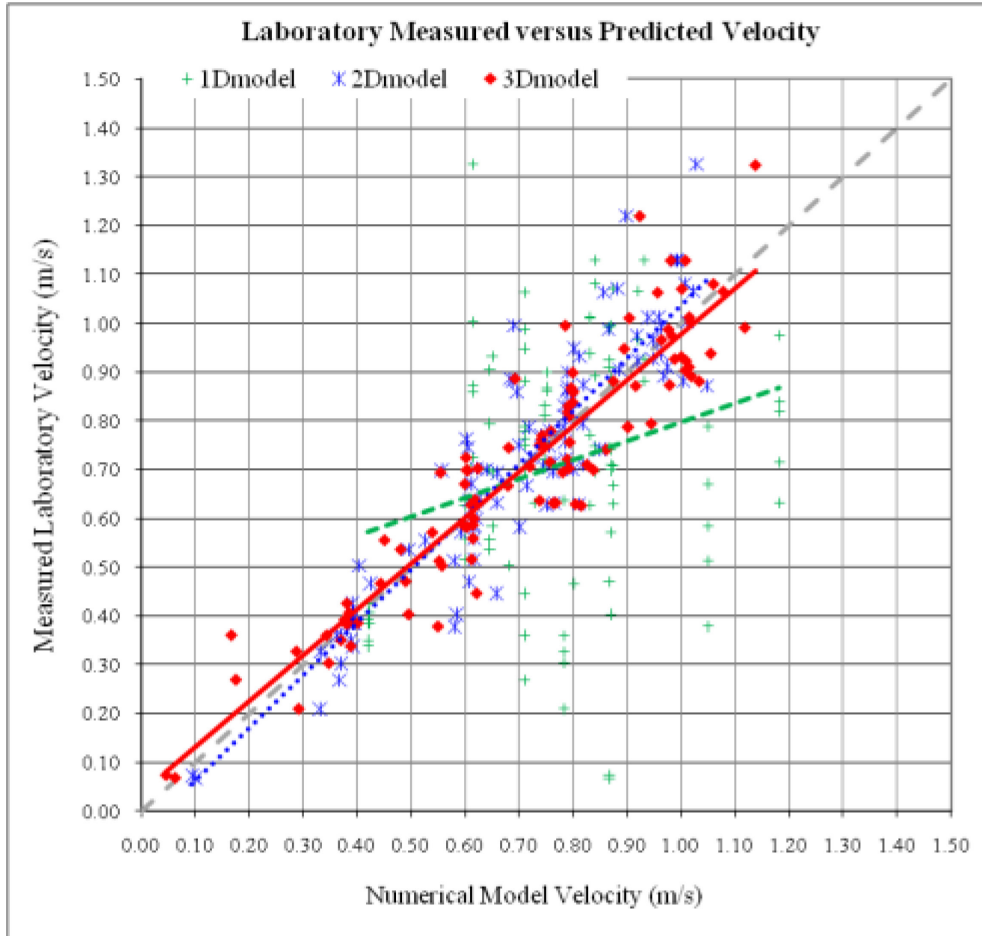
Figure 5.41 – Laboratory measured vs. predicted velocity for laboratory tests 33-35.

When the spatial component of the data is considered (Figure 5.42), it is evident that the 3D model provides a much better prediction of the velocities near the crest of the structure. While the maximum error in the predicted velocity from the 2D and 3D models is around 53 percent (0.19 m/s) and 39 percent (0.13 m/s) respectively, the 1D model has a much greater error at 177 percent (0.57 m/s). A 1D cross section model for rock weirs can meet either water surface requirements or velocity requirements, but not both. The 1D model is not able to properly simulate the vertical velocity vectors and associated energy dissipation that occur downstream of the structure crest. Additionally, the 1D model computes an average cross section velocity and therefore cannot account for the lateral variations in velocity caused by the redirection of flow over the weir crest.

Box and whisker plots showing the magnitude of 1D, 2D, and 3D model percent error in velocity are presented in Figure 5.43. It is evident from the comparison that of the three numerical models, the 3D model provides a better overall

prediction of the velocities throughout the study reach with 95 percent confidence interval of 3–11 percent compared to 6–14 percent for the 2D model and 13–28 percent for the 1D model.
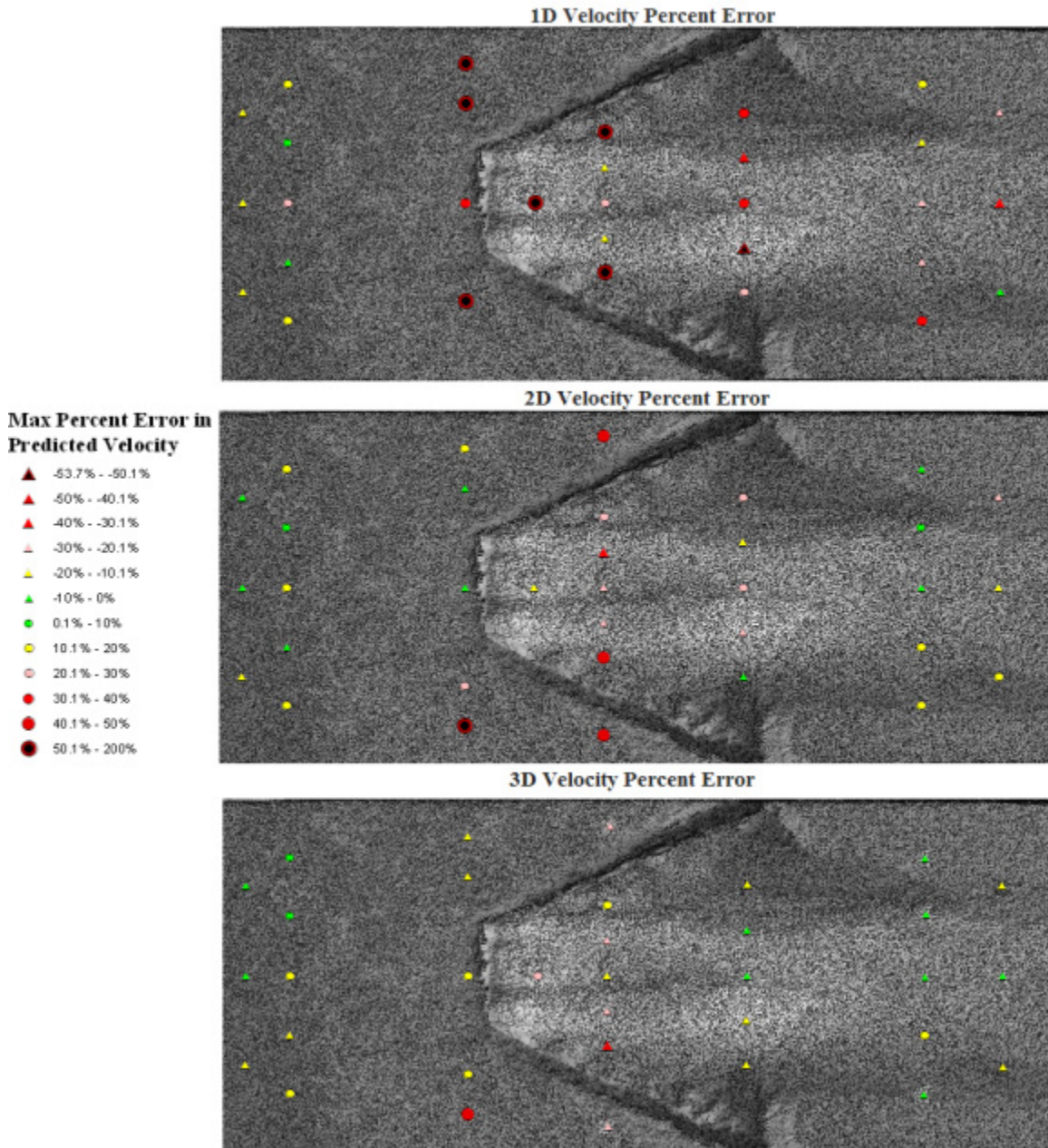
Figure 5.42 – Laboratory and numerical model velocity maximum percent error comparison.
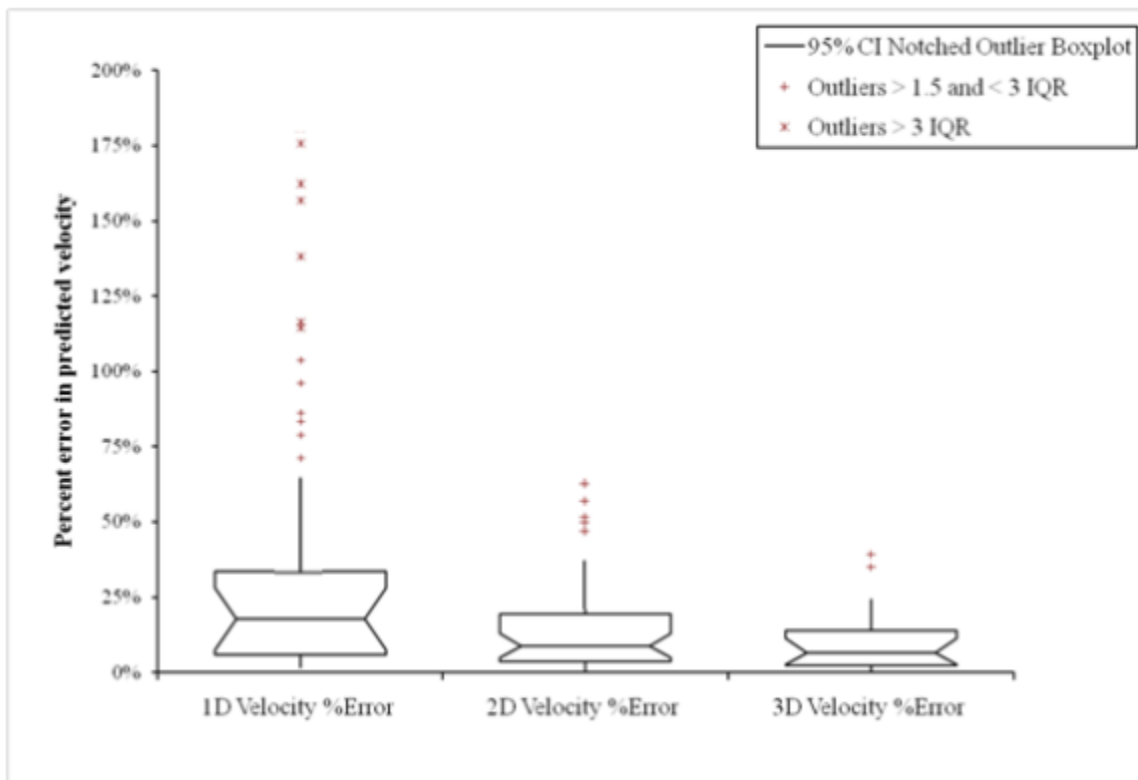
	n	Min	1st Quartile	Median	95% Confidence Interval	3rd Quartile	Max	IQR
1D Velocity %Error	102	0.07%	5.82%	17.88%	13.40% to 28.10%	33.43%	177.84%	27.61%
2D Velocity %Error	102	0.52%	3.65%	8.69%	5.75% to 13.85%	19.41%	53.46%	15.76%
3D Velocity %Error	102	0.16%	2.30%	6.64%	2.68% to 11.29%	13.91%	39.20%	11.61%

Figure 5.43 – Laboratory U-weir velocity comparison, percent error magnitude box-plot with maximum 1.5 interquartile range (IQR) and 95% confidence interval (CI) for 1D, 2D, and 3D models.

5.6.3 Summary of 1D, 2D, and 3D Numerical Model Comparisons

The comparisons of the three numerical modeling methods (1D, 2D, and 3D) presented above provide assistance to designers in selecting the appropriate numerical modeling method based on its applicability, limitations, and ability to meet project goals. Each of the numerical models provides varying degrees of information related to water surface elevation and velocity patterns associated with river spanning rock structures.

Depending on the complexity of the site and overall project goals, designers can use this information to select the numerical model (1D, 2D, or 3D) that provides them with the appropriate level of data to compare and select the structure geometry that provides the best hydraulics and local flow conditions for meeting their specific project needs.

One dimensional modeling is recommended to be applied to a channel reach to determine average channel hydraulics prior to installation of a rock weir or at a

sufficient distance upstream or downstream of the structure to fall outside of the hydraulic influence of the structure. One-dimensional models can also be used to compute the change in the water surface elevation upstream of the structure crest. One-dimensional modeling results of channels prior to rock weir installation can be used as input to backwater model equations and scour prediction equations in design as discussed in Sections 7.2 and 7.4.

Two-dimensional modeling is recommended at a minimum for determining anticipated water surface elevations, particularly when complex flow patterns involve an irrigation channel or diversion structure. Finally, although time-intensive, 3D models provide the greatest resolution of flow and velocity patterns anticipated for installation of a rock weir and should be considered necessary when a project requires greater detail on these flow features. As the order of modeling increases, the amount of input data and cost associated with analyses also increases. However, generic mesh generators can be applied in cases where data collection is not possible and can provide some comparative information across alternative geometries (e.g. Notches in Section 7.8). While analysis costs may be difficult to justify compared with the cost of installation of a rock weir, the consequences and associated cost of failure should also be considered. More discussion of uncertainty in structure success is provided in Section 8.3.

6 Sedimentation and Scour

The purpose of this chapter is to describe sediment transport and scour-related questions that may need to be addressed during initial investigations of river spanning rock weir design and to document the data needs and methods to address those questions. In addition, this chapter details uses, limitations and advantages of three methods for collecting bed material data, which are critical input to trend evaluation, model, and scour depth prediction.

6.1 Questions Related to Sedimentation and Scour

Understanding what questions to address in determining if an instream structure is suitable for a specific river or specific reach within a river is often challenging. While each river is unique and may require varying degrees of sediment transport and scour analysis, some common questions should be addressed in all studies. Sometimes analyses require purely qualitative investigations, while in some rivers, the amount of investigation required to answer sediment-related questions requires detailed investigation and long-term monitoring. The following is a list of questions related to sedimentation and scour that should be discussed before installing a rock weir structure in a river.

1. How has the channel position and bed elevation changed over time at the proposed site of the river spanning rock structure installation?

2. How is sediment transported through the reach where river spanning rock structures are proposed for installation? Is there a high sediment load? Based on the size distribution of bed material and sediment load through the reach of interest, what are the implications on the stability of the structure?

3. What depth of scour is anticipated below the structure with the distribution of sediment sizes present within the channel?

4. Is the channel armored within the vicinity of the proposed structure location, and how will this impact scour depth estimates and sediment movement through the reach?

5. Is the sediment within the channel comprised of a sufficient proportion of fines to fill void in the rocks used in the structure?

6. Based on the slope and the sediment sizes, will a depositional zone form just upstream of the structure? How will this depositional area potentially impact nearby diversion channels or structures?

7. How will the incoming sediment impact the development of a pool downstream of the structure?

6.2 Bed Material Data Collection

Bed material information is a critical component of most river restoration or rehabilitation projects, including those relating to the installation of in-channel structures.

The previous questions and many others can be answered through analyses that incorporate size distributions of bed material. In some cases, bed material size distributions alone can address specific questions, while in others, the bed material is one of many inputs to an equation or model. Three methods of sampling bed material are discussed in this document, each having its own advantages and limitation:

1. Photographic documentation of bed material that can be post-processed in the office

2. Pebble count

3. Sieving

6.2.1 Sampling Techniques

The following sections discuss the uses, advantages, and limitations of each of three methods for sampling bed material. Detailed descriptions of the sampling methodology and guidelines for selecting appropriate sampling locations are provided in Appendix B.

6.2.1.1 Photogrammetric Sediment Sampling

Photogrammetric sediment sampling offers a unique opportunity to sample a much greater spatial area at a lower cost and in a faster manner than traditional means. Additionally, the digital image processing removes biases and errors related to human participation that are present in traditional pebble count methods. Bias leads to a lack of confidence in study results making management decisions difficult and unclear. Recent advancements in processing of photogrammetric sampling suggests that digital imaging of sediment samples may be able to perform with precision equivalent to a pebble count in less than one-sixth of the time (Graham et al., 2004). While the advancements in photogrammetry appear to have some application across all aspects of sediment assessment, the most promising appears to be related to evaluating changes in sediment gradations resulting from habitat restoration actions.

Photogrammetric sampling allows several samples to be quickly documented in the field for post-processing in the office. However, post-processing in the office requires edge-detection software, which has been successfully applied to gravel and cobble bed materials, or pixel intensity software, which has been successfully applied to sand bed materials (and is not discussed in detail here). Edge detection software can range in price from $2,000 to $6,000 per license, the more expensive of which appears to have advanced manual controls, allowing the user to correct edges that have been misinterpreted by the software. Post-processing of each photograph may take anywhere from 10 minutes to 1 hour in the office,

depending on how much time is needed to correct for discrepancies in the edge detection due to shadows, pock marks in the sediment, and angularity of the material. Investigations into the use of edge-detection photosieving for underwater samples and fine sediments suggests that the technique appears to be most useful when applied to exposed, unvegetated, dry gravel and cobble bars (Gordon et al., 2010).

While the photosieving technique has been effective in determining the size distributions of surface sediments, the subsurface sediment distributions are not easily obtained through photogrammetric sampling. Size-gradations based on photogrammetric sampling is generally limited to answering questions related to how sediment gradations change over time (i.e. monitoring) and how size gradations vary throughout a specific reach of interest. Results from photosieving could be used as input to simple scour or sediment transport equations, but is less useful for modeling system and reach-scale transport of surface and subsurface materials.

6.2.1.2 Pebble Count Methods

Pebble counts are most commonly used to determine summary statistics regarding particle-size of gravel and cobble surface sediments on dry bars or within the channel. Data from pebble counts are most often applied to develop a validation data set for sieve samples, or to supplement the sieve data set with additional locations when there are budget and/or time constraints. Pebble counts cannot provide accurate particle size distributions for fine sediments, typically less than 4 mm. Although pebble counts are the most common method of determining size distributions of particle samples (by frequency), several sources of error are associated with the sampling technique that may introduce error into the data in which small differences in particle-size distributions lead to substantial differences in modeling and/or monitoring results. Outlined by Bunte and Abt (2001), the most common sources of error include (1) operator bias toward large particles, (2) operator error and/or bias in site identification and sampling scheme, and (3) statistical errors associated with sample size and precision.

Pebble counts do offer a rapid assessment technique to evaluate surface grain-size statistics across multiple sites. The time required to complete a pebble count and site evaluation ranges from 0.5 to 2 hours depending on site access and sampling size. Pebble count data are most useful in analyzing differences in gradations across long reaches of river (tens of miles) and monitoring changes in average sediment sizes for gravel- and cobble-dominated river channels over time. However, when time, budget, or sampling equipment techniques limit the ability to perform volumetric sampling, pebble count data can be used as input to scour depth equations and in some sediment transport equations that only require surface sample data.

6.2.1.3 Volumetric Sieve Sampling Methods

Sieving is a robust method that accounts for all sediment sizes present in a given sample site. Sediment data collection for sieving involves two samples per site: a

surface sample and a subsurface sample, unless there is no discernable difference between the two layers.

Results of volumetric sieve samples provide the greatest amount of information regarding the sediment present in a sample with the highest accuracy. Assuming the location has been selected to adequately represent the material within a specific reach, volumetric sieving can be used (1) to determine the extent of armoring within a reach, (2) for use as input into scour equations, and (3) for use as input into sediment transport models for understanding how surface and subsurface sediments move through a reach and could potentially impact structural stability. Unlike surface sampling techniques, volumetric sampling allows for determination of sediment variation in the vertical direction.

Higher risk projects with a greater need to understand sediment transport conditions and potential sedimentation issues through rock structures should obtain volumetric sieve samples. However, acquisition of volumetric sieve sampling is time and labor intensive. When combined with pebble counts and photogrammetric sieving, the number of volumetric sieve samples that can be collected by one 3-person crew is approximately 3 or 4 per day, depending on site access. Furthermore, the samples carried off site must be sent to a laboratory for sieve analysis at costs ranging from $100 to $300 per sample.

6.3 Sediment Analysis: What Information Is Needed to Answer Specific Questions?

Sediment analyses are vital to the design of sustainable river spanning rock structures. Various sediment analysis methods and techniques assist in the design of stable rock structures to reliably meet project goals. Sediment analyses can help to determine what type of structure might be best installed in a given stream (or if one should be installed at all), the optimal location for that structure, and the expected maintenance activities and costs.

6.3.1 Scour Depth Prediction Input

For the design of river spanning rock structures, the depth of the scour has been determined to be critical to a structure's long-term stability. As the scour along the structure crest deepens over time with increased velocity and shear stresses along the bed, footers that are undermined tend to slump into the scour hole causing subsequent sliding or rolling of the header rocks.

Multiple equations have been developed to predict the maximum scour that is likely to occur as a result of instream structures, such as rock weirs. The applicability of these scour prediction equations is detailed in Section 6.4. In almost all of the equations, some understanding of the bed material downstream from the structure drop is needed to determine how resistant the channel bed will be to shear forces over the structure. In sand bed streams without cohesive

sediments, the bed material is less likely to resist plunging forces over a rock structure than in gravel-bed rivers, and smaller shear stresses will result in scour.

Common information required as input into scour equations and predictions of maximum scour protection depths includes:

- Median sediment size of surface material, d50

- Bed material gradation, d_{50}/d_{90}

- Whether the channel bed is heavily armored

While the first two can be determined through pebble counts or photogrammetric sampling, no information can be obtained regarding how the sediment sizes vary with depth, which is critical to predicting vertical scour. Understanding how sediment sizes change between the surface and subsurface layers could help predict how accurate application of a particular equation may be. This can only be accomplished through volumetric sampling of the surface and subsurface layers, which also defines how much armoring may be present.

One alternative to volumetric sampling, which is highly labor intensive, is to apply a factor of safety to the prediction of maximum scour depth and protect the structure through a deeper foundation. However, without understanding the difference between the surface and subsurface layers, there will be additional uncertainty in computing the scour depths.

One important question in determining the maximum scour depth is the amount of armoring of the channel bed. A simple surface sample, such as a pebble count, may not detect a layer of much smaller material underneath the surface layer. An overestimate of the grain size that could be mobilized and scoured along the downstream side of the structure crest could lead to a gross underprediction in the depth of anticipated scour. For this reason, channels that are suspected to be armored require a more conservative depth for scour prediction. Where possible, a volumetric sampling scheme can help to understand the difference between the surface and subsurface layers. Channel beds that are heavily armored should consider using the gradations present in the subsurface layers in scour depth prediction equations or employ a high factor of safety in the design of the foundation depth. Knowledge of the degree of armoring can help to reduce the risks associated with structural failure resulting from undermining of the footer rocks.

6.3.2 Incipient Motion

Incipient motion is described as the threshold condition between erosion and deposition (Julien, 1998). When particles of sediment resisting motion are in balance with hydraulic forces acting to move the particles, the particles are at incipient motion. Hydrodynamic forces exceeding the resisting forces cause sediment to become mobilized. Calculations of incipient motion examine the ability of varying flow conditions to mobilize and rework the sediment present in the banks, bed, and bars within the river. Incipient motion does not consider the

supply of sediment or identify the quantity of sediment moved through a given cross section of the river. However, incipient motion does determine if the material present in channel bed and bars is mobilized at specific discharges.

Incipient motion can also be used to better understand the bed coarsening, degree of channel armoring, and potential degradation that may have occurred in each reach. During the process of channel armoring, the balance of sediment load is offset and the channel bed becomes the sediment source. This is followed by degradation of the channel bed. As finer materials are transported through the system, the bed material becomes coarser until a complete layer of coarse material covers the channel bed, thereby blocking transport of finer underlying materials (Yang, 1996). When a channel is substantially armored and the thickness of the armoring layer is known, the depth of the degradation can be estimated.

Multiple criteria exist to define incipient motion. Most incipient motion criteria were derived from the determination of the forces acting on a particle. Regardless of the criteria, the majority of the methods used to define incipient motion use a standard or modified form of the critical Shields parameter or dimensionless critical shear stress (Buffington and Montgomery, 1997).

$$\tau_c^* = \frac{\tau_c}{(\rho_s - \rho)gd} \tag{6.1}$$

where τ_c^* = dimensionless critical shear stress, or Shields parameter;
τ_c = critical bed shear stress for initiation of motion;
ρ_s = sediment density;
ρ = water density;
g = acceleration of gravity; and
d = sediment particle diameter for grain size of interest.

In its original form (Shields, 1936), the commonly used Shields parameter value for initiation of motion of coarse material in rough turbulent flow is 0.06. However, Neill and Yalin (1969) and Gessler (1971) noted that Shields original critical shear stress values were too high for initiation of motion of coarse bed material. Neil (1968) recommended a critical bed shear stress of 0.03 for coarse sediment. Most recently, Sawyer et al. (2010) found that scour always occurred above the 0.045 threshold, but not the 0.03 threshold, and therefore recommended using 0.045 for the value of Shields parameter.

Numerous other widely used methods exist for initiation of motion computation. Incipient motion for bed material size larger than 2 mm can be calculated using Yang's criteria (1973) for flood conditions in gravel-bed rivers in combination with Rubey's criteria (1933) for particle fall velocity. The equation used to determine the critical diameter at which incipient motion occurs is shown below.

$$d_c = 0.0216 v_{cr}^2 \tag{6.2}$$

where v_{cr} = critical average velocity at incipient motion (m/s); and
$\quad\quad\quad d_c$ = grain size diameter at which incipient motion occurs (m).

Grain sizes smaller than the calculated critical diameter are expected to be mobilized for flow velocity assessed. Grain sizes greater than the critical diameter are assumed stable.

Measuring the bed surface material gradation is critical to the computation of incipient motion. Definitions of incipient motion range from movement of a single particle on the bed or bank surface, to mobilization of the median grain diameter on the bed surface d50, to mass movement of surface particles in all size classes (Buffington and Montgomery, 1997). Regardless of the definition, mobilization and transport of the surface layer must be considered in sediment analyses. Once the coarse surface layer begins to move, the finer subsurface material may be exposed to the flow, resulting in accelerated rates of erosion.

Methods for measurement of surface material gradation are presented in Section 6.2.

6.3.3 Transport Capacity Modeling

When the sediment supplied to a reach is known, or when multiple reaches are analyzed together, transport capacity modeling can be used to determine if a stream reach is degradational, aggradational, or in equilibrium for a given discharge or hydrograph at a specific instance in time. Information about the stability of a reach under consideration for installation of a river spanning rock structure may influence the location/placement of a structure as well as other design parameters. One approach to model transport capacity is through a program called SRH-Capacity, a numerical sediment transport model developed by the Sedimentation and River Hydraulics (Sedimentation) Group at Reclamation's Technical Service Center (TSC). The model can be used as a tool to compute sediment transport capacity, incipient motion, and annual sediment loads for given discharges (Huang and Bountry, 2009). Model capabilities include:

- Sediment transport capacity in a river reach for given reach hydraulics;

- Incipient motion hydraulics for each sediment size class;

- Annual sediment load (optional); and

- Application of multiple non-cohesive sediment transport equations to a wide range of hydraulic and sediment conditions.

SRH-Capacity input requirements are relatively simple:

1. Hydraulics for typical cross-section or river reach where sediment transport capacity is to be computed (typically output from a one-dimensional

numerical hydraulic model such as the US Army Corps of Engineers' HEC-RAS model);

2. Bed material sediment gradation;

3. Hydrology data (optional - required only for computation of annual sediment load).

SRH-Capacity employs nearly 20 sediment transport functions to compute capacity for a wide variety of hydraulic and sediment conditions. Each sediment equation was typically developed for application to a specific sediment size range and certain set of flow conditions. Sediment transport functions commonly applied to sand-sized bed material include:

- Engelund and Hansen (1972);

- Ackers and White (1973); and

- Yang (1973).

Typical sediment transport functions used in gravel bed streams include, but are not limited to the following:

- Wilcock and Crowe (2003);

- Parker (1990);

- Meyer-Peter and Müller (1948); and

- Yang (1984).

Some predictive sediment transport equations often used for rivers containing a mixture of sand- and gravel-sized sediment in the bed are listed below:

- Parker (1990);

- Wilcock and Crowe (2003); and

- Wu (2004).

All of the sediment transport functions listed in this section (along with several other functions) are programmed into SRH-Capacity. Selection of a sediment transport function should be determined by site-specific hydraulics and sediment characteristics. It is recommended that a sensitivity analysis be performed when estimating sediment transport rates by utilizing several comparable methods.

The majority of bed load equations were developed using bed surface material gradations. Volumetric samples of the bed surface material should be collected for this type of capacity analysis. Volumetric surface sampling techniques are described in Section 6.2.

An additional method requiring more effort is to compute surface, subsurface, and combination capacities in a design reach. Representative volumetric samples can be collected using methods defined in Section 6.2. If this method is chosen, care

must be taken to separate the surface material from the subsurface during data collections for sediment size analyses.

6.3.4 One-Dimensional Mobile Bed Modeling

One-dimensional (1D) mobile bed modeling allows for evaluation of the vertical changes in elevation over an extended period of time, such as a decade or more. With respect to the design of river spanning rock structures, this type of model could be used to determine the expected changes of a large reach of river over a decadal time scale. Although a river spanning rock structure's lifespan may only be on a decadal time scale, this type of analysis would help identify reaches where river-spanning rock structures have the greatest likelihood for success with respect to anticipated vertical changes in the bed elevation.

Reclamation's model, SRH-1D Version 3.0 (Huang and Greimann, 2013), is a mobile boundary model with the ability to simulate steady or unsteady flows, internal boundary conditions, looped river networks, cohesive and non-cohesive sediment transport, and lateral inflows. The inputs to SRH-1D are similar to those for SRH-capacity. However, SRH-1D tracks bed geometry changes over time and informs the supply of sediment to downstream cross sections, where SRH-Capacity only evaluates one instance in time and does not consider incoming sediment loads. To accurately predict changes in channel patterns, an upstream boundary sediment supply is necessary for varied flow rates. Because suspended and bedload sampling curves are difficult and time-consuming to collect, this model is well-suited for reaches downstream from large dams where almost no sediment is being supplied to downstream reaches or downstream from a gage station where suspended and bedload sediment have been well documented. If no such upstream boundary is available, the model extent should be far enough upstream so that the uncertainty in the incoming sediment load does not affect model results significantly. Details of SRH-1D capabilities can be found in Huang and Greimann (2013).

6.3.5 Historical Trend Analysis

For most river spanning rock structure designs, an understanding of how the river has changed over time is crucial to the structure's success. The most direct method to determine change is through measurements and documented observations. Vertical changes can be measured through quantitative comparisons of repeated topographic surveys along the channel thalweg. Another possible indicator of channel bed elevation change is a detectable change in bed material sizes over time. When the channel bed material notably coarsens, the channel may be experiencing general erosion or localized scour. Similarly, channel aggradation may be detectable through a fining of the bed material over time. Bed material can, however, be influenced by a wide range of localized or basin activities that may convolute a relationship between changes in bed material size and vertical changes in bed elevation, such as increased sediment supply due to a fire or decreased supply due to the installation of a dam.

Lateral changes in channel position are often best documented through mapping of channels using historical and current georeferenecd aerial photography. These two analyses are important in determining whether a river spanning rock structure installed at a particular location is likely to be subject to channel shifting, which may result in structure failure by flanking, or is likely to be subject to channel erosion or deposition, which may result in structure failure by burial or excessive scour. An evaluation of historical channel position and elevation is one of the most important tasks to accomplish at the beginning of an investigation into the design of a river spanning rock structures. Hydraulic and sediment transport modeling can inform upon future trends in channel change, but it is time consuming and often requires substantial data collection. Where possible, simple comparisons of current channel form and position with historical data inform upon past trends, from which the potential for change can be easily assessed.

6.4 Scour

6.4.1 Introduction

Scour downstream of rock weir structures has been identified as the predominant mode of structural failures observed in the field, commonly resulting from scour depth propagation below the footing of the weir. The absence of a foundation below the rocks comprising the weir crest can result in downstream mobilization of the structure and loss of designed function. This section details available, pertinent knowledge of the scour process downstream of grade-control structures in a brief literature review and investigates the applicability of the current state of the practice to the three-dimensional hydraulics associated with rock weirs. Approaches are introduced tailored specifically to the data collected in a Colorado State University physical modeling program, and applications and equation modifications for field scenarios are described.

6.4.2 Scour Depth Prediction Approaches

Literature regarding the prediction of scour downstream of drop structures is extensive, and the physical process has been under investigation since original work conducted by Schoklitsch (1932). Despite numerous investigations, there does not appear to be a procedure sufficiently robust to describe maximum scour depth over a wide range of conditions. Furthermore, little information is available in which methodologies have been developed specifically for A- and U-weir shapes. Bhuiyan et al. (2007) studied a W-weir installed along a model river bend where they modified results from Bormann and Julien (1991) to coincide with collected data. Bormann and Julien (1991) expanded the relationship of Mason and Arumugam (1985) to incorporate variables within their own research and to summarize equilibrium scour equations from Schoklitsch (1932) to Bormann (1988), where exponents of the equation are unique to individual studies, expressed as:

$$y_{SE} + z_d = K \frac{q^a u_o^b H_T^c y_t^d \beta^{/e}}{g^f d_s^i} \tag{6.3}$$

where $a, b, c, d, e, f,$ and i = exponents dependent upon each study;

d_s = effective grain diameter;
g = gravitational acceleration;
H_T = total energy head drop over structure;
K = coefficient dependent on diffusion and bed-material properties;
q = unit discharge over structure;
u_o = jet velocity over structure;
y_{SE} = equilibrium scour depth;
y_t = tailwater depth;
z_d = drop height; and
$\beta^{/}$ = jet-impingement angle at bed impact (Figure 6.1).

Scour depth, y_S, in Figure 6.1 will increase until the path S is sufficiently long enough to dissipate the jet energy below the transport capacity of the scour-bed substrate. At equilibrium conditions, the scour depth (y_S) is equivalent to the equilibrium scour depth, (y_{SE}). A comprehensive review of the exponents specific to each study for which Equation 6.3 applies can be found in Bormann and Julien (1991). Many of the studies Equation 6.3 encompasses, such as Mason and Arumugan (1985), are limited by their experimental usage of uniform grain distributions as noted by Machado (1987). The form of Equation 6.3 is also limited to applications where scour depth is greater than the structure drop height.
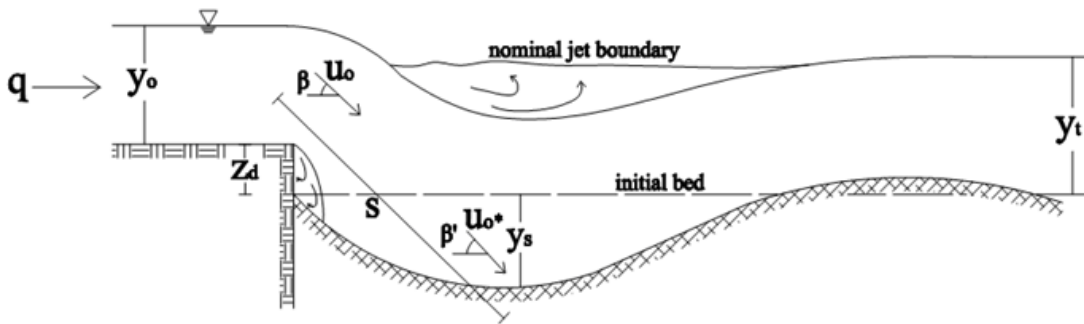
Figure 6.1 – Schematic of flow through two-dimensional grade-control structure (adapted from Bormann 1988).

Scour downstream of rectangular structures has been described by dimensional analysis of predictive variables. Gaudio and Marion (2003), expanding work by Gaudio et al. (2000), described downstream equilibrium scour for uniform grain-size distributions by performing least-squares linear regression on identified predictive variables as:

$$\frac{y_{SE}}{H_S} = 0.180 \frac{A_1}{(\Delta-1)d_{50}} + 0.369 \tag{6.4}$$

where A_1 = morphological jump, defined as the difference between initial and final slope multiplied by the length between structures;

d_{50} = mean sediment diameter;

H_S = critical specific energy over structure;

Δ = specific gravity of material; and

y_{SE} = equilibrium scour depth.

Similar dimensional analysis methodologies incorporating similar parameters as Equation 6.4 include Meftah and Mossa (2006) and Lenzi et al. (2002).

D'Agostino and Ferro (2004) proposed that scour downstream of grade-control structures in alluvial channels can be represented by the following functional relationship:

$$f\left(y_{SE}, z, b, B, y_t, H, Q, \rho_S, \rho, g, d_{50}, d_{90}\right) = 0 \tag{6.5}$$

where b = weir width;

B = channel width;

d_{50} = mean sediment diameter;

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

g = gravitational acceleration;

H = piezometric drop across structure;

Q = discharge;

ρ = fluid density; and

ρ_S = sediment density.

y_{SE} = equilibrium scour depth;

y_t = tailwater depth; and

z = drop height.

D'Agostino and Ferro (2004) applied an incomplete self-similarity theory to variables presented in Equation 6.5, accompanied by a multiple regression using data from Veronese (1937), Bormann and Julien (1991), D'Agostino (1994), and Mossa (1998) to produce the following:

$$\frac{y_{SE}}{z} = 0.540 \left(\frac{b}{z}\right)^{0.593} \left(\frac{y_t}{H}\right)^{-0.126} \left(\frac{Q}{bz\sqrt{g(\Delta-1)d_{50}}}\right)^{0.544} \left(\frac{d_{90}}{d_{50}}\right)^{-0.856} \left(\frac{b}{B}\right)^{-0.751} \tag{6.6}$$

In a similar methodology as D'Agostino and Ferro (2004), Comiti et al. (2006) produced another relationship for scour depth:

$$\frac{y_{SE}}{z} = 2.00 \left(\frac{H_S}{z} \right)^{0.590} \left(\frac{b}{B} \right)^{2.34} \left(\frac{\Delta d_{90}}{z} \right)^{-0.09} \tag{6.7}$$

where b = weir width;
     B = channel width;
     d_{90} = sediment diameter where 90 percent total is smaller by size;
     H_S = critical specific energy over structure;
     Δ = specific gravity of material;
     y_{SE} = equilibrium scour depth; and
     z = drop height.

Work produced by D'Agostino and Ferro (2004) and Comiti et al. (2006) deviate from those previously presented as they account for non-uniform sediment gradations as well as channel geometry. Structure geometry variables represented within their expressions are not included in other dimensional-analysis predictive methodologies, and the dimensionless b/B term provides consideration for three-dimensional flows due to channel contractions. However, data collected were generated from weirs installed normal to the thalweg axis and lateral flows from channel contractions that b/B may account for are relatively negligible compared to lateral flows observed through rock weirs. The definitions of weir length and fall height are ambiguous for three-dimensional structures as these parameters vary along channel length. Furthermore, critical energy at the sill is a complex parameter to determine for three-dimensional grade-control structures. Critical flow depth and velocity are generally considered a one-dimensional flow concept mathematically, based upon cross-section averaged parameters. While critical flow occurs as the jet plunges over the crest of a three-dimensional weir, noted by a downstream hydraulic jump, its location cannot be identified within a typical channel cross section. Figure 6.2 depicts a downstream view of critical flow over a U-weir crest installed in a laboratory flume.

From the literature, the majority of approaches presented approximating maximum equilibrium scour depth, although numerous and varied in their approach, were developed for the case of two-dimensional flow and incorporate parameters not directly applicable to three-dimensional grade-control hydraulics. Accordingly, methodologies tailored to the dataset collected during the physical modeling process (Section 2.2) were examined. Developed methodologies as they apply to field data are detailed in further sections.
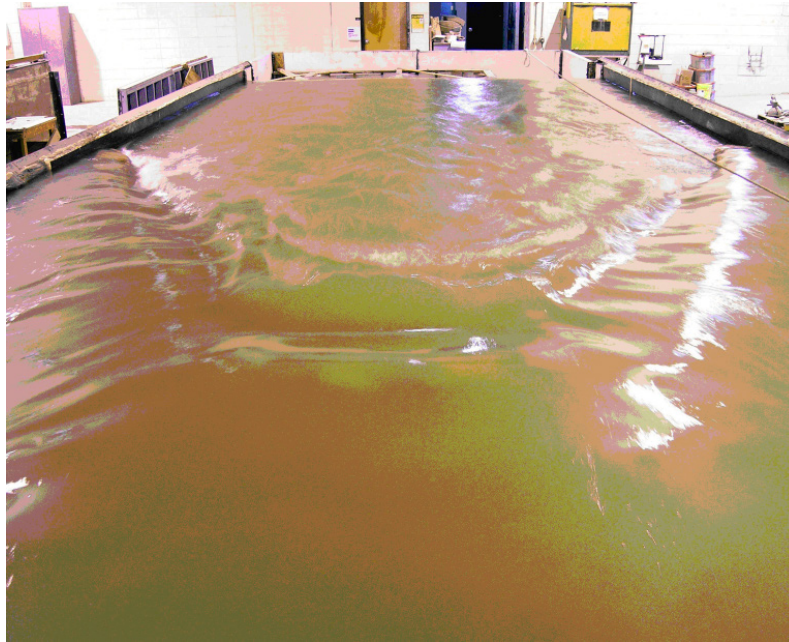
Figure 6.2 – Hydraulics associated with U-weir, downstream perspective.

6.4.2.1 *Scurlock (2009) and Scurlock et. al, (2012) Dimensional-Analysis Predictive Methodology Development*

Considering the limitations of applicability of two-dimensional scour equations from the literature to rock weirs, examination of approaches to alter existing equation formats to represent rock-weir structures, jet characteristics, and scour geometries was undertaken. After multiple approaches were considered, it was found that D'Agostino and Ferro (2004) best represented parameters manipulated during the test matrix, incorporated parameters readily available from field data, and accurately predicted equilibrium scour depth when certain modifications were introduced. In order to address challenges from the application of a one-dimensional equation to three-dimensional flows, the effective weir length and weir height were expressed as functions of the channel width, B, plan angle, θ, outer arm plan angle, ζ, and inner arm plan angle, η, for the W-weir. Figure 6.3 depicts profile and plan-view angles for each weir and presents descriptions of weir geometry related to bankfull width. An average of the weir height extending across the channel width emulated the flow contacting the weir at a single location, rather than along the length of the channel. Effective weir length was defined as the total length taken along the weir crest. For A-weir geometries, the upstream weir segment was neglected and the effective length and height were calculated from the downstream crest and arms only. The upstream portion of the structure was excluded from determination of the effective length because the main scouring forces, attributed to jet impingement and vortex formation, were generated downstream of the excluded crest.

Weir	Plan view		
		Effective length	

$$b_A = \frac{B}{3}\left(\csc\theta\sec\zeta + 2\right)$$

Mean height

$$z_A = \frac{B}{12}\csc\theta\tan\zeta + \frac{2}{3}z_d$$

Profile view

$$b_U = \frac{B}{3}\left(2\csc\theta\sec\zeta + 1\right)$$

Mean height

$$z_U = \frac{1}{9}B\csc\theta\tan\zeta + z_d$$

Profile view

$$b_W = \frac{B}{2}\csc\theta\left(\sec\zeta + \sec\eta\right)$$

Mean height

$$z_W = \frac{1}{16}B\csc\theta\left(\tan\eta + \tan\zeta\right) + z_d$$

Profile view
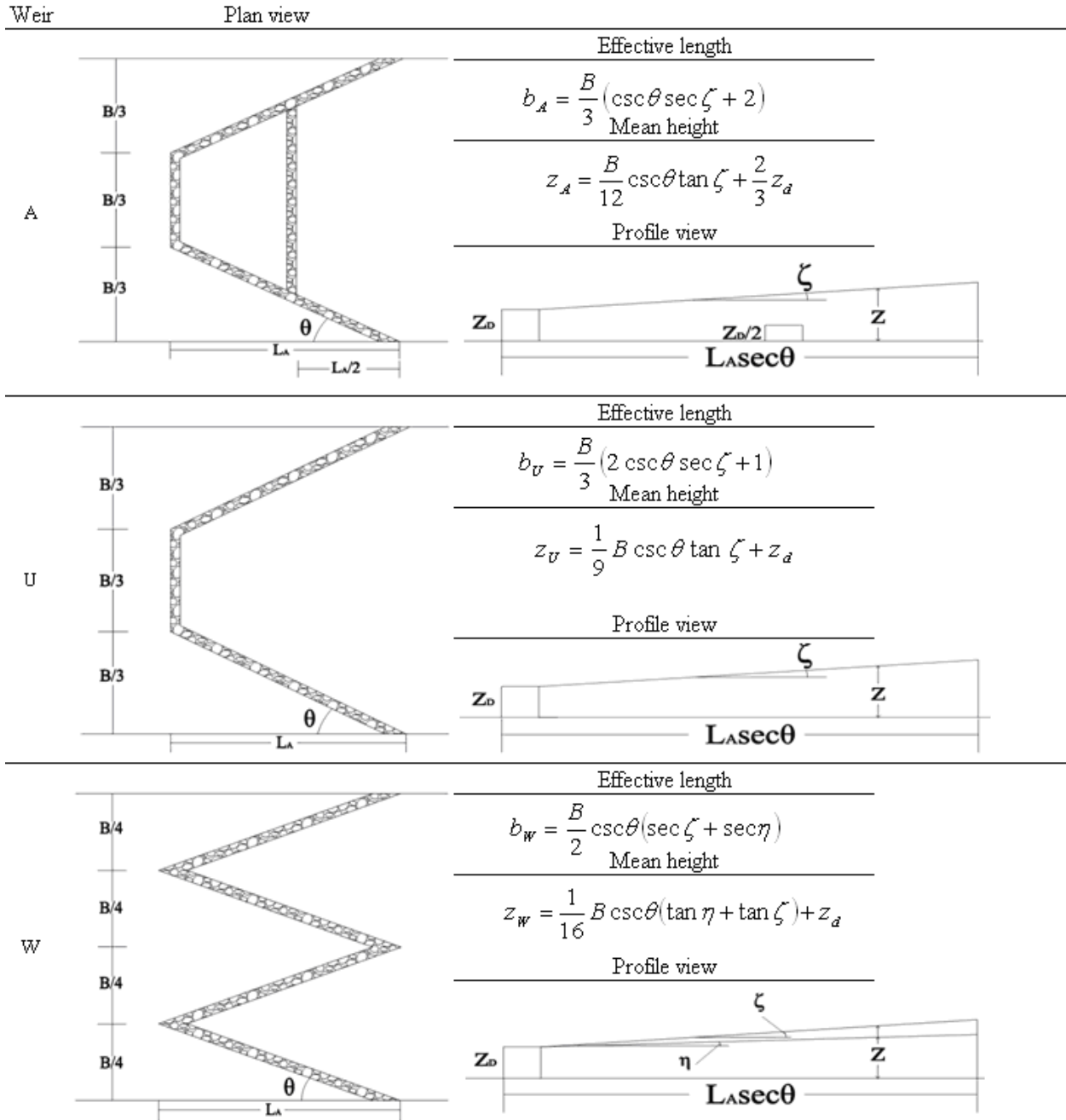
Figure 6.3 – Weir geometry schematics.

Further alteration of Equation 6.6 was found to be beneficial in the prediction of equilibrium scour depth for three-dimensional weirs. The densimetric particle Froude number, $Fr^* = Q/\{bz[g(\Delta-1)d_{50}]^{1/2}\}$, was modified to include the d_{90} grain diameter instead of d_{50}. Schoklitsch (1932), Jaeger (1939), and Eggenberger (1943) utilize d_{90} instead of d_{50} due to the presence of larger material within the scour hole that forms an armor layer in the equilibrium state. Since normal depth was chosen as the tailwater depth for each configuration and discharge, Equation

6.6 was modified to incorporate channel slope and material roughness through the Manning equation and Strickler's relationship. The piezometric head difference was calculated as the difference between the average water surface measured approximately 0.5 channel widths upstream of the weir crest and the average water surface measured 1.5 widths downstream of the crest. Substituting these changes into Equation 6.6 yields:

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

$$i = \{A, U, W\}$$

(6.8)

where a_1 through a_6 = regression coefficients;

b_i	= effective weir length;
B	= channel width;
d_{50}	= mean sediment diameter;
d_{90}	= sediment diameter where 90 percent total is smaller by size;
g	= gravitational acceleration;
H	= piezometric drop across structure;
Q	= discharge;
y_{SE}	= equilibrium scour depth;
y_t	= tailwater depth;
Δ	= specific gravity of material; and
z_i	= mean weir height above bed.

Modifications from the original work by Scurlock (2009) were made to eliminate certain U-weir tests that did not adhere to the structural specifications of the bulk of the dataset, namely the early tests in which the rock-step construction methodology had not yet been codified. A multivariate, backwards linear regression was performed on the natural logarithms of the terms in Equation 6.8 generating a_1 through a_6 regression coefficients for the A-, U-, and W-weir. Terms not significant at a 0.05 significance level were eliminated. For the U-weir analysis, the parameter d_{90}/d_{50} was determined to be significant with a large, positive exponent of 9.633. Intuition of the hydraulics governing sediment transport leads to the conclusion that this exponent should be negative such that an increase in d_{90} with a fixed d_{50} would limit scour potential, coinciding with results from D'Agostino and Ferro (2004), Schoklitsch (1932), Jaegar (1939), and others. Therefore, d_{90}/d_{50} was removed from the U-weir regression process. Equations 6.9, 6.10, and 6.11 are the laboratory calibrated results for determining maximum depth downstream of A-, U-, and W-weirs, respectively (Scurlock et al., 2012).

$$\ln\left(\frac{y_{SE}}{z_A}\right) = -16.905 + 4.078\ln\left(\frac{b_A}{z_A}\right) + 0.888\ln\left(\frac{Q}{b_A z_A \sqrt{g(\Delta-1)d_{90}}}\right) \tag{6.9}$$

$$\ln\left(\frac{y_{SE}}{z_U}\right) = -53.649 + 13.548\ln\left(\frac{b_U}{z_U}\right) + 0.481\ln\left(\frac{Q}{b_U z_U \sqrt{g(\Delta-1)d_{90}}}\right) - 11.329\ln\left(\frac{b_U}{B}\right) \tag{6.10}$$

$$\ln\left(\frac{y_{SE}}{z_W}\right) = 2.618 + 0.831\ln\left(\frac{Q}{b_W z_W \sqrt{g(\Delta-1)d_{90}}}\right) - 1.649\ln\left(\frac{b_W}{B}\right) \tag{6.11}$$

The coefficients of determination are 0.978 for Equation 6.9, 0.965 for Equation 6.10, and 0.913 for Equation 6.11. For proposed equations, the root mean squared error was 0.032 m (0.11 ft). Predicted maximum equilibrium scour depth is compared to observed scour depths in Figure 6.4. Comparative results for all tests between the original D'Agostino and Ferro (2004) method and the series of proposed dimensionless equations are depicted in Figure 6.5. As observed in Figure 6.5, D'Agostino and Ferro (2004) uniformly over-predicted maximum equilibrium scour depth for the tested structures, while Equations 6.9, 6.10, and 6.11 do not show significant over- or under-prediction. Figure 6.6 depicts the results of Equations 6.9, 6.10, and 6.11 as compared with the most accurate approach from Schoklitsch (1932) using cumulative square error as the measure. Mean percent error between observed and predicted maximum scour depths for proposed methodologies was 10.45 percent. Schoklitsch (1932) generated a mean percent error of 37.12 percent and D'Agostino and Ferro (2004) produced a mean error of 183.05 percent.
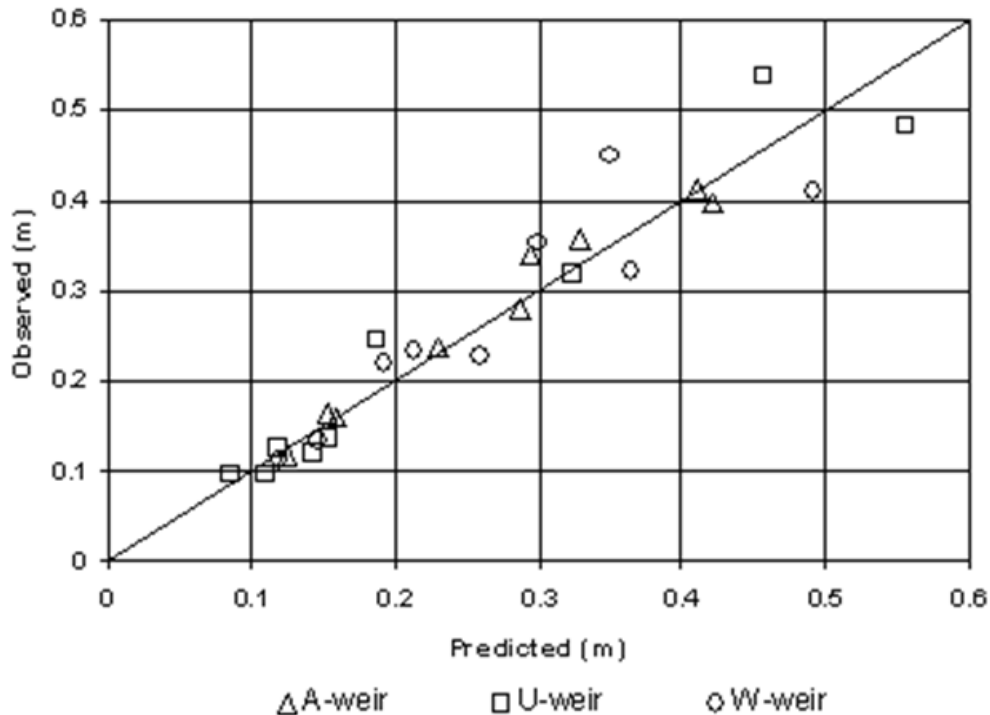
Figure 6.4 – Results of laboratory calibrated scour-depth equations.
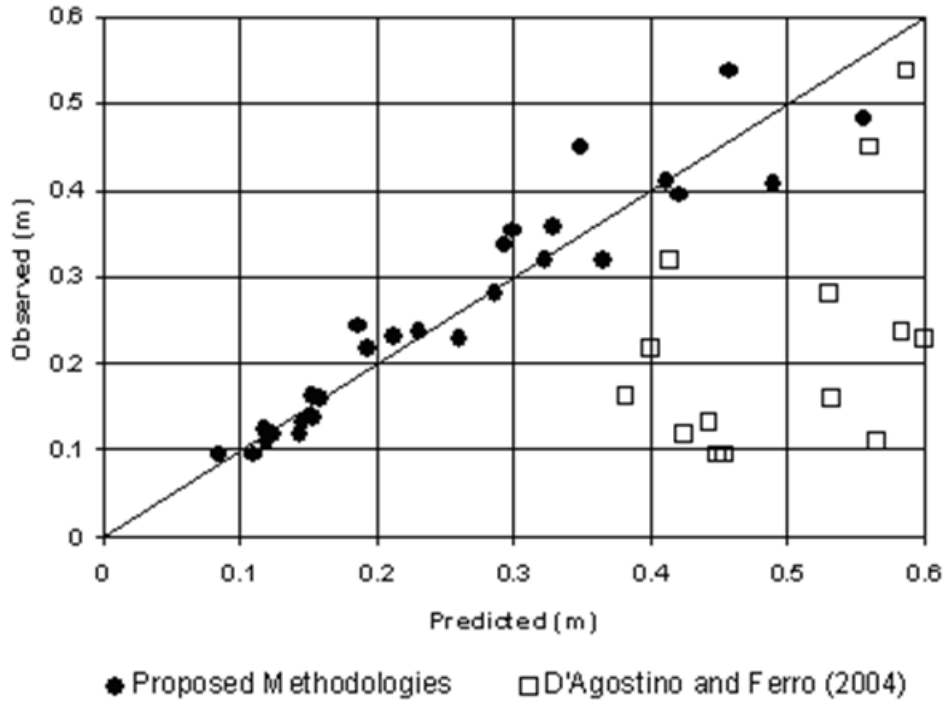
Figure 6.5 – Proposed scour depth methodologies compared with D'Agostino and Ferro (2004).
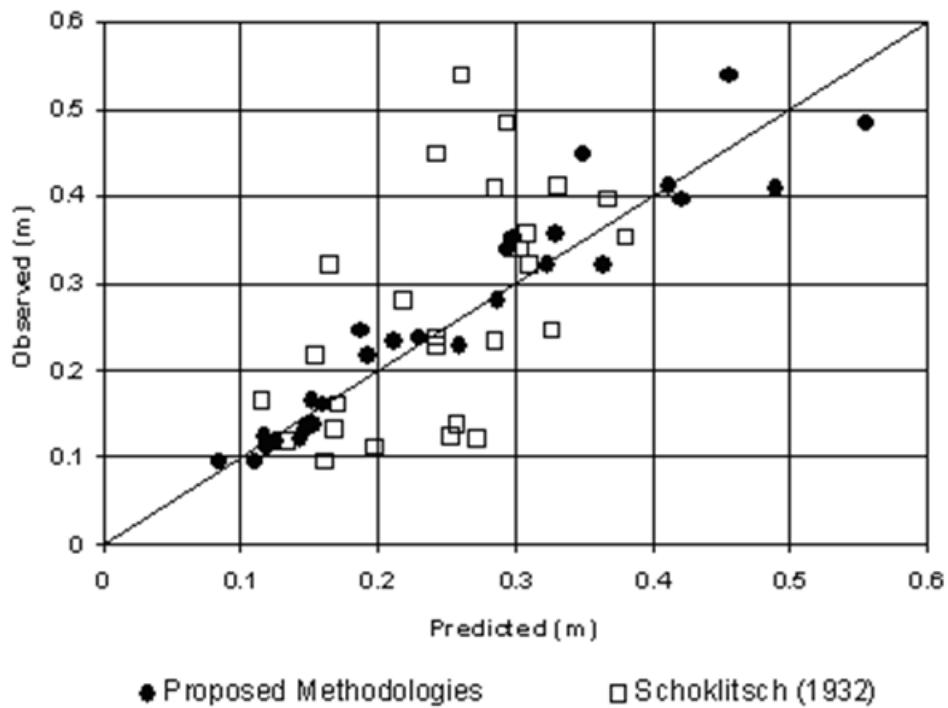
Figure 6.6 – Proposed scour depth methodologies compared with Schoklitsch (1932).

Equations 6.9, 6.10, and 6.11 predict laboratory data to a greater degree of accuracy than other approaches found within the literature; however, the small amount of data available for equation development and associated parameter value ranges place limitations on the applicability of Equations 6.9, 6.10, and 6.11 for design. Equations are sensitive to ranges of parameter values used for calculation. If values fall outside of ranges tested, in such cases as noted by Reclamation (2009b), extrapolation error may be substantial. Table 6.1 depicts boundaries of applicability for the dimensionless terms presented in the developed equations where π terms are defined as the ratios of Equation 6.8 moving left to right. Additional field or laboratory information supplementing the existing database would expand the applicability of proposed laboratory calibrated methodologies. Equations 6.9, 6.10, and 6.11 are not recommended for application outside of the limits of the laboratory data ranges. Even if the values fall within the laboratory ranges, the particular combination that is present in the field could be such that the result is not reasonable.

Table 6.1– Tested parameter ranges, where π terms are defined as the ratios of Equation 6.8 moving left to right.

Shape	$y_{SE\text{-}observed}$ M	π_1	π_2	π_3	π_4	π_5
A						
Max	0.411	74.216	2.149	2.894	1.876	1.414
Min	0.118	70.069	0.717	0.875	1.435	1.334
U						
Max	0.548	102.098	4.113	2.323	1.876	2.042
Min	0.064	69.640	0.980	0.527	1.620	1.733
W						
Max	0.448	129.348	4.018	1.614	1.876	2.931
Min	0.112	120.508	1.015	0.399	1.435	2.231

6.4.2.2 Re-Evaluation of the Laboratory Data based upon Physical Processes

While the equations developed by Scurlock (2009) and Scurlock et al (2012) statistically perform well compared with other existing scour prediction equations and are tailored to rock weirs specifically, the exponents in the equations are much too high and the directionality of the terms presented in the equations do not always make physical sense. For example, the non-transformed version of Equation 6.10 is represented mathematically as:

$$\left(\frac{y_{SE}}{z_U}\right) = 5.018*10^{-24}\left(\frac{b_U}{z_U}\right)^{13.548}\left(\frac{Q}{b_U z_U \sqrt{g(\Delta-1)d_{90}}}\right)^{0.481}\left(\frac{b_U}{B}\right)^{-11.329} \qquad (6.12)$$

In this equation, the term $\left(\dfrac{b_u}{z_u}\right)^{13.548}$ indicates that as the structure drop height (z_u)

increases just a small amount, the predicted equilibrium scour depth (y_{SE})

decreases by a 12.548 power. Furthermore, $\left(\dfrac{b_u}{z_u}\right)$ and $\left(\dfrac{b_u}{B}\right)$ are strongly

correlated and are offsetting one another. Understandably, the interaction with the other variables results in this statistical response. A re-evaluation of the laboratory tests was completed to develop a set of equations that make physical sense and are statistically significant, even if the strengths of the relationships are weaker than those produced using all terms. Furthermore, inclusion of all of the five terms evaluated in D'Agastino and Ferro (2004) and Scurlock (2009) into a single equation to define equilibrium scour depth creates difficulty in ensuring that the behavior is reasonable over the full range of expected conditions. One desired outcome of this analysis is a set of equations that can be applied to an expanded number of conditions and improved over time. Because the data set is small an equation that combines all three structure types was also desired. Multiple equations were produced from the re-evaluation with all terms having a significance of p<0.01 and variance inflation factors less than 10 (indicating independence of variables). However, to ensure reasonable behavior of the terms in the equations that is physically meaningful and to eliminate correlation of terms, the presented equations were reduced to only include the third term of Equations 6.8, a modified form of the densimetric Froude number, such that:

$$\frac{y_{SE}}{z_i} = a_1 \left(\frac{Q}{b_i z_i \sqrt{g(\Delta - 1)d_{90}}} \right)^{a_2}$$

$$i = \{A, U, W\}$$

(6.13)

where a_1 and a_2 = regression coefficients;
      b_i = effective weir length;
      d_{90} = sediment diameter where 90 percent total is smaller by size;
      g = gravitational acceleration;
      Q = discharge;
      y_{SE} = equilibrium scour depth;
      Δ = specific gravity of material; and
      z_i = mean weir height above bed.

Scurlock et al. (2012) removed 10 U-weir test cases from the analysis originally presented in Scurlock (2009) because the cases either had an insignificant (~0) head drop resulting in inconsistent scour patterns or because the tail water conditions were not set to normal depth as had been done with all other test cases affecting the use of the second pi term of Equation 6.8, y_t/H. To include the greatest range of conditions within the laboratory and because the second term was excluded for this analysis, all structures used in Scurlock (2009) were included in the development of the resulting equations for each structure and for

all structures combined, as shown in Equations 6.14, 6.15, and 6.16 for A-, U-, W-weirs respectively. These equations were developed through linear regression on the natural logs of the values of the terms.

$$\ln\left(\frac{y_{SE}}{z_i}\right) = 0.547 + 0.899 \ln\left(\frac{Q}{b_A z_i \sqrt{g(\Delta-1)d_{90}}}\right) \tag{6.14}$$

$$\ln\left(\frac{y_{SE}}{z_i}\right) = 0.677 + 0.580 \ln\left(\frac{Q}{b_U z_i \sqrt{g(\Delta-1)d_{90}}}\right) \tag{6.15}$$

$$\ln\left(\frac{y_{SE}}{z_i}\right) = 1.217 + 0.768 \ln\left(\frac{Q}{b_W z_i \sqrt{g(\Delta-1)d_{90}}}\right) \tag{6.16}$$

The composite equation developed for all structures combined is presented in Equation 6.17.

$$\ln\left(\frac{y_{SE}}{z_i}\right) = 0.764 + 0.624 \ln\left(\frac{Q}{b_i z_i \sqrt{g(\Delta-1)d_{90}}}\right) \tag{6.17}$$

$$i = \{A, U, W\}$$

The coefficients of determination (R^2) and root mean square errors (RMSE) for Equations 6.14 through 6.17 are shown in Table 6.2.

Table 6.2 – R^2 and RMSE values for predicting scour with Equations 6.14 through 6.17.

Equation Purpose	Equation number	Coefficient of Determination (R^2)	Root Mean Square Error (RMSE, ft)
A-weir Scour	6.14	0.927	0.13
U-weir Scour	6.15	0.209	0.55
W-weir Scour	6.16	0.699	0.27
Rock-weir Scour	6.17	0.368	0.44

A visual representation of the strength of the relationships between $\ln\left(\frac{y_{SE}}{z_i}\right)$ and

$\ln\left(\frac{Q}{b_A z_i \sqrt{g(\Delta-1)d_{90}}}\right)$ for each of the structure types is illustrated in Figure 6.7.

These relationships are clearly not as strong as those presented in Equations 6.9, 6.10, and 6.11. However, they are more likely to be applicable to a wider range of conditions and behave in a physically meaningful way.
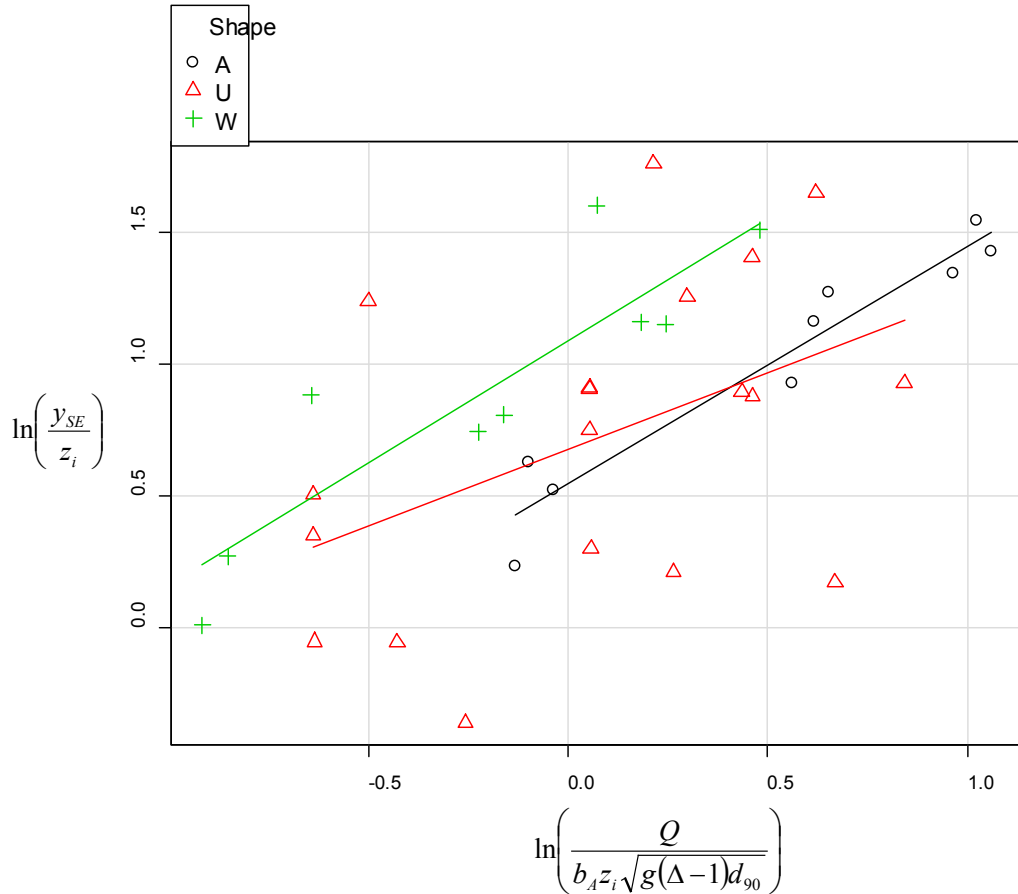
Figure 6.7 – Scatterplot of the natural log of y_{se}/z_i on the y-axis and natural log of the modified densimetric Froude number on the X-axis. The regressions equations 6.14 to 6.16 are shown as lines.

6.4.2.3 *Pagliara and Palermo (2013) Dimensional Analysis of Scour below Rock Grade Control Structures*

Recent work by Pagliara and Palermo (2013) continued the exploration of scour depth relating to rock grade control structures. They studied flow and scour patterns through both rock grade control structures and stepped gabions in a flume. In the analysis of scour mechanisms and of scour hole length, they determined that the flow regime on the structures largely influences maximum scour hole depth and they developed simple empirical relationships to predict maximum scour hole depth and length based on dimensional analysis, which are presented below:

$$\frac{z_{max}}{E_o} = 3.28A_{50}^3 - 6.28A_{50}^2 + 4.74A_{50} - 0.95 \qquad (6.18)$$

Equation 6.18 is valid for rock grade control structure without filter material at the upstream end.

$$\frac{z_{max}}{E_o} = 3.46A_{50}^3 - 6.96A_{50}^2 + 5.42A_{50} - 1.09 \qquad (6.19)$$

Equation 6.19 is valid for rock grade control structures with pervious filter material upstream of the structure.

$$\frac{z_{max}}{E_o} = 13.46A_{50}^3 - 22.02A_{50}^2 + 13.12A_{50} - 2.32 \qquad (6.20)$$

Equation 6.20 is valid for rock grade control structure with filter material upstream of the structure and an impervious cover layer.

The variable definitions are:

z_{max} = maximum scour depth;
E_o = total energy head upstream of the structure;
A_{50} = $q/[H[gd_{50}(\Delta\rho/\rho)]^{0.5}]$;
q = unit discharge;
H = structure height;
g = acceleration due to gravity;
d_{50} = median grain size;
$\Delta\rho$ = $\rho_s - \rho$;
ρ_s = density of bed material; and
ρ = density of water.

To apply this equation to design, one would be required to estimate the energy drop through the structure using a stage-discharge rating curve or through hydraulic modeling. Estimation of upstream water-surface and energy head through various methods is expounded in Section 7.2.

As part of their analysis, Pagliara and Palermo also developed a simple non-dimensional equation to estimate scour hole length downstream from a grade control structure. To compute scour length, they proposed the following equation:

$$\frac{l_s}{E_o} = 3.16\frac{z_{max}}{E_o} \qquad (6.21)$$

where l_s = scour length.

6.4.2.4 Pagliara and Kurdistani (2013) Dimensional Analysis of Scour below Cross Vanes

Pagliara and Kurdistani (2013) further explored scour development below cross vanes in a small flume (0.342 m wide by 0.7 m long) under clear water conditions using uniform plastic material with a particle size of 3.52 mm and relative density of 1.29. They experimented with I- and U-shaped structures with slopes between 1 and 5 percent and developed new analytical functions derived from non-dimensional analysis to predict maximum scour depth and scour length below cross vanes. Equations 6.22 and 6.23 can be used to predict scour depth and length, respectively.

$$z_m/h_{st} = 1.3\left[\left(l/B\right)\cdot\left(1+S_o^{0.1}\right)\right]^{0.9}\eta^{0.4} \quad 0.05 < \eta < 15 \qquad (6.22)$$

$$l_m/h_{st} = 8.8\left[\left(l/B\right)\cdot\left(l-S_o^{0.4}\right)\right]^{1.9}\eta^{0.6} \quad 0.05 < \eta < 15 \qquad (6.23)$$

where z_m = maximum depth of scour;
h_{st} = height of the structure defined as the average height of the stones top;
l = length of the structure;
B = channel width;
S_o = slope of the initial bed;

$$\eta = F_d^2 \cdot \Delta y/h_{st}$$

$F_d = Q/\{l\cdot h_{st}[g(G_s-1)d_{50}]^{0.5}\}$ densimetric Froude number;
Q = flow discharge;
g = acceleration due to gravity;
G_s = bed material density divided by the density of water (relative density)
Δy = water surface elevation difference upstream and downstream of the structure; and
l_m = scour length.

The authors tested the dataset presented in Scurlock (2009) and found that all data fit within ± 20 percent of the predicted value. Equations 6.22 and 6.23 require a method of approximating the upstream water surface elevation for computation of Δy. Details of methodologies for backwater estimations are provided in Section 7.2.

6.4.2.5 Reclamation (2009b) Equation Development for Field Application

Reclamation (2009b) investigated the modification of the D'Agostino and Ferro (2004) relationship of Equation 6.6 to incorporate the geometric relationships of mean weir height and effective weir length introduced by Scurlock (2009) while maintaining the original regression coefficients from D'Agostino and Ferro (2004):

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

$$(6.24)$$

where all variables have been previously defined.

Predicted scour depths from the modified equations compared to the U-weir prototype site scour depths showed closer approximations than relationships calibrated to laboratory data. Results of the analyses are presented in Table 6.3. Additional comparisons with measured field data are necessary to determine the widespread applicability of the modified D'Agostino and Ferro equation in predicted scour depths; however preliminary results show promise.

Table 6.3 – Results of applying equations to field sites at two-year discharge.

Prototype location	Observed (ft)	Scurlock (2009) (ft)	Modified D'Agostino and Ferro (2004) Eqn. 6.24 (ft)
Entiat RM 5.1	4.1	2.9	4.2
East Fork Salmon 7-8	4.7	9.1	6.9

6.4.2.6 *Scour-Depth as a Function of Mean-Weir Height*

Design of rock weirs to prevent scour from undermining the structure foundation requires a universal design methodology. It was shown that approaches proposed by Scurlock (2009) and subsequent analyses using the laboratory data are limited to ranges spanned by the test matrix. Reclamation (2009b) methods may prove to apply well in prototype scenarios, but currently, not enough data exist to determine if they apply beyond the two sites considered. A method incorporating a factor of safety and relying on prediction of scour depth with a simple relationship derived from a readily determined parameter offers a good design option provided the current level of available data. Such a methodology should apply to all laboratory and prototype cases available in order to provide designers with the most reliable tool to prevent structure failure, regardless of structure type and design choices such as interstitial flow.

The maximum scour for each laboratory configuration tested was identified and tabulated with the field data scour depths and mean weir height of each geometry. The ratio of the maximum scour depth to the mean weir height was determined for each case and data are provided in Table 6.4. On average, the scour depth was found to be 3.4 times the mean weir height of the structure. The predicted scour depth (y_{SE}) at the 95-percent confidence interval was 4.2 times the mean weir height (z_i), which mathematically can be expressed as:

$$y_{SE} = 4.2z_i \tag{6.25}$$

Table 6.4 – Scour depth as a function of mean weir drop height.

Test	Shape	y_{SE} M	z_i m	y_{SE}/z_i
29	A	0.41	0.09	4.70
38	A	0.36	0.09	3.83
44	A	0.40	0.09	4.17
22	U	0.13	0.10	1.23
25	U	0.54	0.09	5.81
35	U	0.24	0.10	2.41
31	W	0.45	0.09	4.96
41	W	0.32	0.10	3.16
50	W	0.35	0.11	3.19
Entiat RM 5.1	U	1.25	0.70	1.78
East Fork Salmon 7-8	U	1.43	0.71	2.01

6.4.2.7 *Scour Depth Prediction Using Explicit Neural Network Function (ENNF)*

A more recently developed method that offers an alternative to nonlinear regression approaches for scour prediction below grade control structures is based on explicit neural network functions (ENNF). Guven and Gunal (2008) used multiple experiment studies of over 250 training sets to obtain normalized maximum scour depth (s/z; scour depth/structure drop height) as a function of dimensionless parameters used in D'Agostino and Ferro (2004). Their statistical analysis (not based on physical justification) developed a function that proved to be a better predictor of scour depth downstream of grade control structures than those offered using conventional nonlinear regression analyses. While this process offers an improvement in prediction capabilities, it is not yet practical for design. However, the procedure may be an option in the future if the intricacy of application is reduced.

7 Design Guidance

From the methods and results presented in the previous sections, it can be seen that placing a river spanning rock weir in the channel can greatly affect the local hydraulics, scour development, and sediment transport. The degree to which these effects occur depends on a number of important variables that influence the way in which a structure functions in the stream. When designing rock weirs, designers are advised to use due diligence in using existing design guidelines and apply formulas and methods described in this document to estimate the resulting effects a specific rock weir design has on local hydraulics, scour development, and overall structure performance.

Hydraulic parameters used in designing rock weirs include flow depth, velocity, and bed shear stress. These parameters should be determined for a range of flows for both existing and post-project conditions. Common design discharges applied to the design of river spanning rock weirs are related to both high flow (i.e., bankfull flow) and low flow conditions (i.e., base flow). These parameters are used to size the weir rock as well as rock used for scour protection, to demonstrate that project goals (such as irrigation diversion and fish passage) are being met, and to determine overall structure performance. It should be noted that design of river spanning rock weirs in a natural environment, using natural materials, involves a significant degree of uncertainty. This research project focused on the relative change in local hydraulics (flow depth, velocity, and bed shear stress) associated with variations in structure geometry applicable to rock weirs. The methods and equations presented provide an increased understanding of the physical processes associated with rock weirs.

Utilizing the methods and results of research provides designers an additional tool that can be used in conjunction with the general steps and considerations found in the literature (Reclamation, 2007, Reclamation, 2009b, Rosgen 2006, Thomas et al. 2000, and WDFW 2004) for the design of river spanning rock weirs. However, these tools should be employed with an understanding of the variability occurring in natural stream systems and sound professional judgment. The installation of river spanning rock weirs should never be conducted without adequate site, reach, and watershed assessments to determine the nature and extent of problems in the watershed and to establish realistic project goals, objectives, and priorities.

7.1 Numerical Modeling

The most suitable methodology to model the hydraulics through varied configurations of rock weirs greatly depends on project goals, funding, and data resources. Currently, modelers use a range of modeling methods (e.g., Stage-discharge equations, 1-D modeling, and 2-D modeling) in an attempt to best represent the channel geometry and obtain accurate hydraulics used for structure

design. The methods vary in how much channel survey information is available, how the cross sections are configured (e.g., dog-legged to match flow patterns, straight across channel), and the subsequent spacing of cross sections (surveyed and interpolated). The method selected is typically at the modeler's discretion rather than based on a known comparison of which method is capable of providing the most accurate results. The following provides a summary of the different modeling techniques, advantages and disadvantages, and recommended applications. A detailed comparison between the different methods was described in Section 5.6.

7.1.1 One-Dimensional Models

As mentioned in Section 5, the incorporation of rock weirs into the one-dimensional model HEC-RAS was investigated to determine the potential value and quantify the limitations of one-dimensional modeling for design. The findings suggest that one-dimensional models are highly limited in their ability to replicate the hydraulics through rock weirs because rock weirs create three-dimensional, rapidly varied flow conditions. One-dimensional models are capable of resolving hydraulic jumps only if they occur uniformly across the channel. However, the hydraulic jumps that occur at rock weirs are not uniform. In addition, the cross-sectional averaging of one-dimensional models fails to capture the increased velocities through the throat of a rock weir, plunging flows into the scour pool, and the slow moving water along the structure arms. Although readily available, one-dimensional models are not appropriate to capture hydraulic conditions through weirs. One useful application of one-dimensional models in rock weir design may be in providing tail water inputs to stage-discharge relationships. A rating curve developed from stage-discharge relationship can be input into a one-dimensional model using an internal rating curve at a given cross section. Detailed stage-discharge equations developed from the research are presented in Section 7.2 below.

7.1.2 Two-Dimensional Models

Water surface elevations derived from two-dimensional models incorporate longitudinal and lateral variations in the water surface. Two-dimensional modeling of rock weirs may be used for predicting water surface elevations through the rock weir, but will not accurately represent water surface elevations through any hydraulic jump associated with the structure. Calculated water surface elevations derived from two-dimensional models have been shown to match reasonably well with measured values (within 10%), especially upstream of the structure crest (Holmquist-Johnson, 2011). Numerical modeling with two-dimensional models can serve many purposes in the design of rock weirs, particularly those with intricate features within the reach that need to be evaluated as part of the design. The need for two-dimensional modeling should be evaluated on a case-by-case basis, and must be coupled with other design tools. Two-dimensional modeling requires detailed bed topography, which can be time-consuming and expensive to collect, and expertise in understanding the limitations for application of the model results.

7.1.3 Three-Dimensional Models

Three-dimensional flow models utilizing solution algorithms to the Reynolds averaged Navier-Stokes equations (i.e., U2RANS) have been shown to represent hydraulics (water surface elevations and velocities) associated with river spanning rock structures with a high degree of accuracy (Holmquist-Johnson, 2011). While two-dimensional modeling provides velocity data that can be used to determine the approach velocities to the structure, simulated velocities just downstream of the structure do not adequately capture the vertical component of velocity. As a result, if the velocity in and around the structures is of the most importance, then three-dimensional modeling is required in order to calculate the vertical velocity component that might be required in determining things like predicting scour, scour protection, foundation depth, and fish passage requirements.

Three-dimensional numerical models are generally computationally intensive and not widely used by practicing engineers. Furthermore, high resolution data collection of the bed topography must be obtained before models can adequately operate. Therefore, three-dimensional models of rock weirs are not recommended as a design tool for common application but have been shown to be useful in determining the detailed local hydraulics (horizontal and vertical flow components) associated with a specific structure design.

7.2 Stage-discharge Relationships

The stage upstream of structures is an important variable used to determine water surface elevations required by irrigation diversions and when multiple weirs are used in series. A stage-discharge relationship allows a designer to estimate stage upstream of a structure for diversion purposes in addition to spacing between structures to ensure that tail water conditions for an upstream structure are met. The following sections describe the stage-discharge analysis and results from the laboratory studies conducted at Colorado State University and the numerical modeling conducted by Holmquist-Johnson (2011).

7.2.1 Stage-Discharge Relationships for Unsubmerged U-, A-, and W-weirs Developed in Laboratory

Stage-discharge relationships define a relationship between water-surface elevation or depth and the corresponding discharge in the channel. Although relationships with common in-channel weir structures, such as broad-crested weirs, have been extensively studied and reported, stage-discharge relationships have not been fully developed for channels in which U-, A-, and W-weirs have been installed. Research conducted at Colorado State University by Meneghetti (2009) and Scurlock (2009) provided a series of U-, A- and W-rock weir configurations tested in a laboratory stream bed to ascertain the stage-discharge ratings for one-third bankfull, two-thirds bankfull, and bankfull discharges. The resulting stage-discharge relationship is presented in Equation 7.1 as a function of the effective weir length and discharge coefficient (Thornton et al., 2011). A

unique discharge coefficient was developed for each of the weir configurations (Equations 7.2-7.4) as a function of the median crest stone size (d50), effective weir height (zi), effective weir length (bi), and stream width (B). In addition, a composite relationship for the discharge coefficient (Equation 7.5) was developed by performing a regression analysis on the dataset that included all three structure configurations. Figure 6.3 depicts the channel and weir parameters.

$$h_{weir} = \left(\frac{Q}{\frac{2}{3} b_i \cdot C_d \sqrt{2g} \cdot} \right)^{2/3} \tag{7.1}$$

where Q = discharge;

g = gravitational acceleration;

C_d = discharge coefficient for U-, A-, W- weirs individually and composited;

$$C_{dU} = 0.652 \, [d_{50}/z_i]^{-0.708} [b_i/B]^{0.587} \tag{7.2}$$

$$C_{dA} = 22.109 \, [d_{50}/z_i]^{-1.789} [b_i/B]^{-7.952} \tag{7.3}$$

$$C_{dW} = 0.002 \, [d_{50}/z_i]^{1.868} [b_i/B]^{4.482} \tag{7.4}$$

$$C_{dcomposite} = 1.139 \, [D_{50}/z_i]^{-0.703} [b_i/B]^{-0.261} \tag{7.5}$$

g = acceleration due to gravity;

d_{50} = median crest stone size;

B = stream width;

z_i = effective weir height, which is a function of weir geometry;

b_i = effective weir length, which is a function of weir geometry;

i = U-, A-, or W-weir.

It is recognized that these stage-discharge relationships are founded on a limited data base from controlled testing conditions for U-, A- and W-rock weirs. However, the findings are encouraging in that it may be possible to develop a reliable stage-discharge rating relation applicable to multiple rock weir shapes in the field setting. The high powers of b_i/B in the A- and W-weir equations indicate uncertainty in the equation application. The composite discharge coefficient appears applicable to a wider range of structure conditions and has coefficients and exponents that seem reasonable. Because a three-dimensional model was developed for U-weirs (Holmquist-Johnson, 2011), additional evaluation of a stage-discharge relationship was completed in an effort to expand the utility of the equation to conditions present outside of the laboratory. An improved U-weir stage-discharge rating curve is presented in the subsequent section.

7.2.2 Improved Stage-Discharge Relationships for Unsubmerged U-weirs Developed using 3D Numerical Modeling

The following section describes modifications that were made to the equations developed by Thornton et al. (2011) to increase the range of applicability as well

as the development of an improved stage-discharge relationship utilizing results from the numerical modeling conducted by Holmquist-Johnson (2011).

Holmquist-Johnson (2011) found that current relationships had limited applicability to the range of structure geometries included in the original development and that additional analyses were needed to develop equations applicable to a wider range of conditions. Utilizing output from the numerical model, regression analyses were conducted to develop new stage-discharge relationships that were applicable to a wide range of structure parameters and flow conditions for a U-weir. Results of the stage-discharge analysis conducted by Holmquist-Johnson (2011) found that the relationship developed from Equations 7.6 and 7.7 (see Figure 7.1 for depiction of channel and U-weir parameters) provides a method to predict the weir flow depth for a given U-weir geometry and reach characteristics with an absolute mean error of 6.7 percent and standard deviation of 4.9 percent.

$$h_{weir} = \left(\frac{Q}{\frac{2}{3} W_u \cdot 4.386 \left(\frac{W_u}{Z_u} \right)^{-0.601} \left(\frac{L_t}{T_w} \right)^{-0.429} \cdot \sqrt{2g} \cdot} \right)^{2/3} \quad for\ Q < \frac{2}{3} Q_{bkf}$$

(7.6)

$$h_{weir} = \left(\frac{Q}{\frac{2}{3} W_u \cdot 9.766 \left(\frac{W_u}{Z_u} \right)^{-0.73} \left(\frac{L_t}{T_w} \right)^{-0.359} \cdot \sqrt{2g} \cdot} \right)^{2/3} \quad for\ Q > \frac{2}{3} Q_{bkf}$$

(7.7)

where Q = discharge;

Q_{bkf} = bankfull discharge;
g = gravitational acceleration;
W_u = effective weir width, function of structure geometry;

$$W_U = W_t + 2 \left(\frac{(Zu - Zd)\sin\theta}{Tan\phi} \right)$$

Z_u = effective weir height, function of structure geometry;

$$Z_u = \frac{1}{3} \left(\frac{(T_w - W_t)}{2} \cdot \frac{Tan\phi}{\sin\theta} \right) + Z_d$$

L_t = total weir length, function of structure geometry;

$$L_t = W_t + 2 \cdot L_a{''}$$

$$L_a{''} = \left(\frac{(T_w - W_t)}{2} \cdot \frac{1}{\sin\theta \cdot \cos\phi} \right)$$

T_w = channel top width;
W_t = weir throat width; and
Z_d = structure drop height.
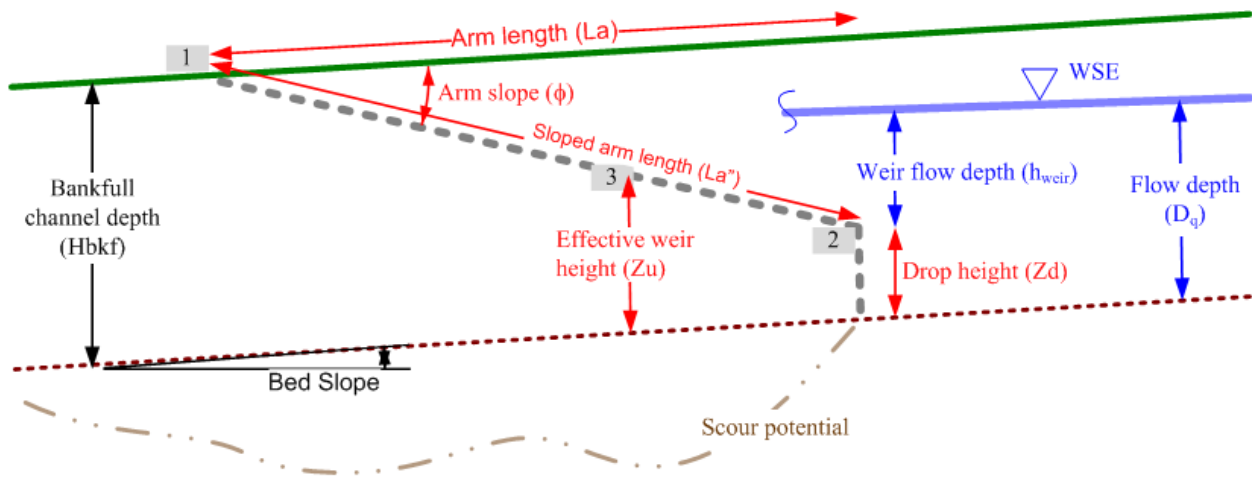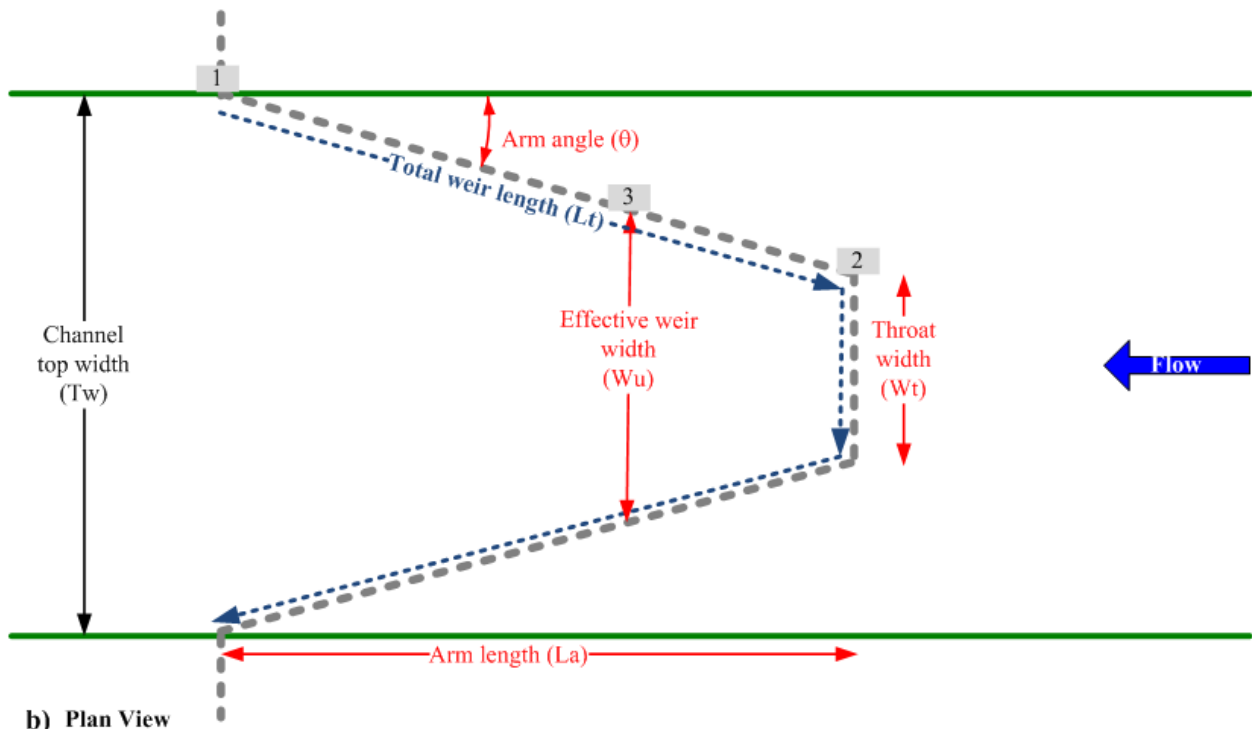
Figure 7.1 – Depiction of channel and U-weir parameters: a) profile view and b) plan view (Holmquist-Johnson, 2011).

A box and whisker plot showing the variation in percent error in predicted weir flow depth between the four stage-discharge equations described in Holmquist-Johnson (2011) are presented in Figure 7.2.
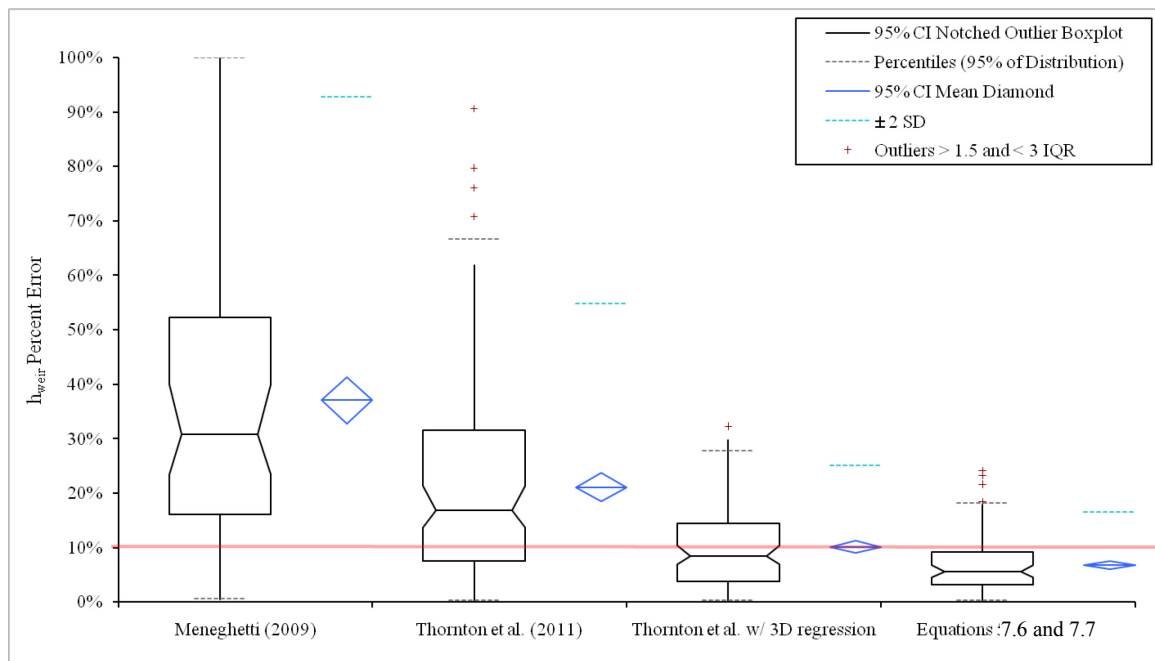
	n	Mean	95% Confidence Interval	Standard Error	Standard Deviation
Meneghetti (2009)	165	37.09%	32.80% to 41.37%	2.17%	27.88%
Thornton et al. (2011)	165	21.09%	18.51% to 23.68%	1.31%	16.83%
Thornton et al. w/ 3D regression	165	10.13%	8.98% to 11.28%	0.58%	7.47%
Equations 7.6 and 7.7	165	6.74%	5.98% to 7.49%	0.38%	4.92%

	n	Min	1st Quartile	Median	95% Confidence Interval	3rd Quartile	Max	IQR
Meneghetti (2009)	165	0.40%	16.03%	30.87%	23.39% to 39.93%	52.32%	113.39%	36.30%
Thornton et al. (2011)	165	0.10%	7.52%	16.87%	13.66% to 21.34%	31.56%	90.57%	24.05%
Thornton et al. w/ 3D regression	165	0.17%	3.83%	8.48%	6.91% to 10.44%	14.51%	32.30%	10.68%
Equations 7.6 and 7.7	165	0.10%	3.19%	5.58%	4.58% to 6.81%	9.20%	24.07%	6.01%

Figure 7.2 – Percent error magnitude box-plot comparison of stage-discharge relationships developed by Meneghetti (2009), Thornton et al. (2011), and Equations 7.6 and 7.7 (Holmquist-Johnson, 2011).

Utilizing data collected from the field and the laboratory, weir flow depth was predicted and compared with measured values. Comparing the measured flow depths with predicted values from Equations 7.6 and 7.7 provides a method to test the applicability of the equations to data that were not used in the development of the newly improved stage-discharge relationship. Although the field site is limited, the equations developed by Meneghetti (2009) and Thornton et al. (2011) were also used to predict the weir flow depth for the field site and compared with the results from Equations 7.6 and 7.7 (Holmquist-Johnson 2011).

Figure 7.3 shows the observed versus predicted weir flow depth for the field site and laboratory data set utilizing the three equations described above. The results

show that Equations 7.6 and 7.7 predicted the weir flow depth for the field site and laboratory data set well with an absolute mean error of 4.5 percent and standard deviation of 4.0 percent. Comparing the observed versus predicted values for the field and laboratory data independently from the numerical model data set demonstrates the applicability of Equations 7.6 and 7.7 to measured data in the field and in the laboratory setting. Figure 7.3 also shows that the equations differ in their ability to predict the weir flow depth for the field site. The percent errors associated with each equation for the field site are as follows: Meneghetti (2009) percent error = 48.3%, Thornton et al. (2011) percent error = 32.8%, and Holmquist-Johnson (2011) Equations 7.6 and 7.7 percent error = 10.6%.
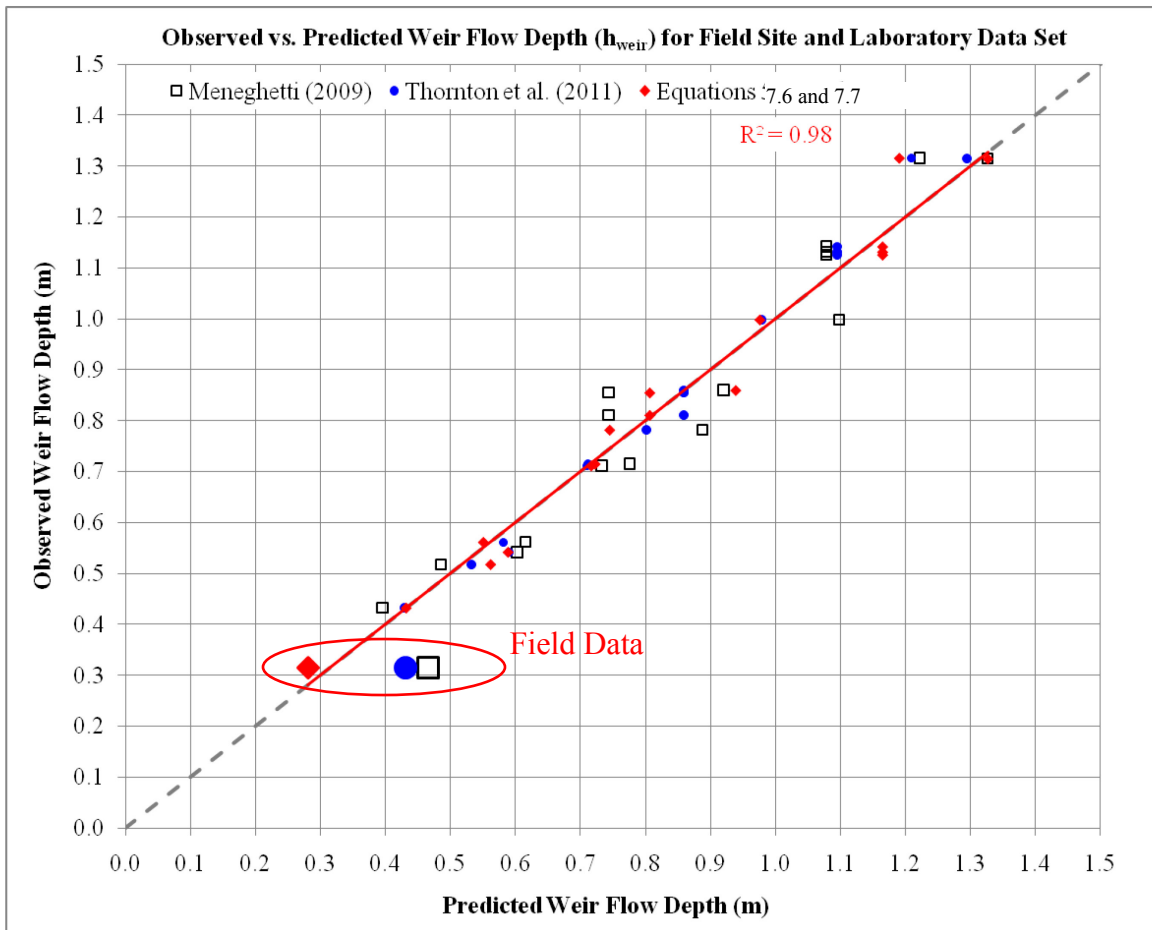
Figure 7.3 – Observed versus Predicted weir flow depth using stage-discharge relationships for field site and laboratory data set (Holmquist-Johnson 2011).

7.3 Linking Results of Field Investigations to Design Recommendations

Data from the field investigations were used in a quantitative analysis (Reclamation, 2009b) to identify relationships between specific rock weir design parameters and structure stability. 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 for certain design parameters (Table 7.1). Figure 7.4 illustrates each of the measured parameters for an example U-weir where a notable relationship to failure was identified. The values in the table 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 is 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.

The most commonly referenced guidance for the design of river spanning rock structures is Rosgen (1996, 2001, and 2006). 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 7.2. 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.

a) **Profile View**
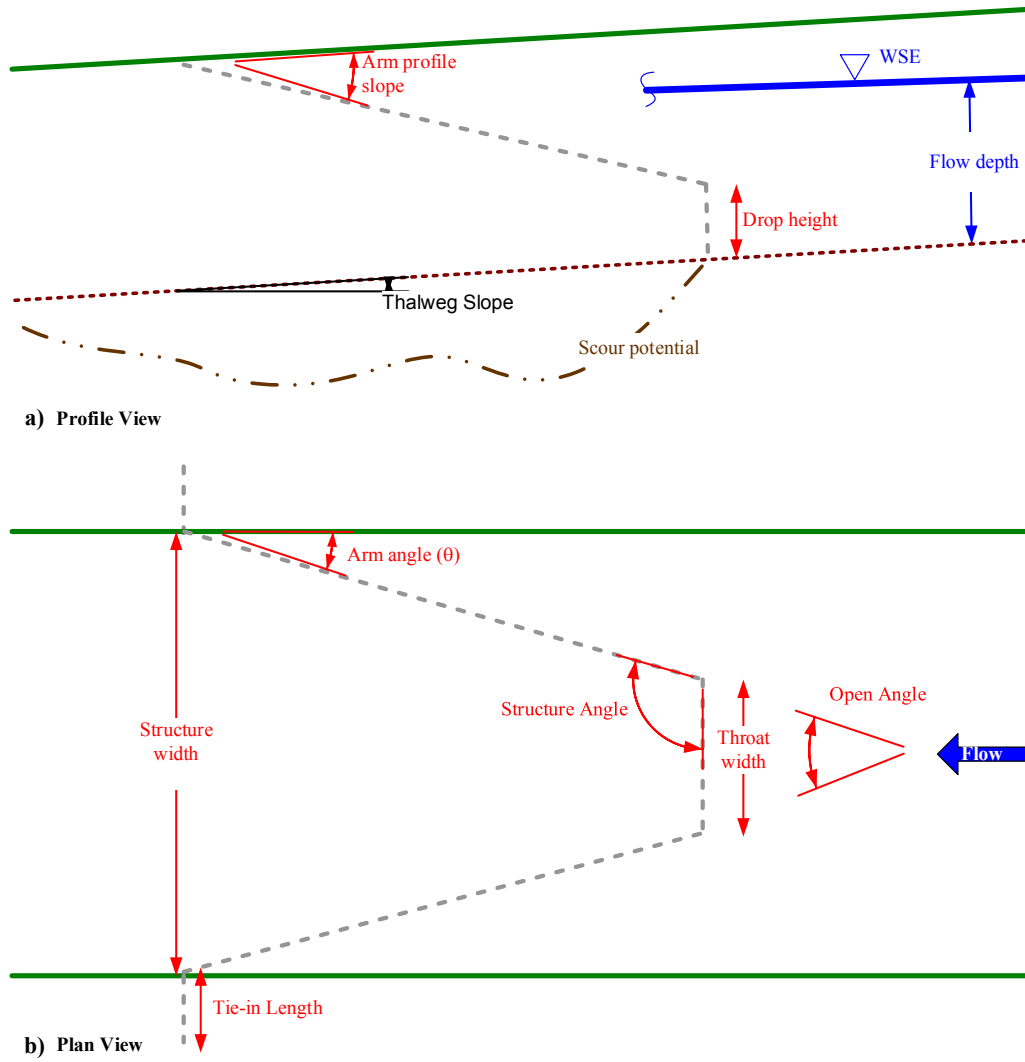
b) **Plan View**

Figure 7.4 – Profile (a) and plan-view (b) of select design parameters for U-weir with notable relationships to failure.

Table 7.1 – Values for select variables 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.

Variable	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-view arm angle	U-,V-weirs	Greater than 35 degrees
	Asymmetrical U-weirs	Greater than 33 degrees (partial), 40 degrees (full)
Maximum structure 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%

Table 7.2 – Comparison of recommended cross-vane design ranges (Rosgen 1996, 2001) and ranges for U-,V- weirs surveyed in the field.

Parameter	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

7.4 Scour Prediction and Foundation Design

7.4.1 Recommended Scour Prediction Method

Multiple methodologies were evaluated for the prediction of scour depths in the design of rock weirs, including: (1) A-, U-, and W-weir regression equations developed with laboratory data, (2) U-weir equations based on D'Agostino and Ferro (2004) and applied to limited field data, and (3) an all-geometry inclusive scour depth function of mean weir drop height based on laboratory and field data. Within this section of the report, recommended methods to predict scour are presented. Details on each method can be found in Section 6.4.

In the design of the foundation depth of a rock weir, the following procedure is recommended:

1. Develop a table of predicted scour depth values predicted from the different equations. It is important to use at least 3 different equations to evaluate your predicted scour depths. This following scour equations are recommended:

   a. Apply the relationships that only use a modified form of the densimetric Froude number presented in Section 6.4 (Equation 6.14, 6.15, 6.16, and 6.17). The composite equation (Equation 6.17) may provide a reasonable prediction for any structure type, but the individual equations for the A- (Eqn. 6.14) and W- weirs (Eqn. 6.16) have improved prediction capabilities in comparison to the composite equation. Because the number of terms in these equations is limited, the equations may have an improved transferability to conditions outside of the laboratory values when compared with Equations 6.9, 6.10, and 6.11.

   b. Apply Equation 6.7 (Comiti et. al, 2006).

   c. Apply the modified D'Agostino and Ferro 2004 Equation (Equation 6.24 Reclamation 2009b).

   d. Apply Equation 6.22 (Pagliara and Kurdistani, 2013).

   e. Apply the generalized design guideline from Equation 6.25, which states that the maximum scour depth is expected to be limited to less than 4.2 times the mean height of the structure, regardless of structure geometry class, flow rate, or bed material properties.

2. Compare the resulting predicted scour depths. Select the predicted scour depth that is most appropriate with application of engineering judgment. If all scour predictions are within a reasonable range of one another, consider using the average of the predicted values. If necessary, remove any outlier predictions.

3. Design the foundation depth to meet or exceed the predicted scour depth, applying a factor of safety if necessary based upon the allowable risk of failure.

   a. If the required depth of foundation is not feasible based upon predicted scour depths due to the presence of bedrock, use the depth of bedrock as the foundation depth.

   b. If the required depth of foundation is not feasible due to construction limitations, consider alternative options for structure design, including the use of grout, eco blocks or possibly building a rock ramp in lieu of a rock weir. Alternatively, anchor the structure base to the deepest point possible and anticipate some maintenance over time.

7.4.2 Footer Design

The previous section describes the maximum depth of the foundation of the structure, which signifies the depth at which the footers should be placed to protect against geotechnical failure and slumping of the footer rocks. In practice, the footers at the throat are commonly buried to the maximum depth of the foundation, and as the arms of the structure slope upwards toward the banks, the footers similarly slope upward toward the banks. This leaves the location of the greatest drop height over the structure (typically along the arms close to the bank) protected with the least amount of foundation depth.

During field observations conducted as part of this research effort, the most common failure mechanism was determined to be development of a scour hole on the downstream side of the structure and subsequent geotechnical slumping of the footer rock. Frequently, the scour and footer slumping occurred along the structure arms. A quantitative investigation of structure parameters found that the profile arm angle was positively correlated with potential for structure failure (Reclamation, 2009b). 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.

One recommended approach to remedy the foundation issue along the arms is to place all the footer rocks to the required foundation elevation determined from the scour depth prediction equations, including a factor of safety. Construct the weir by placing the rocks along the structure arms to meet the design slope either using multiple rows of design-sized rock components or increasing the size of the rocks along the structure arms. For added stability, the length of the structure along the channel (parallel to flow) may need to be increased by one diameter rock, at least along the foundation. This design may increase material costs, but should result in a more stable structure that is less prone to failure and reduce maintenance costs over the long-term. Images displaying the typical and proposed designs are illustrated in Figure 7.5 and Figure 7.6, respectively.
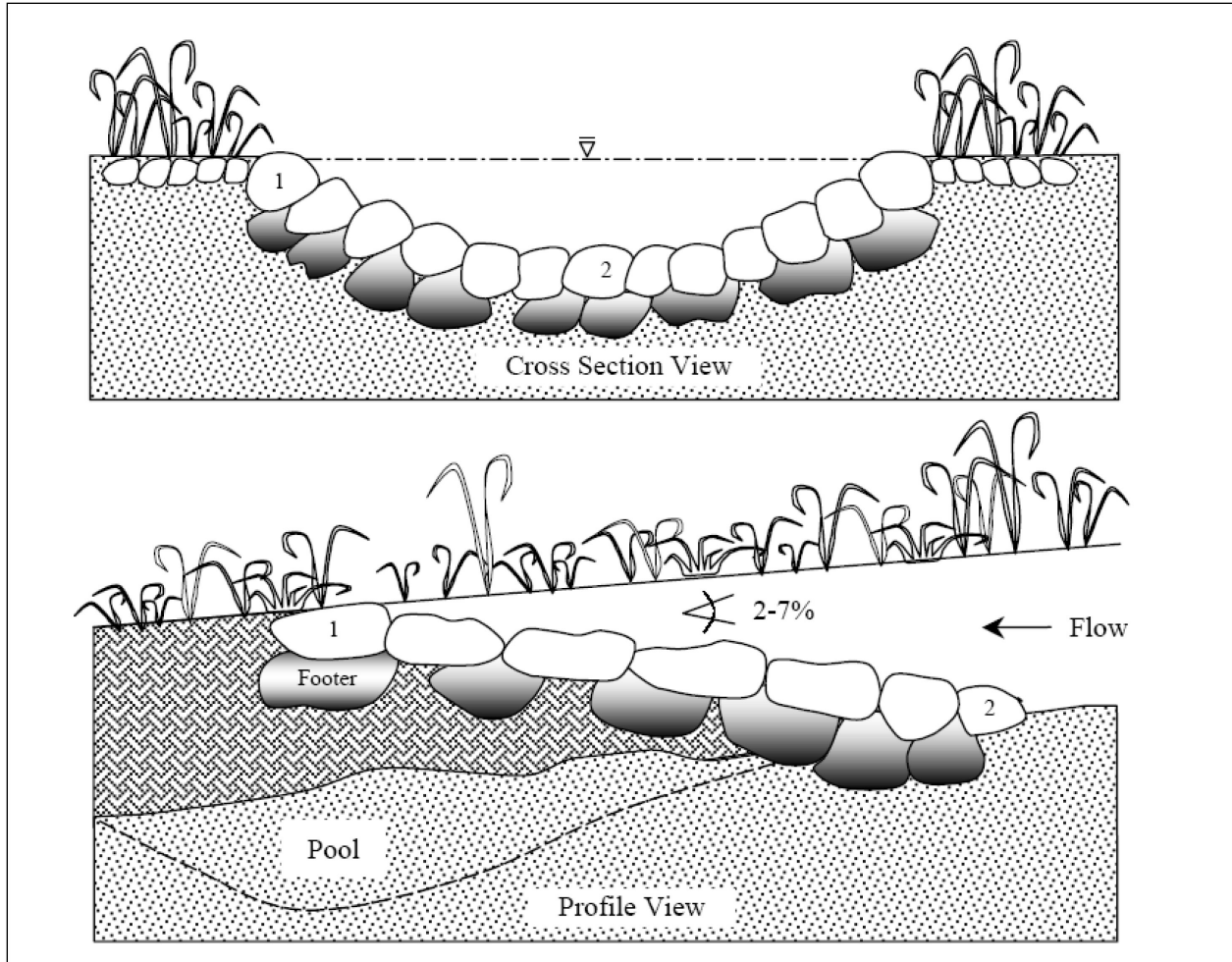
Figure 7.5 – Typical rock weir cross section and side view profile (Rosgen, 2001).
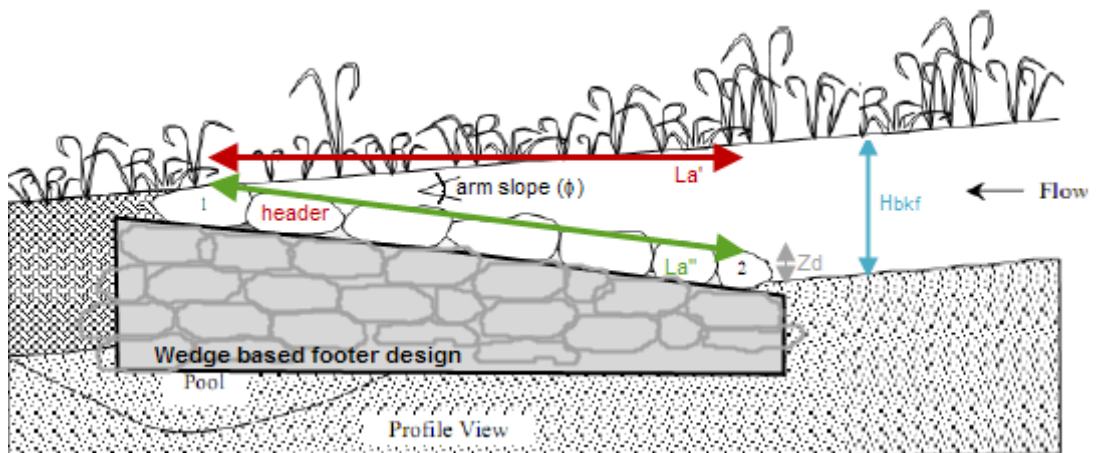
Figure 7.6 – Illustration of proposed wedge-based footer design.

7.4.3 High Flow Analysis

Designing structures to withstand potential scour resulting from high flows requires adequate scour protection. Input to several of the scour prediction equations requires information regarding headwater depth, tailwater depth, and piezometric head loss through the structure. These parameters can be extracted from a model for the design flow. However, the discharge responsible for the greatest scour is not necessarily associated with one particular frequency across all structures. Geomorphology of the valley and reach of interest influence what discharges are associated with the greatest scour potential. For example, a structure in a wide, flat valley may experience greater scour during discharges of smaller recurrence intervals, such as a 2-year discharge, than during less frequent discharges, such as a 10-year discharge. Flows of extreme magnitudes may begin to back up at a particular geologic constriction, resulting in reduced velocities and shear stresses within the channel and hydraulically submerging the structure. In contrast, a structure in a narrow valley with a steep slope may experience increases in bed shear stresses and velocities with increasing recurrence intervals up to a certain point at which they would begin to level off within the channel.

Determining the flow responsible for the greatest scour requires both an understanding of the geomorphic properties of the reach of interest and rating curves providing hydraulic properties associated with each discharge. The appropriate design discharge for a specific construction configuration is resolved through an iterative process, in which a scour prediction equation is computed across a full range of discharges. The greatest scour computed is therefore associated with the discharge to which scour protection should be designed (scour design discharge). An outline of steps for how the discharge responsible for the greatest scour could be determined using a one-dimensional HEC-RAS model is illustrated below.

1. Initially, set up a HEC-RAS model based on existing conditions with sufficient longitudinal length to capture the water surface slope through the reach of interest and with an increased frequency of cross sections within 2 channel widths upstream and downstream of the proposed location of the rock weir.

2. Run the model across a full range of discharges that the structure may experience with a maximum of bankfull discharge. Set up a spreadsheet with a rating curve of hydraulic properties for each discharge. For each flow scenario:

   a. Determine tail water depth assuming normal depth conditions downstream of the hydraulic influence of the structure. Tail water can be determined from the average depth of the cross sections just downstream from the proposed location of the scour pool. Be sure to exclude depths from cross sections within existing pools.

   b. Determine head water depth using equations developed from laboratory experiments and 3D modeling from Section 7.2. For A-

weirs and W-weirs, apply Equation 7.1. The composite discharge coefficient is recommended for application. In the case of a U-weir, apply the appropriate form of Equation 7.6 or 7.7 depending on the flow scenario. Note that these equations will not apply in submerged conditions.

c. Compute the anticipated water surface elevations upstream and downstream of the proposed structure by adding the local bed elevation to the computed headwater and tailwater depths, respectively. These computed water surface elevations can be later used as an internal rating curve in HEC-RAS in step 4.

d. Compute the pieziometric head drop across the proposed structure as the difference between the computed upstream and downstream water surface elevations from part c.

e. Apply a range of scour prediction equations, such as those outlined in Section 7.4.1 to obtain a different scour prediction depth with each discharge evaluated.

3. The discharge with the greatest predicted scour is the scour design discharge.

4. If the structure is incorporated into the HEC-RAS model, use an internal rating curve based on the computed discharge and water surface elevations in step 2c.

7.5 Spatial Extent of Scour Pool

Field reconnaissance observations by Reclamation (2007) and results from laboratory testing (Scurlock, 2009) found channel profiles associated with rock weirs to be characterized by a scour pool located downstream from the structure crest and filling on the upstream side. The longitudinal location of the scour pool varied, but the maximum depth tended to occur at the end of the shortest arm. Lengths of the scour pools also varied but appeared to stretch approximately twice the length of the shortest arm. The lateral width of each pool tended to span the entire area within the structure arms. Many pools showed deposition at the downstream extent of the scour hole, resulting in a longitudinal profile as depicted in Figure 7.7.

The ability to estimate the location and dimensions of the scour pool resulting from a rock weir installation allow the designer to maximize the efficacy of the foundation design and the resultant pool habitat value while improving understanding of patterns of localized erosion and deposition and potential impacts to fish passage and sediment transport through the structure. The following subsections describe numerical modeling efforts and field measurements intended to help inform upon scour pool development and final

dimensions, along with brief guidance on incorporating a pre-excavated scour pool into the design.
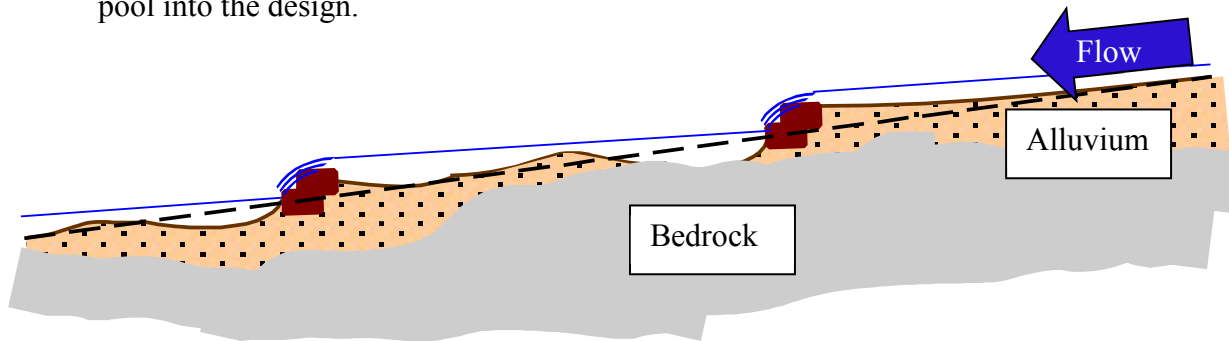
Figure 7.7 – Typical longitudinal profile of sediment deposition and pool patterns (Reclamation, 2007).

7.5.1 Velocity and Shear Stress Magnification of a U-weir

As water flows over a river spanning rock U-weir, it is redirected perpendicular to the structure crest and results in a concentration of flow in the center of the channel and away from the stream banks. As a result, downstream of the structure, near bank velocities and shear stresses are reduced while those in the middle of the channel are increased (Figure 7.8).
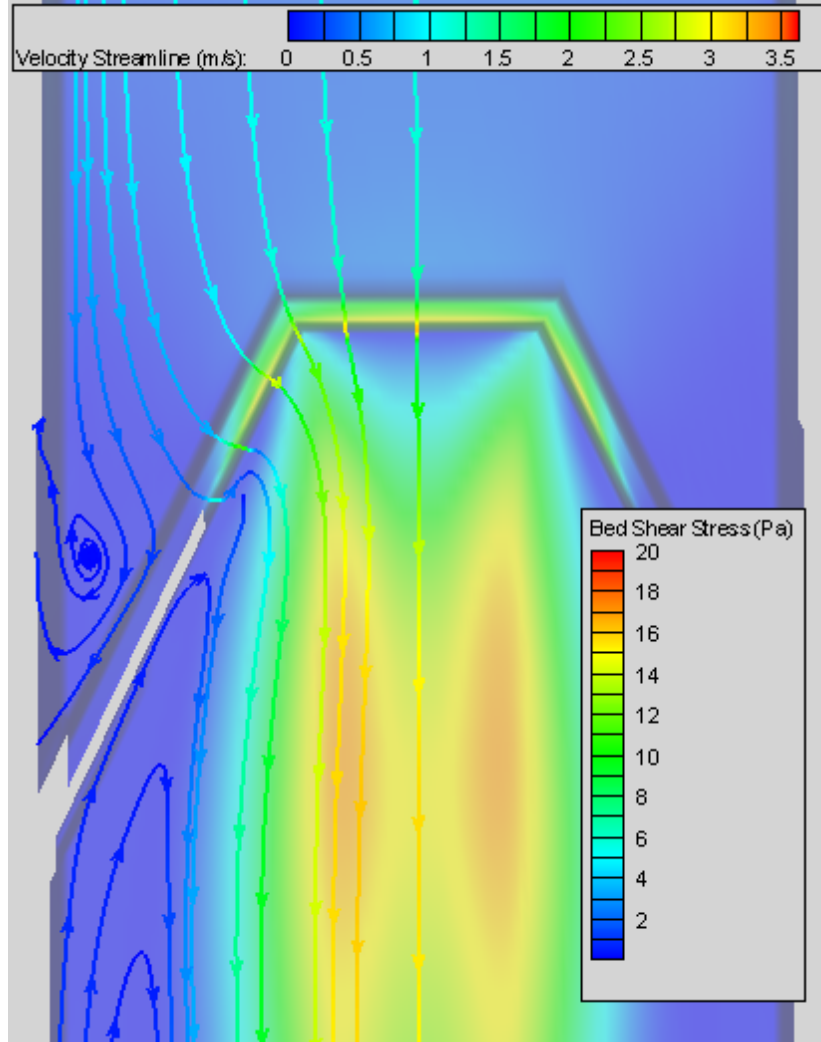
Figure 7.8 – Example of flow redirection over weir crest and increase in mid-channel velocity streamlines and bed shear stress associated with U-weir (Holmquist-Johnson, 2011).

Numerical modeling results were used to determine the maximum velocity and bed shear stress magnification for a given structure geometry and given reach characteristics. Results of the velocity magnification analysis showed that, for the range of conditions tested, the maximum velocity downstream from U-weirs was increased by 1.2 to 4.0 times the original channel velocity with no structure present (Holmquist-Johnson, 2011). Maximum bed shear stress magnification is a function of critical shear stress (i.e. incipient motion) for a given bed material size and provides insight into the potential scour that may occur due to variations in structure geometry. Results of the bed shear stress magnification analysis showed that the maximum bed shear stress downstream from U-weirs was 1.6 to 7.6 times the critical bed shear stress and varied in location from 0.1 to 1.25 times the arm length downstream of the structure crest (Holmquist-Johnson, 2011). Holmquist-Johnson (2011) also found that the largest bed shear stress magnification occurred at flows between two-thirds bankfull and bankfull flow. From Figure 7.9 and

Figure 7.10, it is evident that reducing the length of the structure arm increases the bed shear stress significantly due to the increased flow constriction and redirection of flow through the center of the channel with a greater velocity. The maximum bed shear stress magnification for the shorter weir arm length ($\eta_{\tau max}$=4.29) was approximately double that of the longer weir arm ($\eta_{\tau max}$=2.11).
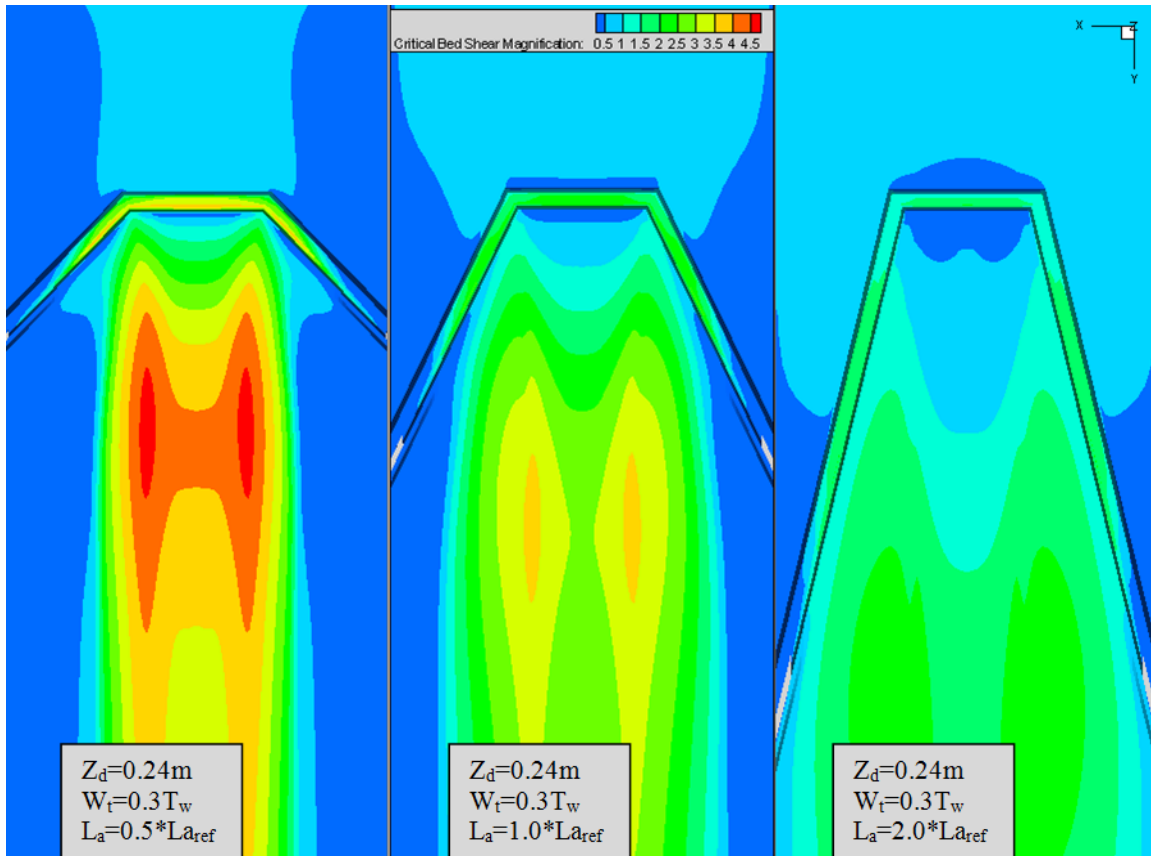
Figure 7.9 – Bed shear stress magnification associated with variation in structure arm length at Qbkf (Holmquist-Johnson, 2011).
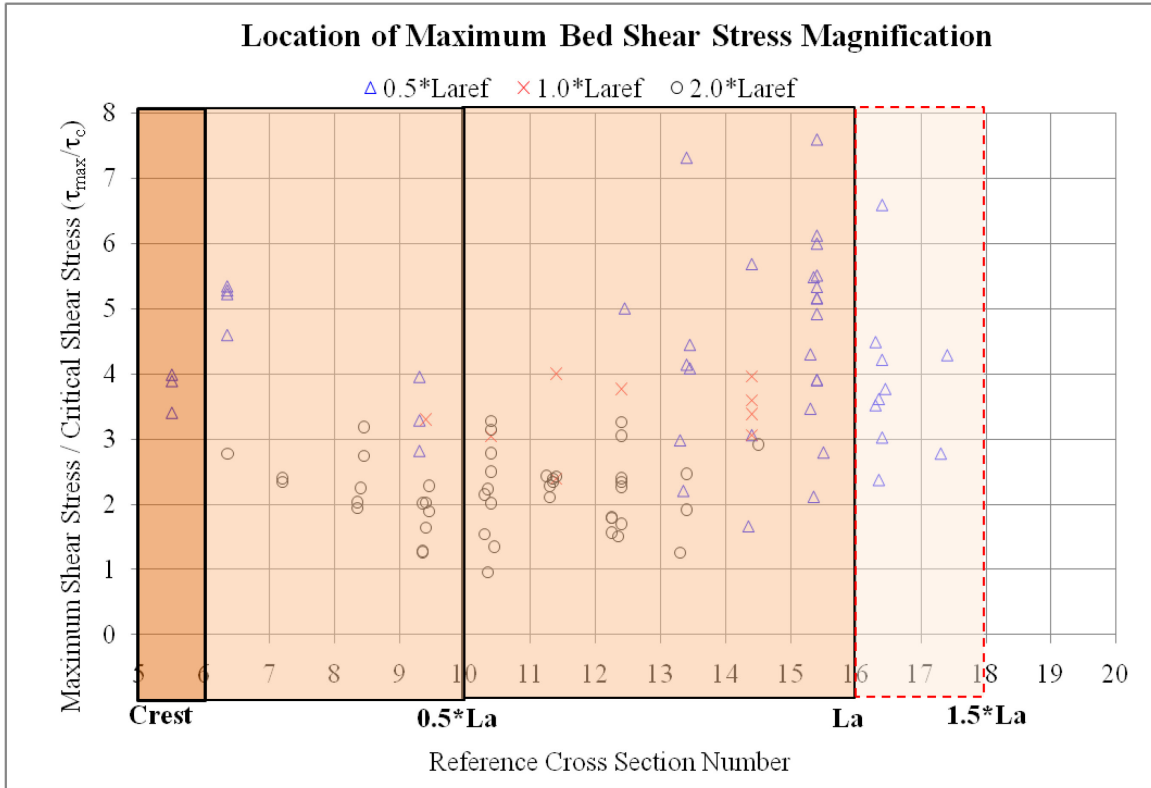
Figure 7.10 – Maximum bed shear stress magnification location associated with variation in structure geometry for flows greater than 1/3Qbkf (Holmquist-Johnson, 2011).

Identifying the location of the maximum velocity and bed shear stress magnification provides insight into how varying structure geometry can have a large influence on where initial scour might occur within the structure. Being able to compare the location and differences in channel bed shear stress associated with various weir geometries is important in the design process to ensure that the structure is not undermined by scour and is able to maintain sediment transport through the reach and the structure itself.

Utilizing the numerical model results, designers can determine where a structure design is within the range of investigated conditions and estimate the maximum velocity magnification and bed shear stress that is associated with that configuration. The results also provide the designer a method to compare variations in structure geometry and whether the resulting hydraulic conditions fall within prescribed guidelines and meet project objectives. If a more detailed analysis is required or the structure configuration is not within the range of the currently available data, additional numerical modeling could be accomplished to estimate how variations in specific channel characteristics and/or structure configurations would alter local flow depths and velocity and bed shear stress distributions.

7.5.2 Field Measurements of Scour Pool Dimensions

Measured scour pool dimensions at existing structures offer guidance in determining the length and width of anticipated scour resulting from the installation of a rock weir. Many designs are now incorporating armoring of pre-excavated scour pools to prevent scour from undermining the structure integrity. However, armoring of the scour pool is not yet a recommended practice as the extent to which it reduces full scour potential is not understood.

Field data collected were sufficient to capture the longitudinal and lateral extent of the scour pool at 18 structures between 2005 and 2010. Structures were considered adequate for investigation of scour dimensions if the scour pool was defined by both a lateral and longitudinal extent, appeared to be consistent with the definition of measured scour at other structures, and had not completed filled with sediment following structure failure. The 18 structures used in the analysis were located along Bear Creek, OR, East Fork Salmon River, ID, Entiat River, WA, Catherine Creek, OR, Lemhi River, ID and Rio Blanco, CO. Scour widths and lengths were estimated based on surveyed points downstream from each structure identified as scour or pool during the field investigations. Measurements of length extended from the throat of each structure to the downstream most point identified as scour or pool. Measurements of width were made parallel to the structure throat and extended from the widest section across the channel identified as scour or pool. Figure 7.12 illustrates the structure variable definitions, while Table 7.3 shows the resultant measured values.

Arm length (La)

Arm slope (φ)

Bankfull
channel depth
(Hbkf)

Drop height (Zd)

2*La

Bed Slope

Scour potential

a) Profile View

Arm angle (θ)

Channel
top width
(Tw)

Throat
width
(Wt)

Arm length (La)

b) Plan View
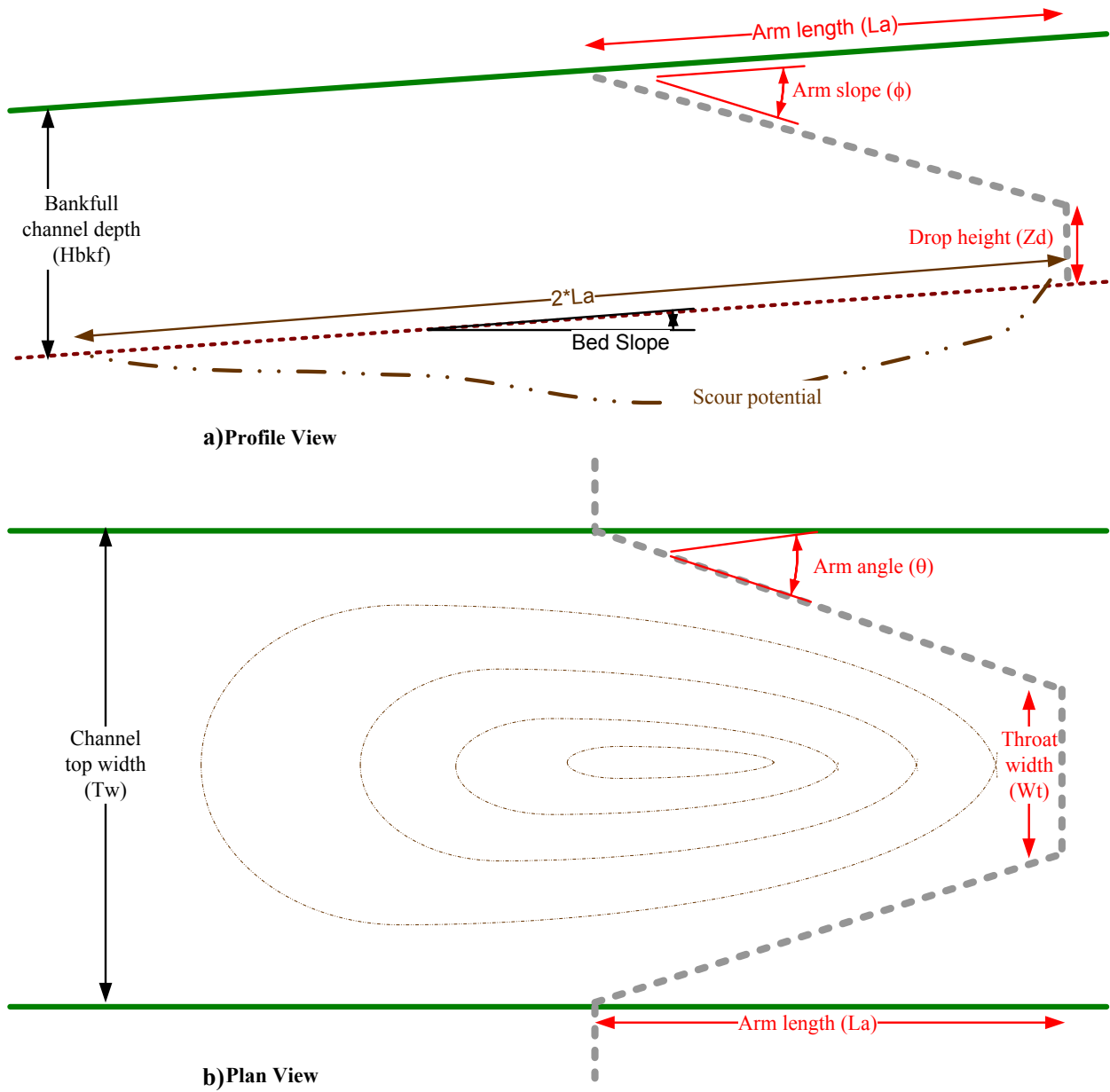
Figure 7.11 – Example of pre-excavated scour pool definition.

Table 7.3 – Measured longitudinal and lateral extent of scour from investigated field sites.

Site Name	Structure Name	Structure Type	Length of Scour (ft)	Width of Scour (ft)	Length of Scour Relative to Channel Width	Width of Scour Relative to Channel Width
Bear Creek	Site B	U-weir	45.7	29.1	0.8	0.5
Bear Creek	Site D1	U-weir	14.5	14.5	0.2	0.2
Bear Creek	Site D2	U-weir	28.4	10.2	0.6	0.2
Bear Creek	Site D3	U-weir	14.0	10.4	0.3	0.2
East Fork Salmon	7-8	V-weir Rock Ramp	96.4	35.1	1.3	0.5
Entiat River	RM 3.1	U-weir	129.8	82.7	1.6	1.0
Entiat River	RM 3.2	U-weir	155.3	57.6	2.1	0.8
Entiat River	RM 3.4	U-weir	58.0	38.2	0.6	0.4
Entiat River	RM 4.6	A-weir	75.6	55.8	1.1	0.8
Entiat River	RM 5.1	A-weir	84.6	30.5	0.6	0.2
Hempe-Hutchenson	2	U-weir	25.2	13.5	0.6	0.3
Lemhi	L3AO	A-weir	80.1	63.4	0.9	0.7
Rio Blanco	A	A-weir	32.5	18.0	0.9	0.5
Rio Blanco	C	U-weir	13.5	12.7	0.5	0.5
Rio Blanco	E	A-weir	45.0	17.9	1.4	0.5
Rio Blanco	F	A-weir	38.0	18.4	0.8	0.4
Rio Blanco	K	A-weir	50.2	18.3	1.8	0.6
Swack-hammer	W	W-weir	30.9	8.4	0.4	0.1

Results of the numerical modeling and field measurements indicate that the longitudinal length of scour may vary from 0.2 to 2 channel widths downstream from the structure throat and that the lateral width of the scour pool may vary from 0.1 to 1 channel width. These measurements suggest that the scour pool resulting from installation of a rock weir may extend across the entire lateral extent of the structure and approximately 2 times the structure arm length downstream from the structure throat. While armoring of the scour pool as a means of maximizing the protection against structure failure is a common practice, its efficacy in reducing structure failure has been minimally studied and is not currently recommended.

7.5.3 Pre-Excavated Scour Pools

While the general design of rock weirs does not specify the use of a pre-excavated scour pool, it may be useful in creating flow diversity and increased habitat downstream of the structure immediately following installation. This can be especially important if larger flows that are typically associated with scour events are not encountered in the first year after installation.

Rivers are continually adjusting to the amount of water and sediment being moved through the system and therefore the scour pools associated with rock weirs are dynamic in nature. As a result, design guidelines associated with pre-excavated scour pools are vague. Results from this research provide the following guidelines related to pre-excavated scour pools with the caveat that changes in pool structure will occur due to variations in hydrology, incoming sediment load, and overall transport capacity:

- Use the scour prediction equations as discussed in Section 7.4.1 to determine the maximum scour depth associated with a specified structure geometry and reach characteristics;

- Scour pool geometry should extend longitudinally approximately 2 times the length of the structure arm (La) with the maximum scour depth located near the shortest arm;

- Scour pool geometry should extend laterally to include approximately 80 percent of the channel width within the extent of the structure; and

- At a minimum, if the maximum scour depth is not able to be determined using the scour prediction methods, excavate scour pools during construction that closely resemble natural pool depths in the reach.

- Regardless of whether pre-excavation of the scour pool is completed or not, the final equilibrium scour may be quite similar.

7.6 Sizing of Rock

In a flowing river, rock weirs are subjected to partially and fully submerged conditions. Research relating to sizing of rock used in loose rock structures is prevalent in existing literature. However, rock sizing related directly to rock weirs is limited in literature, with only Rosgen (2001/2006) presenting equations based on case studies. Rosgen developed an empirical relationship between bankfull shear stress (units incorrect) and minimum rock weir particle diameter for rivers where bankfull discharge is between 0.56 cms (~20 cfs) and 113.3 cms (~4,000 cfs) and bankfull mean depths are between 0.26 m (~1 ft) and 1.5 m (~5 ft) (Figure 7.12). From this figure, it is notable that only 2 of the structures in this data set consist of rocks that are less than 2 feet diameter with most in the 2.5–4 foot range.
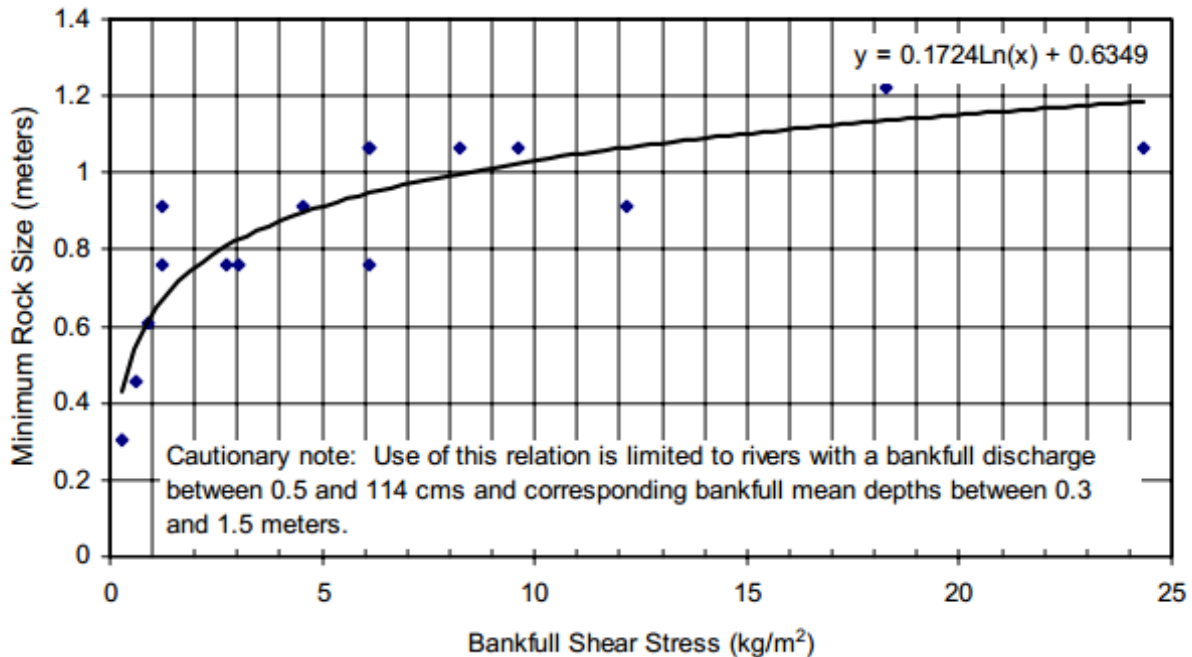
Figure 7.12 – Rock sizing empirical relationship developed by Rosgen (2001). Note that the units of bankfull shear stress along X axis are not correct.

Many Natural Resource Conservation Districts (i.e. NRCS, 2000; NRCS, 2013) utilize technical design notes suggesting that large rock generally greater than 3 feet in size be utilized in rock weirs for efficiency in placement and cost. When large rock is not available, they recommend following standard riprap sizing criteria for turbulent flow using the FWS-Lane method for the design flow with modifications (Lane, 1955). The FWS-Lane method produces a D_{75}, from which a gradation is developed to determine the D_{50} and D_{100} for the riprap, and then transformed for use in the rock weir as follows, presumably where $D_{50\text{-barb}}$ and $D_{100\text{-barb}}$ represent the median and maximum size rock in the structure, respectively.

$$D_{50\text{-barb}} = 2 \times D_{50\text{-riprap}} \tag{7.8}$$

$$D_{100\text{-barb}} = 2 \times D_{50\text{-barb}} \tag{7.9}$$

$$D_{minimum} = 0.75 \times D_{50\text{-riprap}} \tag{7.10}$$

The California Transportation Department produced guidance for sizing of rock weirs (Caltrans, 2007), wherein they suggest using three methods for sizing material used in a rock weir, including a field inspection method, a rock slope protection (RSP) revetment method, and a boulder cluster design method. The field inspection method includes examining upstream and downstream river reaches and identifying the stable rock size within the channel. The RSP revetment method computes the minimum weight of the rock that should be utilized in the design through the following formulation:

$$W = \frac{0.00002 V^6 SG}{0.207(SG-1)^3} \tag{7.11}$$

where W = minimum mass of rock (pounds);
V = 1.33V_{max} (ft/s);
SG = specific gravity of rock; and
V_{max} = the maximum stream velocity for the design discharge.

The Caltrans guidance document recommends that the design discharge for this equation correspond to a minimum of a 50-year flow. However, this method may not incorporate the fact that maximum velocities within the channel and acting on an individual rock within the channel may actually occur at lower flows. The computed weight of the rock will correspond to a class size outlined in the CA RSP report (Racin et. al, 2000), and this is the minimum size of rock that should be included in the rock weir. The third method recommended by Caltrans is the boulder cluster design method, which is a simplistic approach using a table with critical shear and critical velocities for given rock sizes. Using average velocities and shear stresses computed for a specific design discharge, this table indicates what size material will not be subject to incipient motion.

7.6.1 Rock Sizing Recommendations

Recent field studies (Reclamation, 2009b) suggest that rock weir failure due to incipient motion of the header and footer rocks was limited. Of greater importance is the design of the weir structure and protection against slumping of the footer rocks into a scour hole that is deeper than the foundation or footer rocks. In most cases, the structure height at the throat of the structure is not greater than the diameter of header rock. Along the arms, however, the height of the structure subjected to flow may require force balance analysis to ensure that the weight of the rock, particle-to-particle contact forces, and fluid forces are not resulting in movement of the rock. The structure height above the bed at any point along the structure should generally not exceed 3-4 feet for maximum stability unless a modified design is applied, such as grouting of material or a ramp-like structure. The size of the rock is often dependent upon what is available locally. However, most rock weirs use blocky or angular material that is 2-4 feet in diameter.

Based on a brief literature review of existing rock weir sizing guidance, rocks between 3 and 4 feet in diameter should be used where possible. When this material size is impractical to obtain or not justifiable within the field condition of the stream or river, attempt the CA RSP revetment method described above or apply the following equation developed by the U.S. Army Corps of Engineers (1994) for design of steep slope riprap, with slopes up to 20 percent.

$$D_{30} = \frac{1.95 S^{0.555} q^{2/3}}{g^{1/3}} \tag{7.12}$$

where D_{30} = 30 percent material size in the uniform gradation;

S = slope of the bed;

q = unit discharge; and

g = acceleration due to gravity.

This equation was developed based on the use of angular rock with a unit weight of 167 lb/ft^3 in a steep rock slope ranging from 2 to 20 percent, a bed thickness of 1.5D$_{100}$, a gradation (D$_{85}$/D$_{15}$) between 1.7 and 2.7, uniform flow and no tail water. This equation was not intended to be used in the design of rock weir sizing based on these criteria. However, it may provide a good option when larger material is limited or when applied in small rivers with a bankfull unit discharge less than 40ft^2/s. If using this equation for rock weir sizing, it is recommended to set the slope to 20 percent since this is the maximum slope that the equation was developed for even though the slope of a rock weir may be between 30 and 75 percent. Also, as an estimate for the critical unit discharge, consider applying the bankful unit discharge to represent the conditions when mobilization potential could be greatest. Finally, a factor of safety to the computed stone size may be used to account for the differences between the intended use of this equation and how it is being used.

In some cases, rock sizing should consider the potential for vandalism. Consider choosing a minimum size rock that would typically deter someone from moving it. In the case of rock weirs, simply pushing the rock over may destabilize the entire structure. For rip rap, the Army Corps of Engineers designs to a median weight of 80 lbs, which equates to about a 1 ft diameter rock (1994). Overall, when sizing rock weirs, use professional judgment. If the material size that is required to prevent mobilization of constituent rocks is unavailable, consider an alternative design, such as a rock ramp or grouted rock weir.

7.7 Spacing of Structures

7.7.1 Current Methodologies

Structure spacing is an important parameter to consider for structure design. The spacing of structures is highly dependent upon the goal of the project. 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. 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.

Most existing methods for spacing rock weirs are related to structures providing a specific function, such as increased pool volume or grade control. Castro (2000) recommends placing cross-vanes in areas where pools would naturally form and if the elevation change is greater than one foot they should be used in series to meet fish passage criteria. Castro also recommends that rock weirs for grade control be placed no closer than the net drop divided by the channel slope. From a constructability perspective, the California Department of Transportation

(Caltrans, 2007) recommends spacing structures in series at least 25 feet apart, noting that this is the minimum spacing possible without structures intersecting, losing their physical definition, and impacting pool volume and fish passage.

Spacing rock weirs for irrigation diversions is often evaluated as a function of the required head for diversion for a specific design flow. Hydraulic calculations are performed to estimate the backwater effects from a structure, and the next upstream structure is placed at a location where the required tail water elevation for the upstream structure is met (personal communication, Humbles 2009). If the structures are placed too close together, they may become submerged and not function as intended. Hydraulic calculations and backwater effects for a given structure design and reach characteristics can be calculated using the methods presented in Section 7.2.

Multiple research efforts have evaluated channel spacing relative to structure stability and pool maintenance along the three forks of the Little Snake River. Reclamation (2009b) found that asymmetrical U-weirs on 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. Meyer reported that closely-spaced structures increased the likelihood of a downstream structure interfering with the hydraulics of the pool associated with the structure immediately upstream.

Numerous authors recommend using a spacing equivalent to what would be expected to occur in a natural channel between pools. Various methods for calculating natural pool-riffle and step-pool spacing can be found in existing literature (Table 7.4). Natural pool spacing will be variable dependent on the channel morphology and also potentially on regional differences (Chin, 1999). For example, natural step-pool spacing in steep, gravel to boulder-dominated mountain streams characterized as rapids or cascades has been documented to range between 1-4 channel widths and may be estimated by the following general equation from Chin and Wohl (2005).

$$L = \frac{K}{CS^z} \tag{7.13}$$

where L = length between steps;
S = channel slope;
K = bed element height; and
C, Z are constants.

Originally in New Zealand streams (Whitaker, 1987) and later supported by data from Israel (Wohl and Grodek, 1994) and the Western United States (Grant et al.

1990), a K/C value of 0.31 coupled with a z value of 1.19 accurately represented step-pool spacing in meters. Later, Chin (1999) identified a stronger relationship using a K/C value of 2.67 coupled with a z value of 0.206 for streams in Santa Monica, California (metric units).

Natural, free-forming pool-riffle sequences were observed in alluvial and bedrock channels to be strongly correlated to channel width (Keller and Melhorn, 1978) based on the relationship:

$$L = 5.42x^{1.01}$$ (7.14)

where L = length between pools (m), and
x = channel width (m).

A widely used rule of thumb for pool-riffle channels is a pool-to-pool spacing of 5 to 7 channel widths (Leopold et al., 1964; Keller and Melhorn, 1978). More recent research also identified bed material size, channel curvature, and the presence of large wood to affect the spacing of pool-riffle sequences (Lofthouse and Robert, 2008; Montgomery et al., 1995).

Rosgen (1996) developed an equation for pool spacing using data from natural channels with slopes ranging between 1.5 and 8 percent in the form of:

$$P_s = 8.2513S^{-0.98}$$ (7.15)

where Ps = the ratio of pool to pool spacing/bankfull width, and
S = channel slope in percent.

Table 7.4 – Naturally formed pool spacing estimates.

Method	Channel Type and Location	Reference

$L=0.3133*S^{1.188}$
where:
    L=step-pool spacing (m)
    S=channel slope
	Step-pool channels in New Zealand, western United States, and Israel	Whitaker 1987; Grant et al. 1990; Wohl and Grodek 1994

$L=2.67*S^{0.206}$
where:
    L=step-pool spacing (m)
    S=channel slope
	Step-pool channels in Santa Monica Mountains, Southern California	Chin 1999

Step-pool spacing = 2–3 channel widths		Knighton 1998

$L=f(H, ACW, S_o, q_{design})$
where:
    H=Weir Drop Height
    ACW=Active Channel Width
    S_o=Channel Slope
    q_{design}=Design unit discharge
	Step-pool channels	Thomas et al. 2000

Step-pool spacing = 0.43–2.4 channel widths	Step-pool channels in Santa Monica Mountains, Southern California	Chin 1989

$R=4.5/S^{0.42}$
where:
    R=Ratio of mean step length to mean step height
    S=channel slope
	Step-pool channels in Nahal Yael Watershed, Israel with reach average gradients ranging from 0.07 to 0.67	Wohl and Grodek 1994

$H/L=1.5*S$
where:
    H=Mean step height
    L=mean step length
    S=channel slope
	Flume data combined with 18 steep headwater reaches in the Adirondack Mountains, New York and Lake District, England	Abrahams et al. 1995

$Ps = 8.2513 S_{\%}^{-0.9799}$ where: Ps = pool spacing/ bankfull width	Slopes ranging from approximately 1% to 8%, locations not documented	Rosgen 1996/2006
Pool-riffle spacing = 5 to 7 channel widths	Alluvial and bedrock pool-riffle systems	Leopold et al. 1964; Keller and Melhorn, 1978

$Y=5.42x^{1.01}$
where:
    Y=pool-to-pool spacing (m)
    x=channel width (m)
	Alluvial and bedrock stream channels in Indiana, Virginia, North Carolina, and California	Keller and Melhorn, 1978

7.7.2 Recommendations on Structure Spacing

For grade control and structures that require maximum stability, 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 will protect the downstream structure from experiencing increased velocities and shear stresses that result from the plunging flows over the upstream structure. When a series of structures are used to divert flow for

irrigation, the structures should be spaced such that the downstream structure causes sufficient backwater to create a tailwater condition that meets the required drop height (often determined by fish jumping requirements) for the upstream most structure while promoting sufficient head for diversion. Computational hydraulic modeling is 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 and to maintain consistency with natural channel configurations, spacing of structures should entail a geomorphic investigation to determine the average size and frequency of pools that the system can sustain. Because of the variability in the "natural" spacing of step-pool and pool-riffle morphologies, a field investigation of the system should be accomplished to understand the processes controlling pool spacing and to identify typical spacing that would likely occur naturally in each stream. Design of structures in series may use natural pool spacing to guide the longitudinal proximity of the structures. If the project area spans a long reach of channel, consider adjusting structure spacing along the channel to correspond to variability in channel slope (Meyer, 2007). When budget and time constraints limit the ability to perform a geomorphic investigation of pool spacing, consider applying the equations presented in Table 7.4 using the conditions of each study to help determine which spacing is most appropriate for a given system.

7.8 Notches in Rock Weirs

Notches are often used in rock weir designs to ensure that fish passage criteria are met for low and medium sized flows. Notches also promote the transport of sediment through the structure that might otherwise be deposited just upstream of the structure. While notches provide benefits to the structure, they may also hinder the ability of the structure to maintain sufficient head for irrigation diversion if sized inappropriately. To improve understanding of how notches in rock weirs impact water surface elevations just upstream of the structure and velocities throughout the structure, a two-dimensional modeling effort was undertaken as a first look into the hydraulics associated with the presence of various sized notches. A brief discussion of the model and finding is presented in this section. A more detailed report of the investigation is provided in Appendix A.

A two-dimensional model (SRH-2D) was developed to analyze the influence of notches on depths and velocities for one channel configuration, three discharges, and four notch scenarios, and was modeled after a typical Pacific Northwest gravel-bed river. Channel dimensions, including width, depth, and slope, were derived from the Entiat River in Washington, which is characterized by a fairly constant slope of 1 percent and a channel width of approximately 100 feet for the lower 16 river miles. The four topographic scenarios included: (1) a no notch condition, (2) 5-ft notch, (3) 10-ft notch, and (4) 15-ft notch. Discharges used in

the analysis included 150 cfs, 500 cfs and 3,000 cfs. A discharge of 150 cfs has an exceedance probability of approximately 66 percent and represents the average base flow from September to March. The 500 cfs discharge represents a 25 percent exceedance probability on the Entiat River and about a one-third bankfull depth. The 3,000 cfs represents an approximate 2-year discharge on the lower Entiat River and a bankfull discharge for the design geometry.

Results of the modeling effort (Table 7.5) illustrate that the structure geometry along with the discharge determine how far upstream of the structure the water surface elevation is impacted for a specific channel geometry configuration. Differences in simulated depths between the notched and no notched conditions are illustrated in Figure 7.13 for low flow, and differences in simulated velocities are presented in Figure 7.14 for bankfull discharge. Slight increases in velocity were noted upstream of the structure when a notch was present due to the reduced ability of the structure to create the same backwater condition as is produced when no notch is present. A notch resulted in reductions in velocity in the structure throat, likely due to the removal of the drop over the structure. With a notch, reduced velocities were also simulated along the crest of the structure and along the downstream side of the structure arms as more flow is conveyed through the throat of the structure and less over the structure arms. Just upstream and downstream of the structure throat, increased velocities were predicted for each notch configuration extending into the scour pool.

Results from this effort are given as examples and may not be consistent across all channel slopes, geometries, and rock weir configurations. However, results do indicate that water surface elevations can impact the required head for irrigation differently across a range of flows from low flow to bankfull flow. Comparisons of the water surface elevations for each notch configuration suggest the need to allow for freeboard in the design of a diversion structure that is located within at least 2 channel widths of the structure or to consider placing the structure far enough downstream from the diversion intake that a notch would not impact the head for the design discharge. The sill of the notch and structure may also be adjusted to meet necessary depth requirements upstream, provided that fish passage criteria can still be met.

Table 7.5 – Distance upstream from structure throat that water surface elevations for each notch scenario deviate from the no-notch condition for the modeled channel and structure geometry.

Size of Notch	Discharge (cfs)	Distance upstream WSE is impacted (ft)	Channel widths upstream WSE is impacted
5 ft	150	122	1.31
10 ft	150	130	1.39
15 ft	150	137	1.47
5 ft	500	115	1.23
10 ft	500	136	1.46
15 ft	500	147	1.58
5 ft	3,000	75	0.80
10 ft	3,000	106	1.14
15 ft	3,000	126	1.35
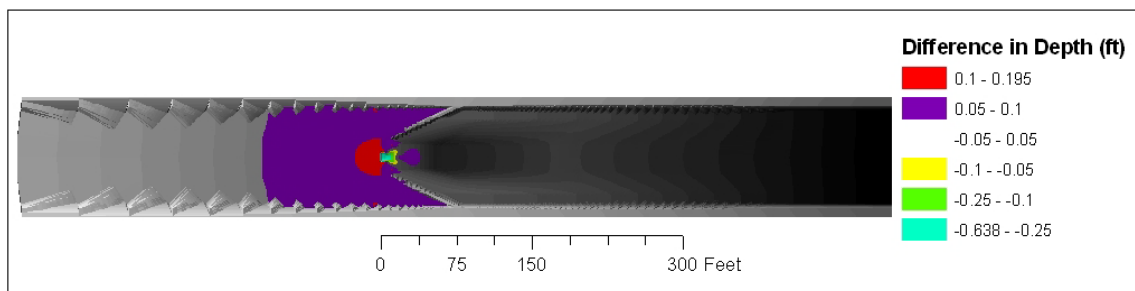
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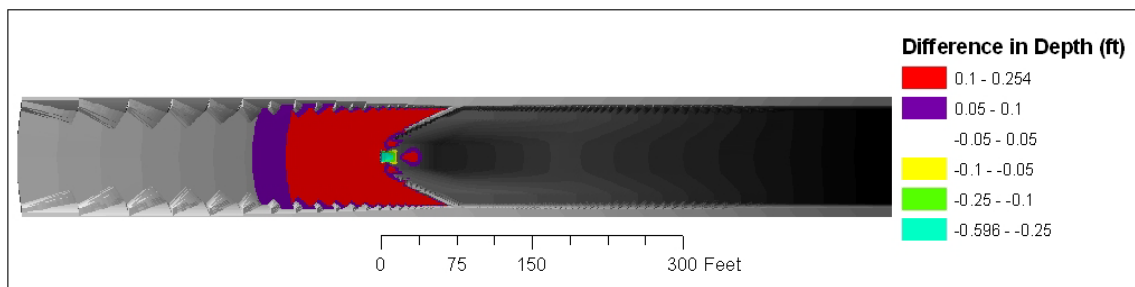
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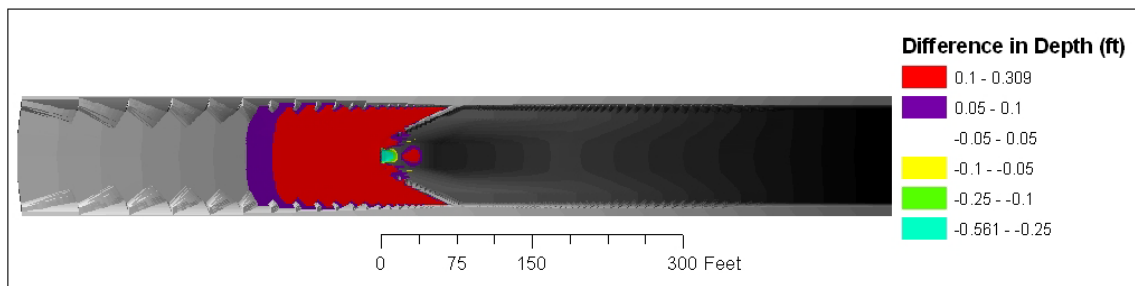
c

Figure 7.13 – Difference in depths between the no notch condition and the 5-ft notch (a), 10-ft notch (b), and 15 ft notch (c) for a discharge of 150 cfs. Positive differences indicate that the no notch condition depths were greater than the notched conditions.
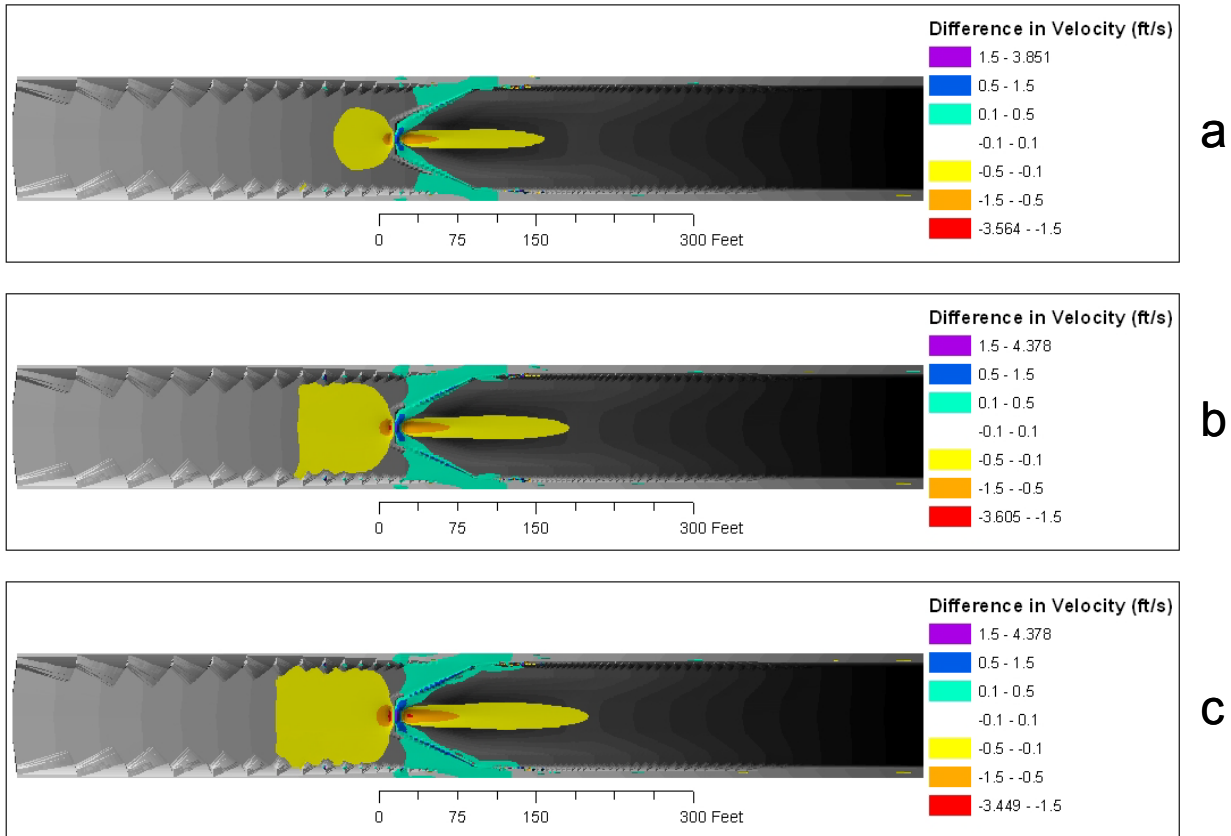
Figure 7.14 – Difference in velocities between the no-notch condition and the 5-ft notch (a), 10-ft notch (b), and 15 ft notch (c) for a discharge of 3,000 cfs. Positive differences indicate that the no-notch condition velocities were greater than the notched conditions.

Investigation of differences in the velocities reveals considerations that should be made in the design of rock weirs. First, notches may reduce the amount of fine material that settles upstream of the structure by allowing a channel through which sediment can be conveyed. At low flows, the notch may result in temporary increased deposition of fines in the scour pool. However, this material would likely scour out of the pool during the medium and high flows. In addition to sediment transport, the presence of a notch appears to slightly reduce the velocities along the downstream side of the structure arms at all flows, which may slightly reduce the required depth of foundation along the structure arms. For each modeled discharge, velocities just downstream of the structure throat increased with notched conditions, which suggests the need for additional foundation protection along the throat of the structure and into the scour pool when notches are used in the design.

A two-dimensional model is recommended for each reach where rock weirs are proposed for irrigation diversion to evaluate impacts on velocities and water surface elevations upstream of the structure. If budget and time constraints do not allow for collection of adequate topographic information for the development of a

two-dimensional model, an automated mesh generator (as used in this analysis) could be applied to develop a simplified version of the topography and model different notch scenarios. Because of the three-dimensional nature of plunging flows along the structure crest, the differences in velocities should not be taken as absolute values and were only used in this modeling effort to investigate potential design considerations. If detailed velocity information is desired along the structure and within the scour pool, a three-dimensional model may be needed.

In addition to the impacts on the water surface elevations, field investigations of multiple river spanning rock structures performed in 2008 (Reclamation, 2009b) included input from biologists on each structure's design. Biologists repeatedly indicated interests in having more flow diversity at low flows for fish passage and habitat. The notches helped provide the desired "messiness" associated with increased flow diversity. Multiple small notches were inadvertently created at several structures along Bear Creek in Oregon during high flow events, providing multiple "sneaks" through which young fry were capable of moving during low flows. While the single notch concept helps meet jump criteria for some species, the diversity of velocities through multiple notches may improve passage across multiple life stages. The concept of multiple notches or a structure defined by more flow diversity may be more useful when the structure is intended for grade control or to improve fish passage rather than to provide required head for irrigation diversion.

7.9 Long-Term Maintenance Issues and Expectations

Assessing structure maintenance, replacement, and risk in a design is often related to the likelihood of a specific hydrologic event taking place. However, failures and maintenance of instream structures are not always directly related to flood events. Although a structure may be designed to withstand a 25-year event, multiple other processes may influence the structural stability prior to the occurrence of a 25-year event. For example, in a highly dynamic system, a structure may fill with sediment or active migration may cause flanking of the structure. Furthermore, this research effort has determined that the chance for failure does not always increase with increasing discharge. As more water is conveyed out of bank, the shear stresses and velocities experienced by a structure or channel bed do not always increase; these are dependent upon the channel and valley morphology.

When designing a river spanning rock structure, the size of the components of the structure is typically determined by that required to remain immobile during a specific flow event. One method to determine the design flow event is presented in Section 7.4.3 and is based upon the flow responsible for producing the maximum scour depth. The selection of a design event is often used to balance the present cost of constructing more resilient structures with the cost, effort, and likelihood of replacing or repairing weaker structures if a larger flow event occurs.

Hydrology drives the hydraulic forces that may result in the failure of a structure. A design can protect against mobility of constituent rocks for flows up to a predetermined magnitude. Protecting against mobility at larger flow events may require more expensive materials in larger quantities. Smaller, less resilient structures may require more frequent repairs but result in lower economic costs. Future flow rates cannot be deterministically predicted, but stochastic analysis can describe the likelihood of flow patterns over long time periods at many structures. Even with an understanding that mobility of structure components is not the sole or primary method of failure with river spanning rock structures, an evaluation of the likelihood for a specific event to occur can be a valuable tool for assessment of long-term maintenance.

The following sub-sections describe methods to determine the probability of design events occurring in a structure's lifetime, as well as present a method to quantify the potential need for maintaining or replacing a damaged structure.

7.9.1 Probability of Failure and Maintenance Requirements

A hydrologic analysis determines the likelihood of occurrence for a given flow rate. The likelihood is typically expressed in terms of a yearly return interval on the maximum annual peak discharge. For example, the 10-year return flow represents the flow rate that has a 10-percent chance of occurring in any given year. The 10-year event does not occur in a regular or predictable pattern, but can happen in any year or even several years in a row. A binary distribution describes the likelihood of experiencing multiple events within a particular time period. The binary distribution can predict the likelihood of the 10-year event occurring six times over a 50-year period. Figure 7.15 illustrates an example binary distribution for the number of 10-year recurrence flows over a 50 year time period.

Using Figure 7.15, one can predict the probability and number of times that a 10-year event will occur over 50 years. For example the probability that a 10-year discharge will occur exactly 7 times within 50 years is 11 percent, and the probability that a 10-year discharge will occur 7 times or less within the 50 year is 87 percent. The probability of experiencing the 10-year event exactly five times is 18 percent and is the most likely outcome over the 50 year period. This follows expectations as 50 years divided by a 10 year average occurrence equals 5 events. There is a 94-percent chance the structure will experience the 10-year event up to and including 8 times within the next 50 years. Conversely, there is less than a 6-percent chance the structure would experience the 10-year event more than eight times. There is a 0.5-percent chance the structure will never see a 10-year event throughout a 50 year time span.
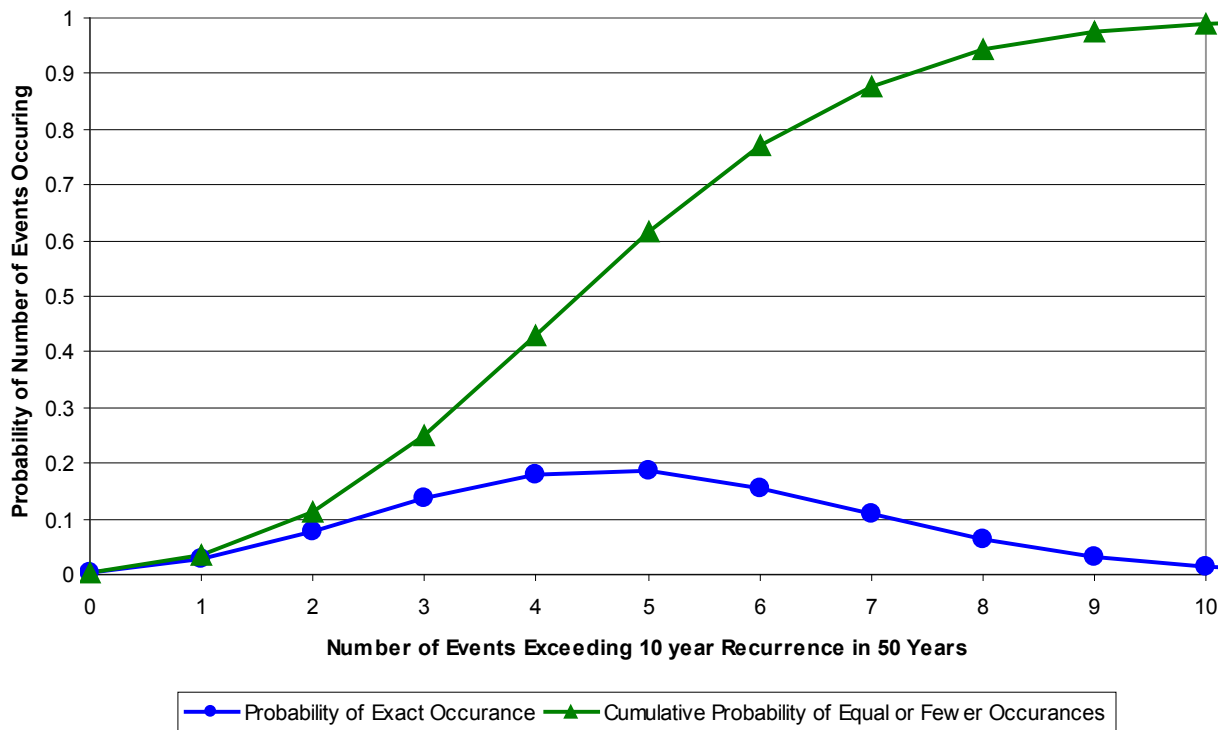
Figure 7.15 – Probability of a 10-year discharge occurring during a 50-year time span.

The probability of an event meeting or exceeding a specific discharge can be used to represent a risk of repair or replacement requirements for structures over a given time period. The cost to repair or replace a structure is the consequence of that risk. Figure 7.15 is a general plot applicable across all sites. However, changing the lifespan from 50 years to another time period or changing the recurrence event will result in a different curve. Equation 7.16 shows the equation for computing the binary distribution for a specific number of outcomes.

$$b = \binom{n}{x} \cdot p^x \cdot (1-p)^{n-x} \quad ; \quad \binom{n}{x} = \frac{n!}{(n-x)!} \qquad (7.16)$$

where b   = probability of exactly x events occurring in n years;

    x   = number of events occurring in n years;
    n   = number of years;
    p   = probability of x event occurring in a single year (recurrence probability); and
    !   = a mathematical operator called a factorial.

A design event provides input to determine specific design parameters and evaluate processes active in the reach during the event, such as sediment transport, scour, and floodplain interaction. For rock weirs, the design discharge can impact the selection of riprap size and quantity, foundation depth and possibly

structure type (e.g., U-, A-, W-weirs) and material type (e.g. blocky, rounded, concrete, natural).

In the assessment of expected maintenance due to structure mobility, economic planning can divide lifecycle costs into repair and replacement. Failure to maintain or repair structures will likely result in a need for subsequent replacement after much lower flows than the design discharge. A specific design event may be associated with some mobility of a structure or partial failure, thereby requiring repair, while another event may be associated with catastrophic movement and complete failure.

Lifecycle probability curves can be applied in design to evaluate potential repair and replacement costs over the expected life of a river spanning rock structure. Figure 7.16 shows two 50-year lifecycle probability curves side-by-side assuming a 10-year event will require repair and a 25-year event will require replacement of the structure. For a given probability, a structure will require replacement less often than maintenance. In the example, there is a 40-percent chance of the structure requiring replacement once, or not at all within a 50-year time frame. There is also a 40-percent chance the structure would require maintenance four times or fewer within a 50-year time frame. There is a 60-percent chance the structure will require 5 or fewer repairs and two or fewer replacements. Both relationships were developed using Equation 7.16 differing only in the recurrence probability, and apply to all sites for the same lifecycle, maintenance probability, and failure probability.

This analysis neglects changes in water use, global climate shifts, and morphologic adjustments which change repair or replacement probabilities. This analysis further neglects temporal and spatial correlation by assuming independent outcomes. Climatic patterns tend to cluster temporally where a series of wetter years follows a series of dryer years in cyclic patterns. Structures in the same or nearby basins should show similar flow patterns. The 50-year event would likely occur over a significant portion of a basin rather than a single structure. More advanced hydrologic analysis is possible, but is beyond the scope of these design guidelines. With many structures over large areas and long time frames, the significance of the correlation decreases. However, longer time frames increase the importance of water usage trends, climate change, and morphologic adjustment.
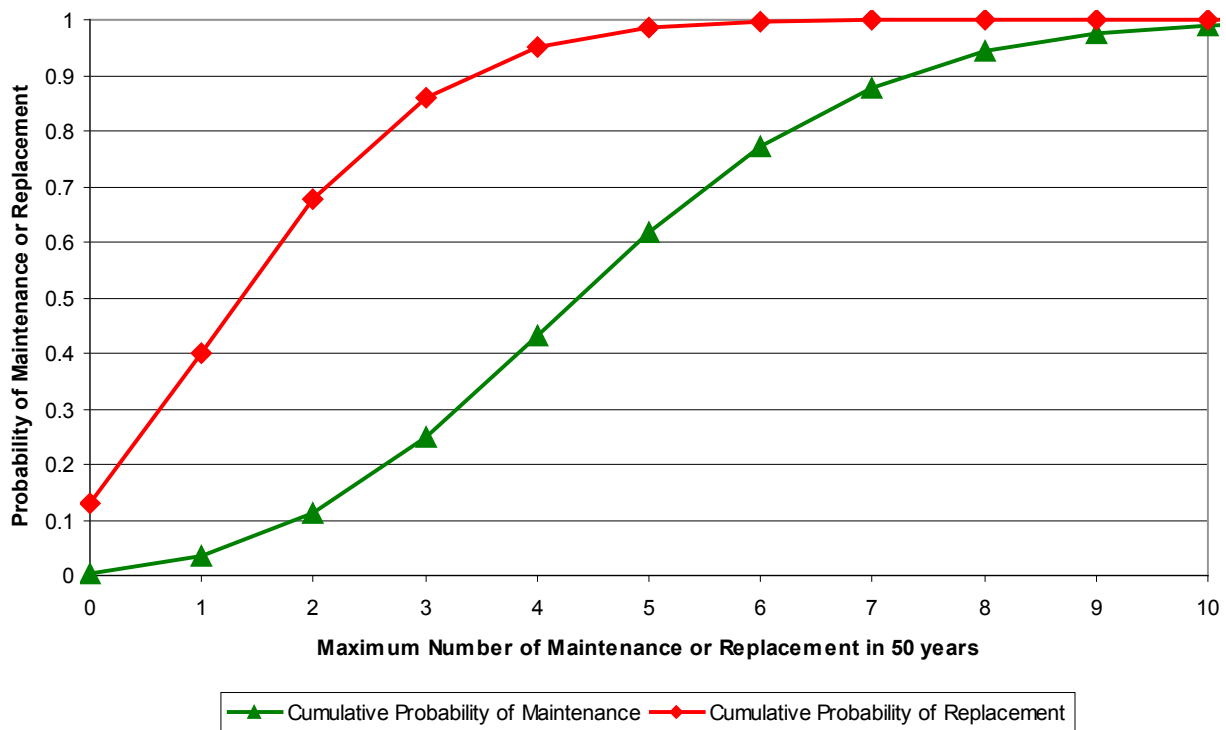
Figure 7.16 – 50-year maintenance and replacement plots assuming a 10-year flood requires repairs and a 25-year flood causes a failure.

Design practices incorporate safety factors to account for the unknown. The use of safety factors increases the resilience of structures, and structures should require maintenance and replacement less often than predicted. Practically, the increased resilience cannot be relied upon and unforeseen circumstances can reduce the lifespan in an unquantifiable manner. Improper or inadequate design and construction can increase the replacement and maintenance rate. In almost all structures visited as part of the field investigation component of this research, some degree of maintenance was needed in the near future or had previously occurred. Rivers are dynamic in nature, and all structures within a river channel are exposed to risks associated with fluvial processes. Because rock weirs attempt to provide only a semi-hardened structure, the potential for experiencing some degree of failure throughout the structure's lifespan is high.

8 Additional Considerations

8.1 Habitat Considerations

River spanning rock weirs often have purposes associated with improving aquatic habitat. In most Bureau of Reclamation river spanning rock structure projects, objectives for aquatic habitat include improved fish passage, improved pool depth, area, or volume, and improved fish cover. During field investigations, biologists from various regions were questioned regarding the usefulness of river spanning rock structures in meeting habitat objectives or in unintentionally increasing habitat features. Although qualitative in nature, biologists provided insight into key considerations for design. Within this section, information contributed by biologists in the field and considerations for design found within recent literature is documented.

River spanning rock structures can be beneficial to habitat for many reasons. Rock ramps with dispersed boulder components or weirs that are not interlocking increase diversity of flow velocities over and through the structure. In geomorphically appropriate locations, rock weirs increase scour pool development downstream from the structure crest, which may provide adult holding habitat and/or improve juvenile migration. However, structures in series placed too closely may limit pool volume (Meyer, 2007). In addition, rock weirs are typically designed to concentrate flows through the structure throat, thereby promoting low flow channel passage for fish.

The effectiveness in modifying physical habitat through implementation of instream structures for the benefit of multiple fish species is well documented (Roni et al., 2002). Fewer studies have investigated fish response to the placement of rock weirs. A detailed review of instream structure effectiveness in stream rehabilitation can be found in Roni et al., 2005. In 2002, Roni et al. performed a review of the effectiveness of instream structures on Pacific salmonids and found that if implemented correctly, instream structures, such as rock weirs, provide benefits to species and life stages that prefer pool habitat. However, results of the investigation suggest that species and life stages that prefer shallow edge habitat may not benefit from the placement of instream structures. Adult salmonids benefit from increased spawning habitat resulting from instream structures, of which the most effective structures appear to be ''V'' weirs in streams with slopes less than 3 percent (Anderson et al., 1984; House and Boehne, 1985). Other studies suggest rock weirs and constructed riffles may increase diversity (Shields et al., 1995) and abundance of non-salmonid and entire fish communities (Linlokken, 1997; Pretty et al., 2003).

During field visits to rock weir sites in the Pacific Northwest, biologists identified additional cover in the form of large woody material (LWM) as a potential

method to improve habitat at existing structures. While rock weirs are often used to mimic natural formations of LWM, improved cover is not typically a primary objective of rock weir implementation. However, engineered LWM structures are more frequently being added as components to the river spanning rock structure designs. In one study of rock weirs in Asotin Creek Watershed, Washington, measured juvenile salmonid densities were the greatest at sites that incorporated LWM (Johnson, 2000). River spanning rock structure designs can maximize the potential long-term benefits of stream rehabilitation by coupling the installation of rock structures with more permanent efforts to restore riparian vegetation, which ultimately can become sources for LWM inputs. However, the addition of LWM as structural members of a rock weir design may reduce the weir stability and increase risks of failure and required maintenance.

At several sites where channel adjustments have occurred since the implementation of the structures, fish passage has become a challenge for juvenile salmonids. Biologist Brad Smith with the Oregon Department of Fish and Wildlife recommends adding a step downstream from the throat to reduce drop and improve juvenile passage. Smith also identified partial failures of structure components as potentially beneficial to juvenile passage. At several rock weirs along Bear Creek, Oregon, slots through structure arms formed from structure settlement provided micro channels through which juveniles salmonids could pass during low flows where the elevation drop and/or velocities through the throat are too high. Multiple small notches between the rocks comprising the structure arms increase the potential for flow diversity, but may be difficult to design when a specific irrigation head is required. Several biologists identified a "messy" structure, one that is somewhat deformable, as having the greatest benefit for juvenile fish passage and topographic diversity.

For combined objectives of fish passage and irrigation, most biologists interviewed preferred a rock ramp design to a rock weir design because of the ability of fish of all life-stages to easily migrate through the structure. The ramp provided more diversity of flows, roughened features to allow short-term resting habitat, and larger boulders to provide shadow cover from predators.

Failure rates of rock weirs are generally high over a 10-year time frame. As a result, the benefits of rock weirs on habitat are generally temporary and can be of greatest value when used to provide short-term habitat while longer-term processes, such as vegetation re-establishment and LWM recruitment, are restored. To maximize the benefits of rock weirs in improving habitat, the geomorphology of each proposed site and its ability to sustain a sought after morphologic form must be understood. Because of the short life-span of river spanning rock structures, designs will provide the greatest benefit to the ecosystem through the use of native materials placed consistent in size, type, location, and orientation to that found in natural channels (Roni et al., 2002).

8.2 Rootwads

Rootwads are a type of bioengineering that utilizes the lower trunk and root fan of a large tree. In typical stream restoration projects, individual rootwads are placed in series and utilized to protect eroding stream banks along meander bends while also providing some instream habitat. Using bioengineering methods such as rootwads in conjunction with rock weirs can provide additional bank stabilization and enhanced instream habitat conditions. Rootwad revetments promote the formation of pool habitat and overhead cover along the streamside portion of the rootwad fan while providing sediment deposition along the downstream edge of the bank tie-in location. The use of rootwads along the bank edges and near the tie-in location of the rock weir could provide additional energy dissipation and sediment deposition that would allow for additional protection against bank erosion. Rootwads and the local associated scour holes also provide habitat for fish and other aquatic animals, as well as a food source for aquatic insects.

Due to the increased concentration of flow in the center of the channel caused by rock weirs, placing rootwads within this area is not recommended. Doing so could greatly alter the performance of the rock weir in creating available pool habitat, pose a risk for recreational users (boats/kayaks) by creating in-channel obstructions, and would most likely not remain in place without a large amount of anchor material given the increased velocities.

8.3 Uncertainty Analysis / Risk Assessment

Inherent risks are associated with all stream restoration or rehabilitation projects, particularly with those that aim to install rigid structures in dynamic systems. River spanning rock structures are not permanent features and some continued maintenance should be anticipated and built into an adaptive management approach for their design. Uncertainty is associated with the occurrence of flood events, channel migration, patterns of erosion or deposition, scour hole development, and many other processes that may influence the longevity of a river spanning rock structure. Analyses can and should be performed to reduce the risk of failure, and structures can be designed following the recommendations proposed within this document to decrease the potential for failure. Unfortunately however, there is not a procedure that completely eliminates the potential for failure, whether related to structural components or intended habitat features.

A detailed discussion of the quantification of risk related to hydrology is described in Section 7.9. However, quantification of risk related to all fluvial processes combined is not treated in this discussion because all fluvial processes are not tied to specific flow events. Within this section, a simple procedure is presented to minimize the uncertainty of structure failure through completion of appropriate analyses.

Each river spanning rock structure design requires a distinct set of critical analyses in selecting the appropriate location, applying the appropriate model, and designing the structure parameters. No specific set of analyses can be prescribed for all instream structure designs to maximize certainty. However, basic steps can be followed to reduce the uncertainty associated with instream structure design. These include:

1. Collect and review all available historical and current documentation related to the proposed river spanning rock structure site, including aerial and ground photographs, engineering drawings and reports, interviews with locals, anecdotal evidence, geologic and geomorphic reports, and soils investigations.

2. Perform a field visit with an interdisciplinary team that includes at minimum a fluvial geomorphologist, biologist, and hydraulic engineer.

3. Based on observations in the field and other anecdotal or documented information, identify the necessary analyses to select the location for and type of river spanning rock structure.

4. Throughout the location selection and design process, seek review and feedback from an external team of experts in engineering and geomorphology to ensure that all important aspects of the design are being considered.

Following this protocol will increase the potential for successful design and implementation of a river spanning rock structure.

The challenge with this approach is that the amount of funding and time often required to complete the basic steps may exceed the cost of implementing a river spanning rock structure and/or the time afforded in the project schedule. Costs of analyses and design balanced with the cost of structure installation are often difficult to justify to managers and funding partners. However, the cost of failures and potential impacts to infrastructure or existing habitat may be large compared with the capital cost associated with design. Failure to follow the basic steps outlined may result in greater long-term costs associated with repeated structural modifications, reduced ecological and geomorphic function, decreased confidence from stakeholders, and potentially diminished self-assurance in designers.

No quantifiable method currently exists to determine that if a specific analysis is performed, then the risk of failure will decrease by a certain amount. Monitoring may be initiated to evaluate this type of relationship in the future. However, performing a specific analysis in one river may not be warranted in another, and relationships developed between failure rates and methods of analysis would need to consider differences between controlling processes in each system. Monitoring of this scale would be an immense effort and likely convoluted by differences in analysis approaches. Instead, a more informative approach may be a comparison of success rates with designs that incorporated the basic steps outlined above versus designs that were implemented with no analysis and at a minimal cost.

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Rock Weir Design Guidance
Appendices A and B

U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
Denver, Colorado

Appendix A: Analysis of Notches in Rock Weirs

Notches are often used in rock weir designs to ensure that fish passage criteria are met for low and medium sized flows. Notches also promote the transport of sediment through the structure that might otherwise be deposited just upstream of the structure. While notches provide benefits to the structure, they may also hinder the ability of the structure to maintain sufficient head for irrigation diversion if sized inappropriately. To improve understanding of how notches in rock weirs impact water surface elevations just upstream of the structure and velocities throughout the structure, a two-dimensional modeling effort was undertaken as a first look into the hydraulics associated with the presence of various sized notches.

1. Methods

A two-dimensional model (SRH-2D) was developed to analyze the influence of notches on depths and velocities for one channel configuration, three discharges, and four notch scenarios. The channel was modeled after a typical Pacific Northwest gravel-bed river. Channel dimensions, including width, depth, and slope, were derived from the Entiat River in Washington, where Reclamation has previously installed river spanning rock structures. The dimensions of the modeled channel and structure were similar to two structures installed at river mile 3.1 along the Entiat River. Channel and structure geometries used in the modeling analysis are shown in Table 1.

Discharges used in the analysis included 150 cfs, 500 cfs and 3,000 cfs, and were primarily derived from USGS stream gage 12452990, Entiat River near Entiat, Washington. A discharge of 150 cfs has an exceedance probability of approximately 66% and represents the average baseflow from September to March. The 500 cfs discharge represents a 25% exceedance probability on the Entiat River and about a 1/3 bankfull depth. The 3,000 cfs flow represents an approximate 2-year discharge on the Entiat River and a bankfull discharge for the design geometry. The downstream boundary condition for each modeled discharge was determined using normal depth computations at the downstream end of the modeled reach.

The four topographic scenarios included: (1) a no notch condition, (2) 5 ft notch, (3) 10 ft notch, and (4) 15 ft notch. In each case, the notch was placed in the center of the throat of the structure, and the weir elevation was dropped to the existing bed elevation. The initial bed topography (no notch condition) is shown in Figure 1.

Table 1. Channel and structure geometry for the modeled reach and U-weir. Dimensions were approximated from field measurements on the Entiat River near RM 3.1.

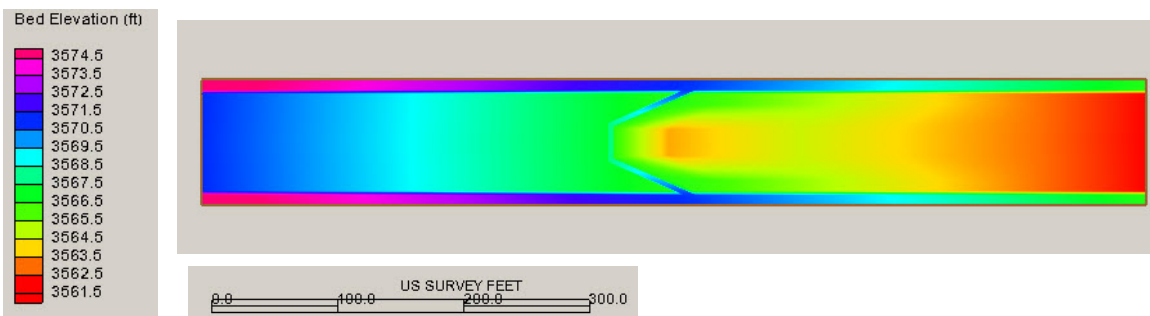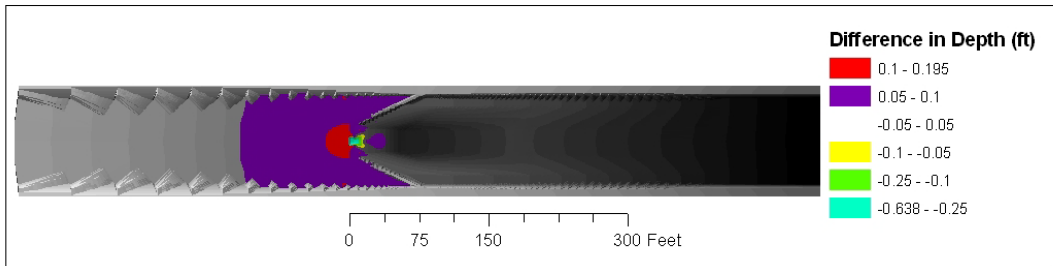| Channel Geometry		Structure Geometry	
Reach Length (ft)	800	Structure Rock Size Width (ft)	4
Slope (ft/ft)	0.01	Structure Rock Size Height (ft)	2.7
Channel Width (ft)	93	Drop Height (ft)	0.8
Depth (ft)	4.5	Throat Width (ft)	33.3
Top Width (ft)	100	Left Arm Angle (degrees)	24
Side Slope (H/V)	0.75	Left Arm Slope (degrees)	2.1
Overbank Width (ft)	10	Left Arm Length (ft)	75
Grain size (d50, mm)	90.5	Right Arm Angle (degrees)	20.3
Manning n	0.040	Right Arm Slope (degrees)	1.7
Normal Depth (Q500, ft)	3.8	Right Arm Length (ft)	90
Normal Depth (Q3,000, ft)	1.3	Maximum Scour Pool Depth (ft)	3.0
Normal Depth (Q150, ft)	0.6		

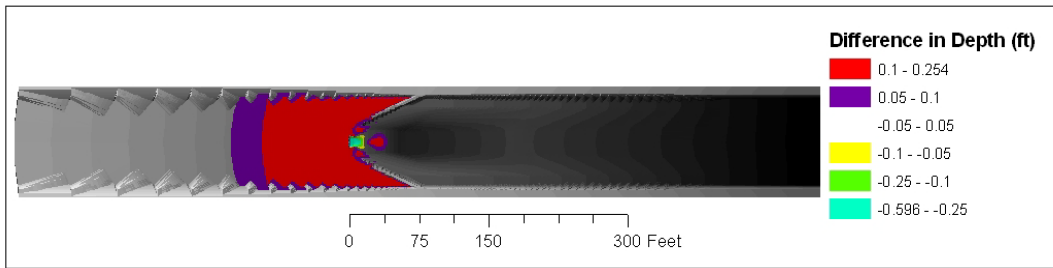**Figure 1. Bed topography modeled for the no notch condition.**

2. Results

Results of the modeling effort illustrate that the channel and structure geometry along with the discharge determine how far upstream of the structure the water surface elevation is impacted (Table 2). Figure 2 through Figure 4 demonstrate the differences in depths between the no notch condition and the conditions modeled with various notch sizes. The side wall triangulation shown in the figures is a byproduct of the node network in ArcGIS and does not accurately represent the walls shown in Figure 1, which have a side slope of 0.75 (Horizontal:Vertical). For a discharge of 150 cfs, the water surface elevations are impacted greater than 0.05 feet from 1.3 to 1.5 channel widths upstream of the structure throat depending on the width of the notch in the structure. However, the degree of impact to the water surface elevations is variable. With a 5 ft notch installed, the water surface elevations are lowered by more than 0.1 ft for a short distance (less than one-half of a channel width) upstream of the structure throat, while the 10 ft and 15 ft notches lower the water surface elevation more than 0.1 ft for a distance of more than one channel width upstream. At 500 cfs, the water surface elevations are lowered for a distance of 1.2 to 1.6 channel widths upstream of the structure throat, and at 3,000 cfs, the water surface elevations are impacted for a distance of 0.8 to 1.4 channel widths upstream from the structure throat.

Table 2. Distance upstream from structure throat that water surface elevations for each notch scenario deviate from the no notch condition for the modeled channel and structure geometry.

Size of Notch	Discharge (cfs)	Distance upstream WSE is impacted (ft)	Channel widths upstream WSE is impacted
5ft	150	122	1.3
10ft	150	130	1.4
15ft	150	137	1.5
5ft	500	115	1.2
10ft	500	136	1.5
15ft	500	147	1.6
5ft	3,000	75	0.8
10ft	3,000	106	1.1
15ft	3,000	126	1.4
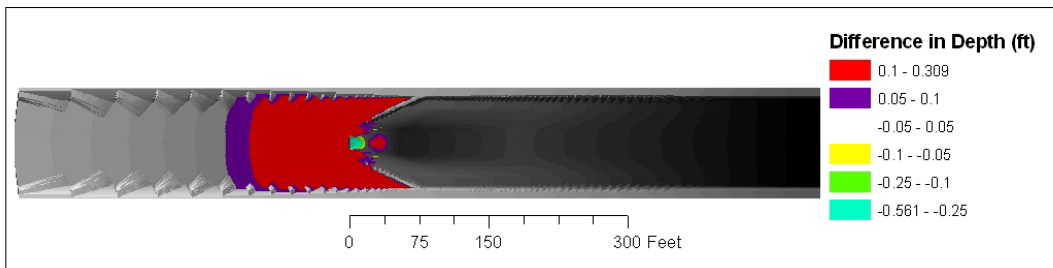
Figure 2. Difference in depths between the no notch condition and the (a) 5-ft notch, (b) 10-ft notch, and (c) 15 ft notch for a discharge of 150 cfs. Positive differences indicate that the no notch condition depths were greater than the notched conditions.
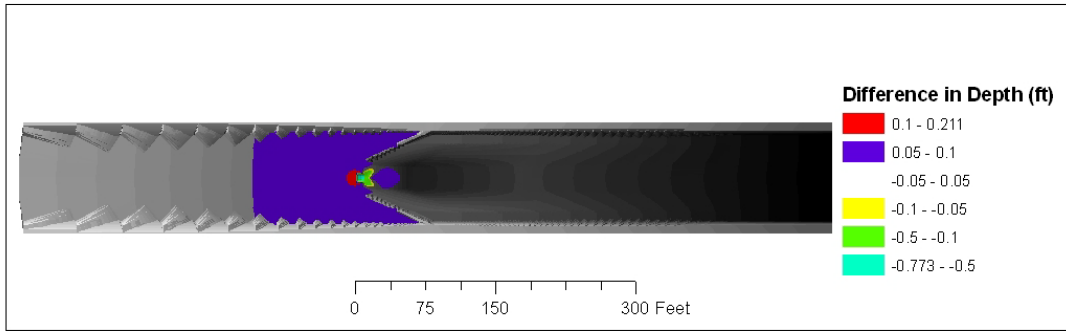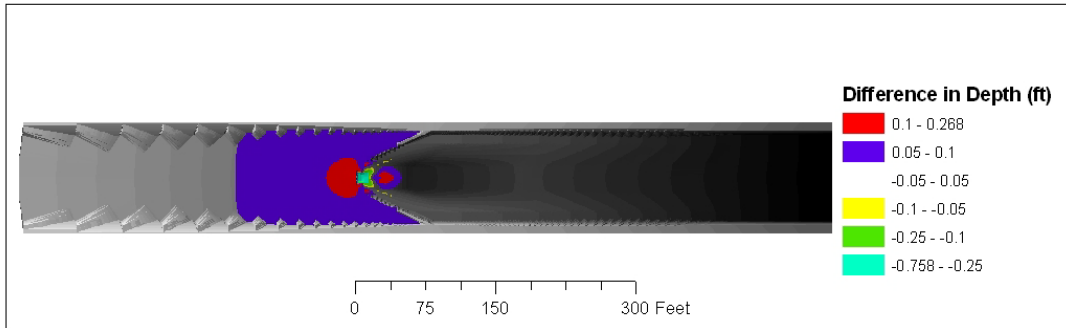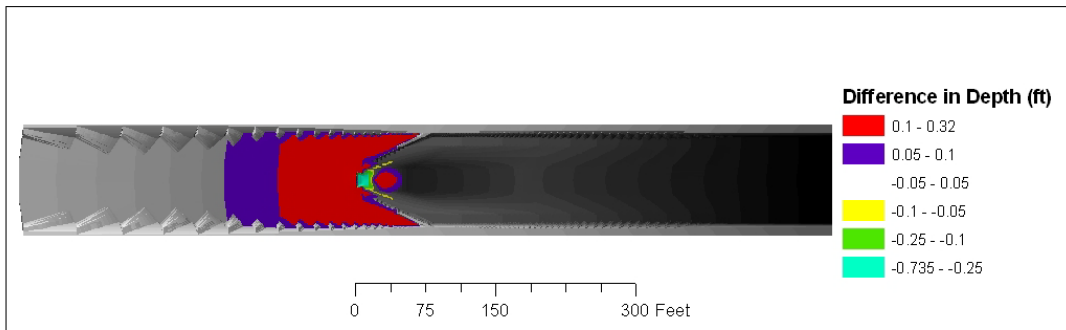
Figure 3. Difference in depths between the no notch condition and the (a) 5-ft notch, (b) 10-ft notch, and (c) 15 ft notch for a discharge of 500 cfs. Positive differences indicate that the no notch condition depths were greater than the notched conditions.

Figure 4. Difference in depths between the no notch condition and the (a) 5-ft notch, (b) 10-ft notch, and (c) 15 ft notch for a discharge of 3,000 cfs. Positive differences indicate that the no notch condition depths were greater than the notched conditions.

Comparisons of the velocities across the different notch conditions and flows are shown in Figure 5 through Figure 7. Difference in velocities less than 0.1 ft/s were considered negligible for this analysis. At low flows, velocities over and just downstream of the structure arms were reduced by the presence of the notch. Slight increases in velocity were noted upstream of the structure when a notch was present due to the reduced ability of the structure to create a backwater condition. More sizeable differences in velocity increases were produced just upstream and downstream of the structure throat. However, reductions in velocity were detected along the crest of the structure throat, likely due to the removal of the drop over the structure. Another area of slightly reduced velocities is present in the scour pool under the low flow conditions.

At the 1/3 bankfull discharge of 500 cfs, slightly increased velocities are present for each notch configuration, again associated with the decreased backwater ability of the structure. Reduced velocities along the downstream side of the structure arms occur with the notch as more flow is conveyed through the throat of the structure and less over the structure arms. Increased velocities just upstream and downstream of the structure throat were predicted for each notch configuration extending into the scour pool.

For the highest flow analyzed, the areas marked by changes in velocities are greater than those of the 150 cfs and 500 cfs model runs. However, the magnitude of the differences is slightly lower along the structure crest, just upstream and downstream of the throat, and along the inside of the structure arms. At 3,000 cfs, increases in velocities due to the presence of the notch extend beyond the footprint of the structure and into the scour pool.

Figure 5. Difference in velocities between the no notch condition and the (a) 5-ft notch, (b) 10-ft notch, and (c) 15 ft notch for a discharge of 150 cfs. Positive differences indicate that the no notch condition velocities were greater than the notched conditions.
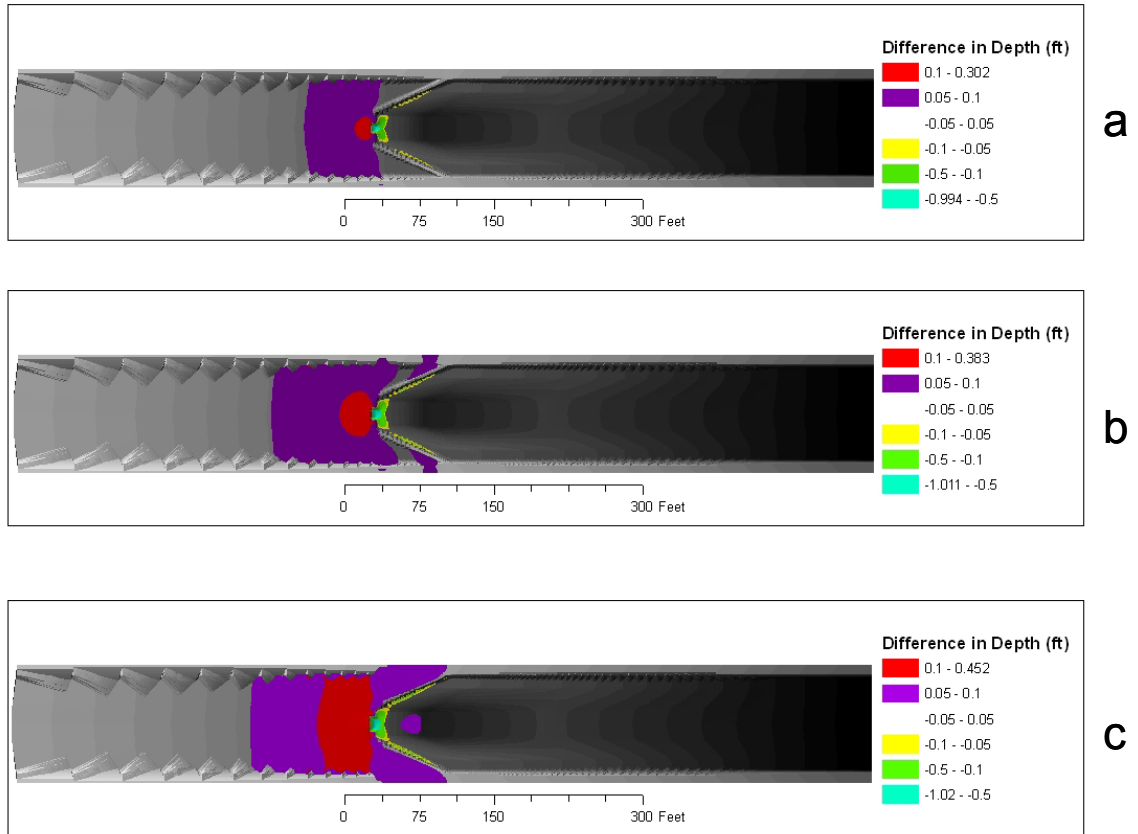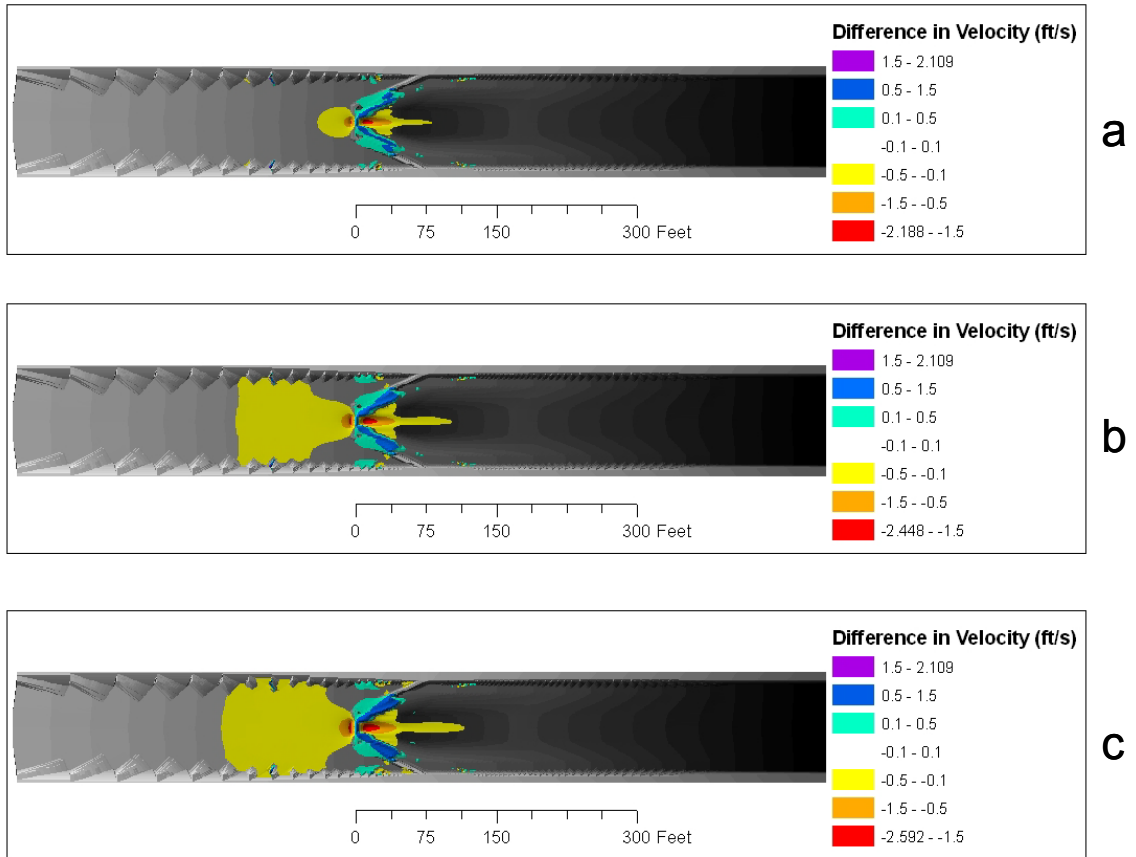
Figure 6. Difference in velocities between the no notch condition and the (a) 5-ft notch, (b) 10-ft notch, and (c) 15 ft notch for a discharge of 500 cfs. Positive differences indicate that the no notch condition velocities were greater than the notched conditions.
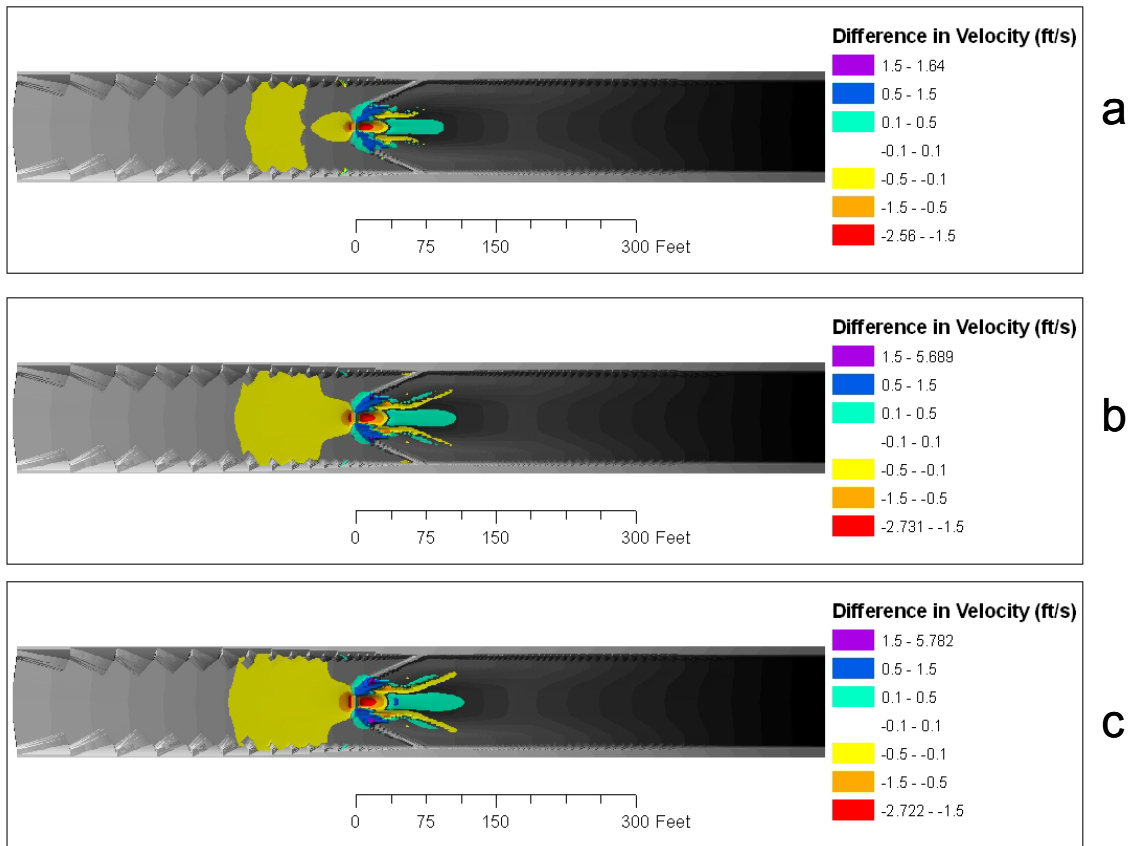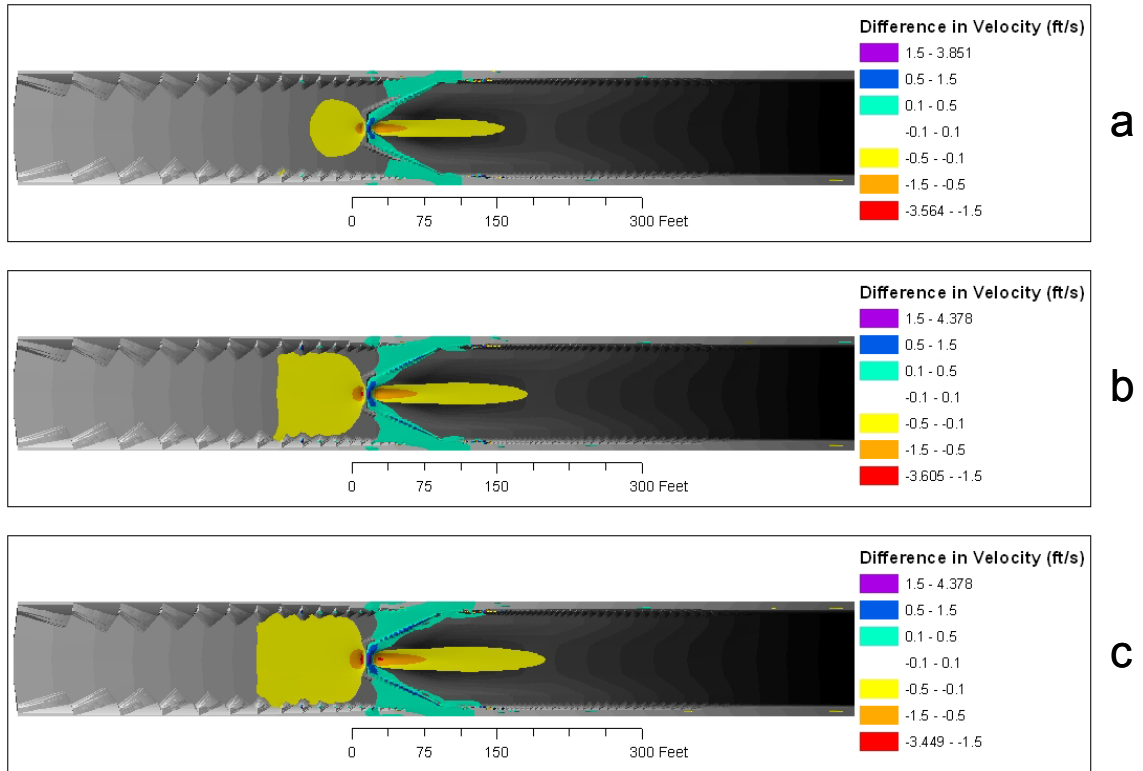
Figure 7. Difference in velocities between the no notch condition and the (a) 5-ft notch, (b) 10-ft notch, and (c) 15 ft notch for a discharge of 3,000 cfs. Positive differences indicate that the no notch condition velocities were greater than the notched conditions.

3. Discussion and Recommendations

The results presented in this section apply only to a simplified U-weir where the channel and structure geometry are similar to those of the Entiat River near RM 3.1. However, the results indicate that water surface elevations can impact the required head for irrigation differently across a range of flows from low flow to bankfull flow. Comparisons of the depths for each notch configuration suggest the need to allow for freeboard in the design of a diversion structure that is located within at least 2 channel widths of the structure or to consider placing the structure far enough downstream from the diversion intake that a notch would not impact the head for the design discharge.

Investigation of differences in the velocities reveals considerations that should be made in the design of rock weirs. First, notches may reduce the amount of fine material that settles upstream of the structure by allowing a channel through which sediment can be conveyed. At low flows, the notch may result in temporary increased deposition of fines in the scour pool. However, this material would likely scour out of the pool during the medium and high flows. In addition to sediment transport, the presence of a notch appears to slightly reduce the velocities along the downstream side of the structure arms at all flows, which may slightly reduce the required depth of foundation along the structure arms. For each

A-9

modeled discharge, velocities just downstream of the structure throat increased, which suggests the need for additional armoring or foundation protection along the throat of the structure and into the scour pool when notches are used in the design.

A two-dimensional model is recommended for each reach where rock weirs are proposed for irrigation diversion to evaluate impacts on velocities and water surface elevations upstream of the structure. If budget and time constraints do not allow for collection of adequate topographic information for the development of a two-dimensional model of the site, an automated mesh generator (as used in this analysis) could be applied to develop a simplified version of the topography and model different notch scenarios. Because of the three-dimensional nature of plunging flows along the structure crest, the differences in velocities should be viewed as comparative and not as absolute values; they were only used in this modeling effort to investigate potential design considerations. If detailed velocity information is desired along the structure and within the scour pool, a three-dimensional model may be needed.

In addition to the impacts on the water surface elevations, field investigations of multiple river spanning rock structures performed in 2008 (Reclamation, 2009) included input from biologists on each structure's design. Biologists repeatedly indicated interests in having more flow diversity at low flows for fish passage and habitat. The notches helped provide the desired "messiness" associated with increased flow diversity. Multiple small notches were inadvertently created at several structures along Bear Creek in Oregon during high flow events, providing multiple "sneaks" through which young fry were capable of moving during low flows. While the single notch concept helps meet jump criteria for some species, the diversity of velocities through multiple notches may improve passage across multiple life stages. The concept of multiple notches or a structure defined by more flow diversity may be more useful when the structure is intended for grade control or to improve fish passage rather than to provide a specified head for irrigation diversion.

Appendix B: Bed Material Data Collection

Bed material information is a critical component of most river restoration or rehabilitation projects, including those relating to the installation of in-channel structures. Three methods of sampling bed material are discussed in this document:

1) Pebble count
2) Volumetric sieve sampling
3) Photogrammetric

1. Sampling Techniques

1.1. Pebble Count Method

Pebble counts are the most commonly used method to determine summary statistics regarding particle-size of gravel and cobble surface sediments on dry bars or within the channel. Data from pebble counts are most often applied to develop a validation data set for sieve samples, or to supplement the sieve data set with additional locations when there are budget and/or time constraints. Pebble counts cannot provide accurate particle size distributions for fine sediments, typically less than 4 mm. Although pebble counts are the most common method of determining size distributions of particle samples (by frequency), several sources of error are associated with the sampling technique that may introduce error into the data in which small differences in particle-size distributions lead to substantial differences in modeling and/or monitoring results. Outlined by Bunte and Abt (2001), the most common sources of error include (1) operator bias toward large particles, (2) operator error and/or bias in site identification and sampling scheme, and (3) statistical errors associated with sample size and precision.

Pebble counts do offer a rapid assessment technique to evaluate surface grain-size statistics across multiple sites. The time required to complete a pebble count and site evaluation ranges from 0.5 to 2 hrs depending on site access and sampling size. Pebble count data are most useful in analyzing differences in gradations across long reaches of river (10s of miles) and monitoring changes in average sediment sizes for gravel- and cobble-dominated river channels over time. However, when time, budget, or sampling equipment techniques limit the ability to perform volumetric sampling, pebble count data can be used as input to scour depth equations and in some sediment transport equations that only require surface sample data.

1.1.1. Procedure

For bars or channels with at least 100 ft of width, the first step of the pebble count data collection method is to lay a 100-ft tape across the section to be measured. For bars, the starting point (zero mark) of the tape begins at the water edge, and extends away from the wetted channel perpendicular to the direction of flow. For channels, the line extends from

wetted edge to wetted edge. At approximately 1-ft intervals, a piece of sediment is selected by the data collector and the intermediate (or "B") axis is measured and recorded. The intermediate axis can be thought of as the limiting width that prevents the particle from fitting through a square grid opening. The pebble is chosen by averting one's eyes from the ground, then leaning down with a pencil. The first rock hit by the pencil is measured, often using a gravelometer at one-half phi sizes. The measurement is recorded by another person and the process repeated. At least 100 data points are collected along each pebble count line. If the particle is embedded (cannot be removed from bed due to large size and buried by finer sediment) or exposed bedrock, the exposed diameter should be measured and noted that it was embedded.

If the bar or channel being sampled is smaller than 100-ft in width or edge effects (vegetation) begin to occur along the outer edges, the pebble count can be collected in smaller adjacent lines or in a grid. For the adjacent lines, two 50-ft or four 25-ft lines are placed and the same method as described above is used. At least 100 total data points are collected.

For the grid method, a rectangular area is staked that is usually between 30 and 50 feet on a side. Two tape measures are used to form a cross in the middle. Each side of the rectangular grid is divided into 10 equal segments forming 100 grid cells within the rectangular area. An example photograph of a sample area is given in Figure 1. One pebble is counted within each grid cell totaling 100 measurements. The grid method is often preferred for exposed bars because the pebble counts can be performed in the same vicinity as a sieve sample and are most representative of the material being collected in the sieve sample. Once the 100 measurements are collected, the data is tabulated into bins of sediment sizes and a particle size gradation curve is computed (see Table 1).

Figure 8. Photograph of grid used to collect Wolman pebble count at Elder Creek.

Table 3. Sample output of pebble count measurement data.

SK-P-01

Particle Size	Particle Count
2	0
2.8	1
4	2
5.6	3
8	2
11.3	3
16	8
22.6	13
32	20
45	15
64	15
90	21
128	15
180	3
190	1
Total	122
Grain Size Summary	
D16	16.2 mm
D35	27.2 mm
D50	39.3 mm
D84	89.2 mm
D95	121.8 mm

SK-P-02

Particle Size	Particle Count
2	0
2.8	3
4	9
5.6	8
8	6
11.3	4
16	14
22.6	17
32	20
45	10
64	11
90	7
128	5
180	0
256	0
Total	114
Grain Size Summary	
D16	5.2 mm
D35	14.5 mm
D50	20.8 mm
D84	52.4 mm
D95	87.0 mm

SK-S-02

Particle Size	Particle Count
2	0
2.8	0
4	1
5.6	2
8	2
11.3	8
16	23
22.6	17
32	31
45	16
64	7
90	0
128	0
180	0
256	0
Total	107
Grain Size Summary	
D16	12.0 mm
D35	16.5 mm
D50	22.7 mm
D84	36.3 mm
D95	48.9 mm

SK-P-03

Particle Size	Particle Count
2	2
2.8	1
4	1
5.6	5
	7
11.3	11
16	19
22.6	17
32	19
45	22
64	12
90	4
128	0
180	0
256	0
Total	120
Grain Size Summary	
D16	8.8 mm
D35	14.9 mm
D50	21.3 mm
D84	42.8 mm
D95	60.4 mm

The sample size (number of pebbles measured) affects the accuracy of the sample. Bunte and Abt (2001) developed a chart to estimate the error in estimating the mean particle

size for a given sample size (Figure 2). The standard deviation (s_p) of the logarithms of the diameters of the sediment population can be estimated as:

$$s_p = 0.5\left|\phi_{84} - \phi_{16}\right|$$

where $\phi = -\log_2 d$, and d is particle diameter in mm. For example, if a sample had a d_{16} of 18.5 mm and d_{84} of 88 mm, s_p would be 1.1. If the sample size used in Wolman pebble count was 100, the standard error about the mean is approximately 0.2 ϕ units (Figure 2). If the true population mean diameter was 60 mm, the mean ϕ would be -5.91. An error of ±0.2 ϕ means that in 95% of all samples the sampled mean can be expected to be within the ϕ range of -5.71 to -6.11. This corresponds to a range of diameters of 52 to 69 mm.

Fig. 5.2: Relation between sample size n and absolute error $e_{\pm\phi m}$ around the mean in ϕ-units based on Student's t-values for approximately normal distributed bed material for various sorting coefficients, and α =0.05. Sample size n was computed iteratively to adjust for the variability of $t_{0.975,\,n\text{-}1}$ with sample size.
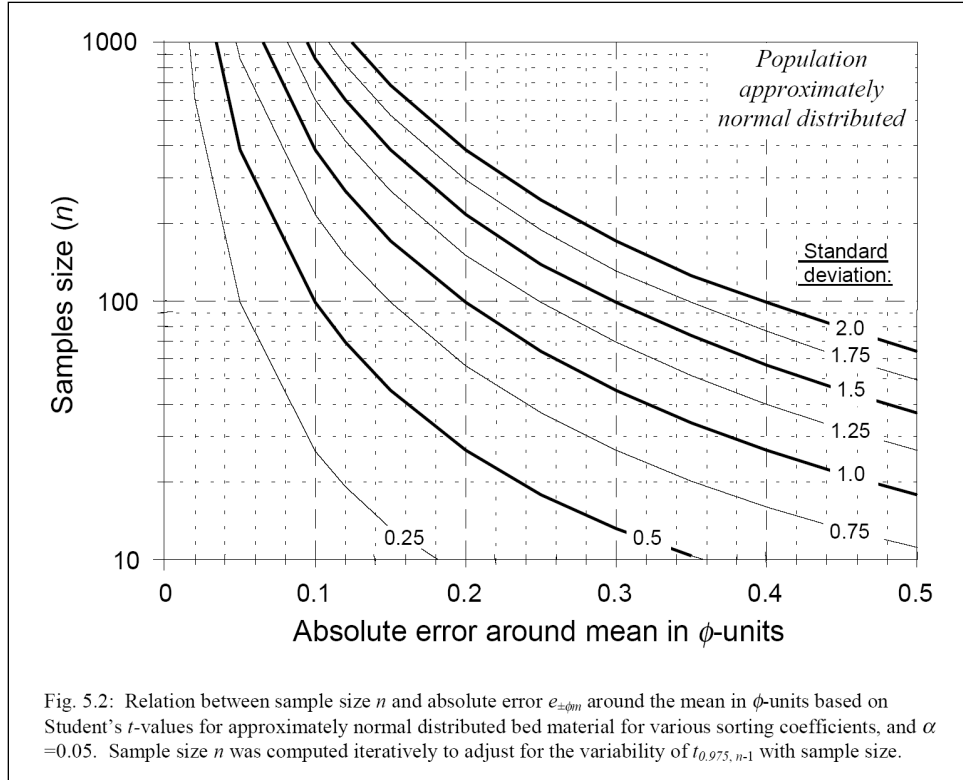
Figure 9. Figure from Bunte and Abt (2001) used to estimate error in estimating mean particle size for a given pebble count sample size. Assumes sediment gradation is normally distributed.

1.2. Volumetric Sieve Sampling Method

Sieving is a robust method that accounts for all sediment sizes present in a given sample site. Sediment data collection for sieving involves two samples per site: a surface sample and a subsurface sample, unless there is no discernable difference between the two layers. When the two layers can be distinguished, often the surface layer is coarser because as the flood hydrograph recedes the ability to mobilize the coarser sediment reduces and only the finer sized sediments are winnowed away.

Results of volumetric sieve samples provide the greatest amount of information regarding the sediment present in a sample with the highest accuracy. Assuming the location has been selected to adequately represent the material within a specific reach, volumetric

sieving can be used (1) to determine the extent of armoring within a reach, (2) for use as input into scour equations, and (3) for use as input into sediment transport models for understanding how surface and subsurface sediments move through a reach and could potentially impact structural stability. Unlike surface sampling techniques, volumetric sampling allows for determination of sediment variation in the vertical direction. Higher risk projects with a greater need to understand sediment transport conditions and potential sedimentation issues through rock structures should obtain volumetric sieve samples. However, this method is time and labor intensive. When combined with pebble counts and photogrammetric sieving, the number of volumetric sieve samples that can be collected by one 3-person crew is approximately 3 or 4 per day, depending on site access. Furthermore, the samples carried off site must be sent to a laboratory for sieve analysis at costs ranging from $100 to $300 per sample.

1.2.1. Procedure

Different sampling procedures are used to collect the bulk samples depending on whether the sample area is above or below the water. If the sample area is dry, a square area approximately 3-ft by 3-ft is typically used; this is the same sample size used for the photo documentation (Figure 9). A tarp is placed adjacent to the sample site. The surface layer is collected and placed on the tarp for analysis. For coarse gravel and cobble-sized sediment, typically one layer of sediment is removed by hand to constitute the surface sample. In areas of more uniform sediment sizes, a shovel is used to remove the surface layer to a depth approximately equal to the largest grain size of the surface sample. After the surface sample is collected, a subsurface sample is collected from within the same area. The depth of the subsurface should also have a depth approximately equal to the largest grain size diameter from the surface sample.

To limit the volume of sediment transported to the lab, the sample is typically passed through larger sieve sizes in the field. Large particles (cobbles) are measured individually, and then weighed on a scale supported by a survey tripod (typically a 70 lb scale with 0.1 lb increments). Smaller particles are then sieved. The sediment remaining on each sieve is then measured and recorded with the sieve opening size. The USBR Sedimentation and River Hydraulics Group typically field-sieves material larger than 32 mm and transports all remaining material to the lab for additional sieving. However, the total volume that one may want to field sieve will depend on the size of the material and the limitations in removing the sample from the site.

Multiple methods can effectively be employed in field sieving, including the use of large field sieves of varying diameters or rocker sieves of one or two diameters combined with the use of gravelometers. Ideally, sediment retained on a 32 mm sieve is graded to half phi size classes (e.g. corresponding to the diameter in millimeters of 32 mm, 45.3 mm, 64 mm, 90.5 mm, etc.). This can be accomplished by sorting and weighing material passing size classes on the gravelometer or using field sieves. If necessary, sediment greater than 32 mm can be graded to full phi sizes (e.g. corresponding to the diameter in millimeters of 32 mm, 64 mm, 128 mm, etc.), and the half phi sizes can be interpolated from the final gradations graphs. The remaining material (smaller than 32 mm) can be transported to a

lab and sieved according to the size chart presented in Table 2 . This process is completed for the surface sample and then repeated for the subsurface sample.

When necessary, water can be used to wash finer sized sediment through the sieves. The amount remaining on each sieve is then put into a bucket with holes in the bottom that is suspended from the scale and the weight recorded. All the water and sediment passing through the sieves is collected in a five gallon bucket that is allowed to spill over into a large plastic tub. After the field sieving is complete, the water is carefully decanted from the top of the bucket and the plastic tub. The sediment remaining in the bucket and tub is put into plastic bags and sent to a lab for sieving. Silts and clays that did not settle in the bucket or tub may be lost with this method. Pictures of the field sieving equipment and setups are shown in Figure 3 through Figure 5.

Table 4. Sieves most closely corresponding to size classes in the Wentworth Scale. All material < 32 mm are typically taken to the laboratory and sieved according to these size classes.

Standard Sieve Size	Corresponding Diameter (mm)
1.25"	32
5/8"	16
5/16"	8
No.5	4
No.10	2
No.18	1
No.35	0.5
No.60	0.25
No.120	0.125
Pan	

Figure 10. Rocker sieve used for on-site sieving.

Figure 11. Field setup of the rocker sieve methodology.

Figure 12. Alternative field sieving setup involving large sieves.

If the sample area is under water, a 30 inch diameter corrugated pipe can be placed in the river, or a McNeil core sampler may be used. For the pipe method, the sample is collected within the still water created by the walls of the corrugated pipe. The surface layer is then carefully removed by hand. Because the sample is collected underwater, more sands, silts, and clays may be lost as the sample is lifted through the water.

The total weight of the bulk sample should be related to the maximum particle size in the sample (Bunte and Abt, 2001; Scott and Haschenburger, 2005). Figure 6 shows recommended sample weights given the maximum diameter of the sample. The percentage lines represent the percentage of sample mass contained in one Dmax particle. It is recommended that the total sample weight correspond to the 1% line where possible for high precision, and not less than the 5% line for normal precision. If the sample area (within the rectangle) does not provide sufficient sample material to meet the minimum weight requirements, the initial rectangle may be expanded or additional samples on the same bar may be combined. This guideline is often difficult to abide by in larger streams due to time and budget limitations, but should be utilized to the maximum extent possible.

Data are processed to determine particle size gradations. Figure 7 provides a sample plot of surface grain size distributions results from volumetric sieve sampling. Figure 8 provides an example of the D84, D50, and D16 for the surface sample compared against the D50 from a pebble count as a function of river mile so various reaches of river can be relatively compared.
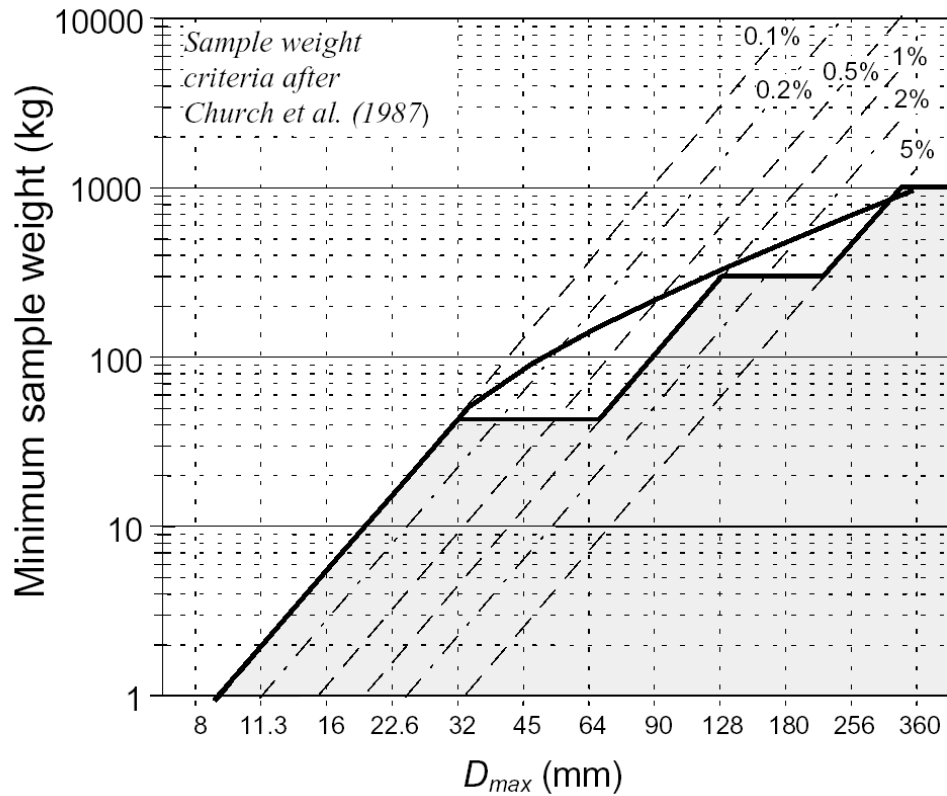
Fig. 4.20: Minimum sample weight for sediment with different D_{max} sizes (D_{max} = 0.1% m for a D_{max} <32 mm, D_{max} = 1% m for a D_{max} <128 mm, and D_{max} = 5% m for D_{max} > 128 mm) (after Church et al. 1987). The thick line represents a linear regression function fitted through the "corner points" of the stair-case function derived from the three sample-mass criteria by Church et al. (1987).

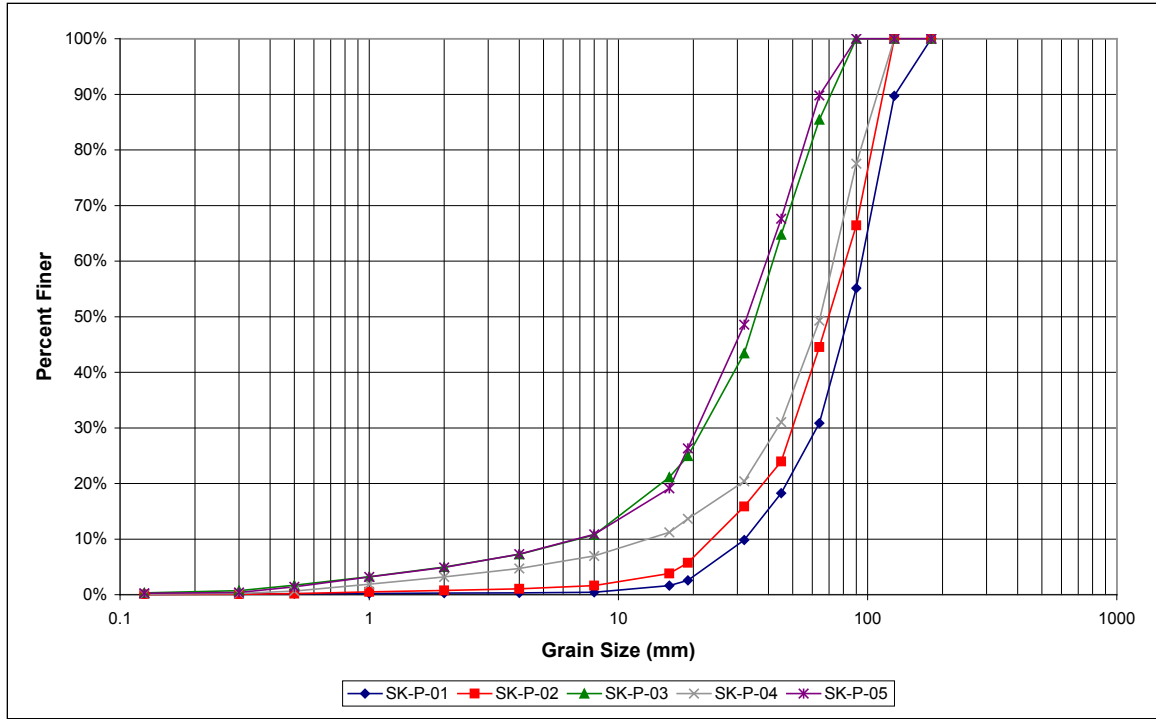**Figure 13. Figure taken from Bunte and Abt, 2001, summarizing recommendations from Church et al., 1987.**

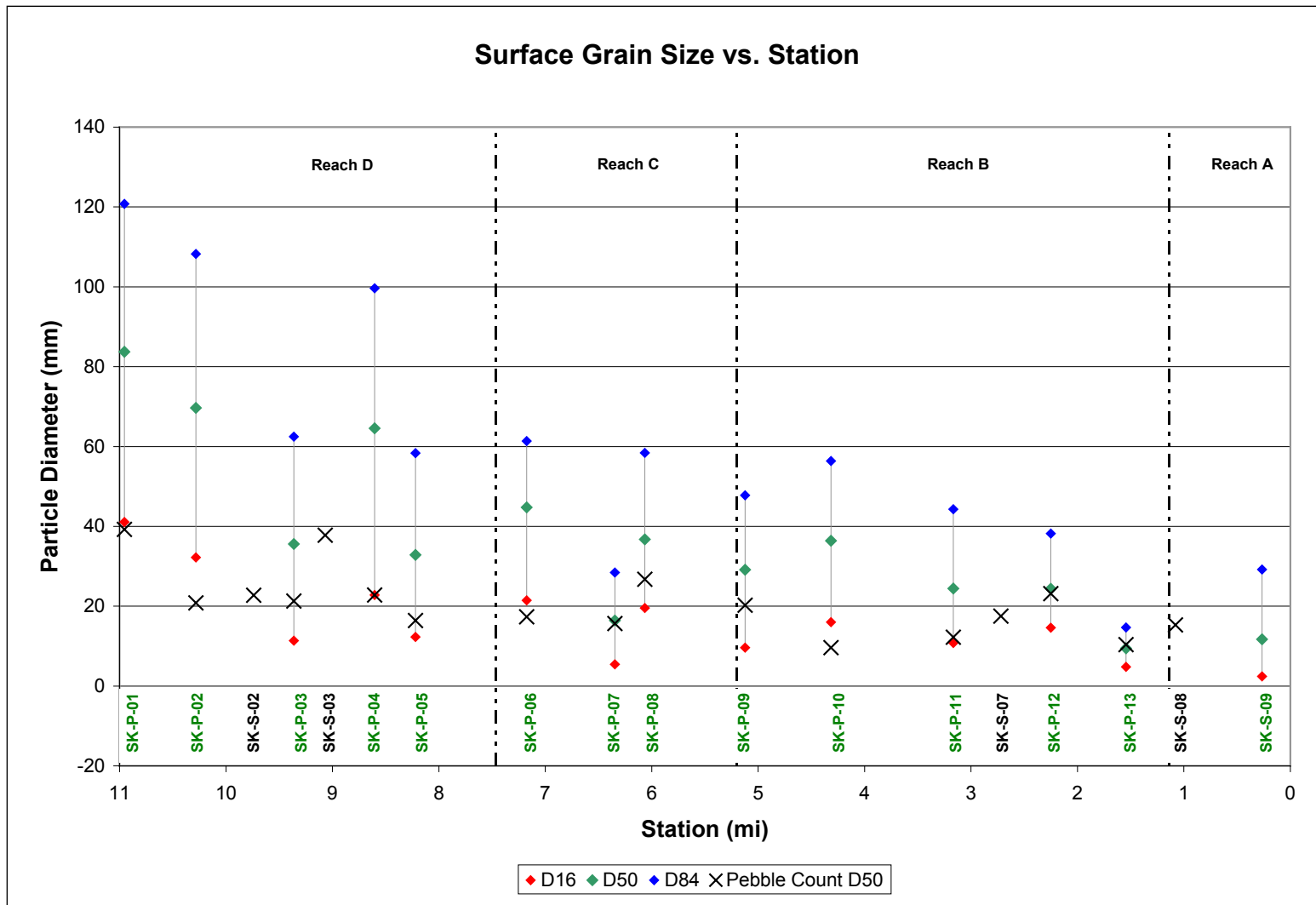**Figure 14. Example of particle size distributions for collected surface samples.**

Figure 15. Example of surface grain size as a function of river mile.

1.3. Photogrammetric Method

Photogrammetric sediment sampling offers a unique opportunity to sample a greater spatial area at a lower cost and in a faster manner than traditional means. Additionally, the digital image processing removes biases and errors related to human participation that are present in traditional pebble count methods. Bias leads to a lack of confidence in study results and makes management decisions difficult and unclear. Recent advancements in processing of photogrammetric sampling suggests that digital imaging of sediment samples may be able to perform with precision equivalent to a pebble count in less than one sixth of the time (Graham et al., 2005). While the advancements in photogrammetry appear to have some application across all aspects of sediment assessment, the most promising appears to be related to evaluating changes in sediment gradations resulting from habitat restoration actions.

Photogrammetric sampling allows several samples to be quickly documented in the field for post-processing in the office. However, post-processing in the office requires edge-detection software, which has been successfully applied to gravel and cobble bed materials, or pixel intensity software, which has been successfully applied to sand bed materials (and is not discussed in detail here). Edge detection software can range in price from $2,000 to $6,000 per license, the more expensive of which appears to have advanced manual controls, allowing the user to correct edges that have been misinterpreted by the software. Post-processing of each photograph may take anywhere from 10 minutes to 1 hour in the office, depending on discrepancies in the edge detection due to shadows, pock marks in the sediment, angularity of the material, and familiarity with the software. Although research is currently underway to evaluate the use of edge-detection photosieving for underwater samples and fine sediments, the technique appears to be most useful when applied to exposed, unvegetated, dry sediments such as those on bars.

While the photosieving technique has been effective in determining size distributions of surface sediments, subsurface sediment distributions are not easily obtained using this method. Size-gradations based on photogrammetric sampling is generally limited to answering questions related to how sediment gradations change over time (i.e. monitoring) and how size gradations vary throughout a specific reach of interest. Results from photosieving could be used as input to scour or sediment transport equations, but is less useful for modeling system and reach-scale transport of surface and subsurface materials.

1.3.1. Procedure

Typically, a known scale is used to mark off a rectangular grid of the sample site. This usually consists of two folding 72 inch (or 2 meter) rulers, folded in half and turned perpendicular to each other to create a 3 ft by 3 ft rectangle (Figure 9). This same area can later be used for the volumetric sieve sampling. A photograph is taken looking straight down on the surface sediment. It is helpful to shade the area being photographed with a tarp so there are no shadowing effects from rocks onto other rocks and from trees or the photographer onto the rocks. Shadows can impact the accuracies of post-processing techniques. While taking the photograph, the camera should remain perpendicular to the sample and be taken at a height that allows for the entire

sample area to be covered in one photograph. A similar photograph can be taken of the underlying layer if desired.

Image-processing procedures will include: correction for radial lens distortion and for the camera axis not being perpendicular to the surface, identification of the grains within a selected region, calibration of the grain sizes according to the scale within the photograph, and finally measurement of the selected grains resulting in a user-defined grain size distribution. This is accomplished through the use of advance photo imaging software.

Figure 16. Rectangular area defined for photogrammetric and volumetric sieve sampling.

1.4.   Site Location Guidelines

Aerial photography can be used to select initial sediment sampling sites. However, the final selection of the sampling site will likely need to be adjusted by the field team. For the purposes of analyzing sediment transport and channel morphology at a reach scale, sediment measurement sites are typically chosen using the following guidelines that may be helpful for the field crew to consider:

1. Locate sites within the unvegetated active channel to represent the portion of channel where the majority of coarse sediment is being transported and stored (e.g. not in overbank floodplain areas where only suspended sediment or fines are being transported and deposited).

2. Do not include sediment from the banks along the active channel. While bank sediment may represent a sediment source in areas of erosion, it may not be representative of the average particle distribution of sediment being mobilized by the river (one exception is in trying to document sediment sizes of point sources being contributed to river).
3. Site should be representative of typical channel conditions and sediment sizes mobilized by the river during bankfull and higher flows.
4. Areas with localized impacts to flow should be avoided, such as riprap, a backwater area, or a scour hole upstream of a large piece of woody debris or boulder.
5. Choose a geomorphic feature and consistently locate sites in a similar setting whenever possible so they can be relatively compared between sections of the river (e.g. middle of sediment bar, upstream end of bar, riffles, glides, etc).
6. Generally sampling is conducted under low flow conditions on actively mobilized (unvegetated) dry sediment bars because they typically represent the material sizes moving through the river reach.
7. To represent the sediment sizes that are being contributed through fluvial processes from tributaries (not debris flows), the sample site should be relatively close to the confluence, but upstream of any backwater effects from the main river channel.
8. If both pebble counts and volumetric sieve sampling are performed, sites should be located in close proximity to be comparable.

Locations of samples should be documented with a data collection form such as shown in Table 3. For each sample location, a GPS position should be recorded and ground photographs taken looking at the site and surrounding area. If aerial photography is available, the location of the sample should also be marked on the photo to validate the GPS point or document the location in areas of poor satellite coverage. It is also helpful to note if bedrock is present in the channel or banks, and if there are any significant deposits of large woody material (LWM) or boulders.

Table 5. Example sample data site documentation form.

Site Name (River mile and reach)	Data Collection Team	Field Date	GPS Location	Photo Numbers	Wet or Dry Sample Site	Site Descriptor (Point Bar, Longitudinal Bar, Riffle, Glide)	Notes about site (human features, LWM, bedrock, embeddedness, boulders, etc)

1.5. References

Bunte, K and S.R. Abt (2001). Sampling Surface and Subsurface Particle-Size Distributions in Wadable Gravel- and Cobble-Bed Streams for Analyses in Sediment Transport, Hydraulics, and Streambed Monitoring, General Technical Report RMRS-GTR-74, USDA, Rocky Mountain Research Station, Fort Collins, CO.

Church, M., McLean, D.G., and J.F. Walcott (1987). River bed gravels: sampling and analysis, In: Sediment Transport in Gravel-Bed Rivers, Thorne, C.R., Bathurst, J.C., and R.D. Hey (eds.), John Wiley and Sons, Chichester, p. 291-338.

Graham, D. J., Rice, S.P., and I. Reid (2005). A transferable method for the automated grain sizing of river gravels, *Water Resour. Res*., 41.

Scott, D.J. and J.K. Haschenburger (2005). Using the hyperbolic distribution to estimate percentile precision of fluvial gravels, *Computers & Geosciences*, 31, 1224-1233.