TECHNICAL SERVICE CENTER
DENVER, COLORADO

UPPER GILA RIVER FLUVIAL GEOMORPHOLOGY STUDY

FINAL REPORT
ARIZONA

US Department of the Interior
Bureau of Reclamation

● AUGUST 5, 2004 ●

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GRAHAM COUNTY, ARIZONA

COST SHARE AGREEMENT 00-GI 32-0054

Graham County, Arizona, and Reclamation are Cost Share Partners in the Upper Gila River Fluvial Geomorphology Study. The views or findings of Reclamation presented in this deliverable do not necessarily represent those of Graham County.

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ARIZONA

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The Fluvial Hydraulics & Geomorphology Team from the Technical Service Center is leading the Upper Gila Fluvial Geomorphology Study. The team consists of geomorphologists, engineers, and biologists. The members have expertise in water resources management, fluvial geomorphology, paleohydrology, hydraulics, and sedimentation.

The team members are:

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INTRODUCTION

This report finalizes the Upper Gila River Fluvial Geomorphology Study. In addition to summarizing the other study reports and findings, this report provides conceptual level recommendations for demonstration projects. The purpose of the projects is to demonstrate techniques for managing the river that take into account the causes of the geomorphic processes that dominate the fluvial system. This report also contains recommendations for a general-purpose monitoring program to accompany demonstration projects.

The other study reports are:

1. Background Information – Arizona
2. Field Data Collection Plan – Arizona
3. Catalog of Historical Changes – Arizona
4. Flood Frequency and Flow Duration Analyses – Arizona
5. Stable Channel Analysis – Arizona
6. Geomorphic Map – Arizona
7. Geomorphic Analysis – Arizona
8. Stream Corridor Assessment – Arizona

These reports in Adobe Acrobat format and other supporting information are stored on the CD's in the folder in the rear of this report.

The Stream Corridor Assessment synthesizes findings of the Background Information report, Catalog of Historical Changes, Flood Frequency and Flow Duration Analyses report, Geomorphic Map, Geomorphic Analysis, and Stable Channel Analysis. Combined, these studies provide a framework for understanding the physical processes that shape the Gila River upstream of the San Carlos Reservation.

The Background Information report is an annotated bibliography of the fluvial geomorphology of the Upper Gila River. The Catalog of Historical Changes traces changes in the Gila River plan form from 1935 to 2000. Flood Frequency and Flow Duration Analyses analyze historical stream flow and rainfall data for trends. The Geomorphic Map and Geomorphic Analysis analyze the fluvial geomorphic changes in the river and determine causative factors for the changes. The Geomorphic Map and Geomorphic Analysis also document major historical geomorphic change along the river primarily related to the construction and subsequent failure of levees, the construction of diversion dams, bridges, and to a lesser degree, the influence of native and invasive riparian vegetation. The Stable Channel Analysis forms a quantitative basis for understanding Gila River sediment transport and channel stability. When combined, these studies cover historical changes in river plan form, historical trends in hydrology, historical and pre-historical sediment flux from the upstream drainage basin, the causes of major historical geomorphic change along the river, and channel stability and sediment transport.

STUDY AREA & REACHES

The downstream limit of the study area is the San Carlos Reservation. The upstream boundary of the study is the Arizona-New Mexico State line. Figure 1 shows the study area and several landmarks, tributaries, towns, and highways. The analyses exclude the Gila Box area.

The length of river channel in the study area, including the Gila Box, is roughly 102 miles. There are two primary reaches in the study area under analysis, an upper and lower reach, separated by the Gila Box. The upper reach includes the river reach between the Highway 191 Bridge and the New Mexico State line. The lower reach includes the river reach between the downstream end of the Gila Box, near the Brown Canal diversion, and the San Carlos Reservation. Some of the analyses in this study further divided these primary reaches into sub reaches.

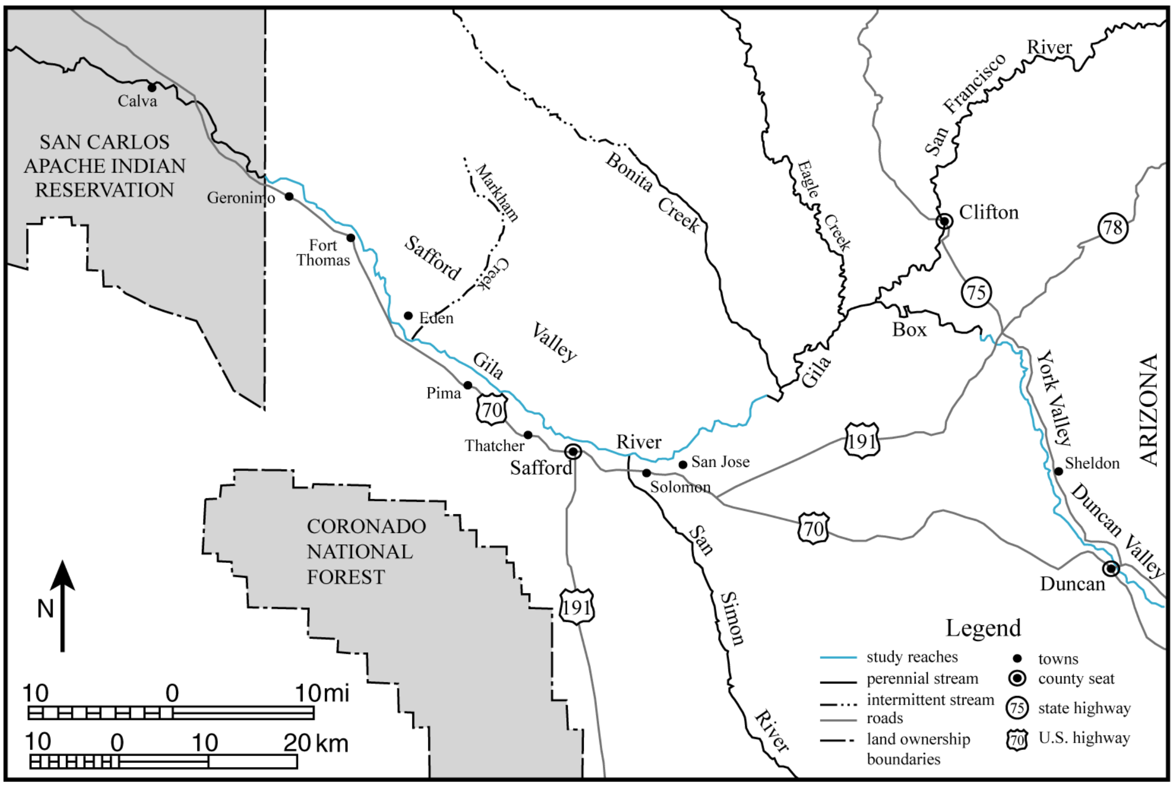

Figure 1. Study area between the San Carlos Reservation and the State of New Mexico.

CONCLUSIONS OF STUDY REPORTS & ANALYSES

This section presents the conclusions of the preceding study reports, including:

- Catalog of Historical Changes – Arizona
- Flood Frequency and Flow Duration Analyses – Arizona
- Geomorphic Analysis – Arizona
- Stable Channel Analysis – Arizona

In addition, this report presents the Arizona Geomorphic Map and a summary of the Arizona Background report.

BACKGROUND INFORMATION – ARIZONA

This document reviews existing studies that contain information that may be useful in the present study of the Upper Gila River. The references include, but are not limited to, hydrologic and geologic data, accounts of floods and precipitation events, studies of channel change and erosion, sedimentation in San Carlos Reservoir, water resources documents, scour studies of bridges on the Gila River, links between flood records and climate, floods and vegetation, land use planning, water quality, and ground water. The document is in two parts: (1) an annotated bibliography that summarizes references that may be pertinent to the present study, and (2) a bibliography of related references that include water quality data, hydrogeological data, fisheries studies, vegetation studies, soils data, and other miscellaneous information that is helpful for background information. This document is subject to amendment as other references become available during the course of the study.

GEOMORPHIC MAP – ARIZONA

A geomorphic map portrays surficial features or landforms that record geologic processes on the earth's surface. In fluvial geomorphology, these processes include erosion and deposition of sediment. Geomorphic landforms such as stream terraces and alluvial fans record sedimentary processes in a river system and are the basis for the delineations on the Geomorphic Map. For the Upper Gila River Fluvial Geomorphology Study, the Geomorphic Map illustrates geomorphic features that will aid in understanding recent channel changes of the Gila River.

The objective of the geomorphic map is to provide a picture of long-term river behavior in the Safford Valley and the Duncan Valley. Understanding long-term river behavior is useful for providing a comprehensive picture of river processes, placing recent channel changes into a long-term context, identifying causes of channel change and property loss in the historical period, and defining the extent of channel migration. The maps present basic geomorphic data on black and white orthophotographs. The Geomorphic Map, along with the Catalog of Historical Changes (Task 7C), fieldwork, and laboratory analyses, are combined in the Geomorphic Analysis (Task 10), a compilation of all geomorphic data developed in the Upper Gila River Fluvial Geomorphology Study.

The emphasis in this task was on defining the extent of lateral channel migration and assessing channel change. Geomorphic features that provide information on lateral migration and channel change include flood-modified surfaces, bedrock, alluvial fans, and older floodplain surfaces. Infrastructure is also a major factor in channel position and behavior of the Upper Gila River (Klawon, 2001). Thus, the maps include levees, diversion dams, and bridges.

The Geomorphic Map combines aerial photo interpretation, field mapping of geomorphic features, soil/stratigraphic descriptions, laboratory analyses, and use of previously published soil surveys to provide a long-term picture of river behavior. The maps utilize 1:4800 scale digital orthophotographs and display geomorphic features and infrastructure important in the recent lateral movement of the Gila River channel.

CATALOG OF HISTORICAL CHANGES – ARIZONA

The Catalog of Historical Changes documents changes in the alluvial channel of the Upper Gila River, Arizona from 1935 to 2000. The objective of the Catalog is to quantify variability in channel width during the historical period and identify reaches of high variability. Measurements of channel width made from historical aerial photography and qualitative observations of lateral migration provide the data necessary for an analysis of trends in channel behavior and lateral stability of river reaches.

CONCLUSIONS

General trends in channel changes from this study parallel those described by Burkham (1972). The early 1900's experienced several extreme floods, causing channel widening to 1935 (Burkham, 1972; Olmstead, 1919). This early information was gathered for Safford Valley and may or may not apply to Duncan Valley. From 1935 to the early 1960's, vegetation encroached on the channel, narrowing it. Levee, dike, and agricultural development also contributed to channel narrowing in this period. From the late 1960's to 2000, the channel widened in response to large floods. It is now roughly the same width, on average, as in 1935. In most cases, flood flow widths at specific channel locations are variable, but not unprecedented in the historical record.

This study has shown that although high variability exists in channel width and position in both Safford Valley and Duncan Valley, many channel positions are not new and channel widths are similar or smaller than 1935 channel widths for the Gila River during the period of study. In many of the case studies, the channel simply reoccupied old channel positions from earlier in the historical period. Average flood widths also show that by 2000, the river channel had reached an average flood width similar to the 1935 average flood width. Some channel changes; however, in recent decades do seem to be unprecedented in the period of study. Examples of such cases include the channel changes near Whitefield Wash, where erosion between 1992 and 1997 caused lateral migration of the left and right banks and greatly increased the sinuosity in the reach. Another dramatic area of channel change occurs downstream of the San Jose Diversion, where lateral movement of the channel toward the right bank has been observed on photograph years of 1981, 1992 and 1997.

The impact of floods on the Gila River channel is evident based corresponding large channel changes following flood years. In Duncan Valley, the most changes in flood width occurred following the 1978 flood and the floods in the 1990's. In Safford Valley, changes occurred following the 1972, 1983, and 1993 floods. The analysis of change using flood flow widths for Duncan Valley and Safford Valley show that Safford Valley has experienced many more perturbations in the period of study than Duncan Valley. This is shown best by the presence of several long, stable reaches in Duncan Valley, compared to a few short stable reaches in Safford Valley. Major channel changes generally occurred following large floods; this highlights the important point that the largest floods in the Gila River system have lasting effects that can be observed in channel morphology for decades following their occurrence.

FLOOD FREQUENCY AND FLOW DURATION ANALYSES – ARIZONA

This report summarizes flood frequency and flow duration for sites within the Gila River basin from the Arizona-New Mexico State line to San Carlos Reservation. These estimates were completed as part of Task 9 of the Upper Gila River Fluvial Geomorphology Study. The primary basis for the flood frequency and flow duration estimates are U.S. Geological Survey peak discharge and mean daily flow records.

The Upper Gila River basin is located in the southeast corner of Arizona and southwestern New Mexico. The area in Arizona is called the Central Highlands physiographic province. Within the study area, the river flows generally westward from its headwaters in the Gila Wilderness area in Grand County, New Mexico to the San Carlos Indian Reservation, Arizona. The main tributaries in New Mexico enter the Gila River upstream of Cliff, New Mexico. The major tributaries in Arizona upstream of Coolidge Dam are the San Francisco River, Eagle Creek, Bonita Creek, and the San Carlos River, which drain from the mountains on the north side of the basin, and the San Simon River, which drains from the south. Elevations in the drainage basin range from 5,650 feet at the western boundary of the study area (San Carlos Indian Reservation) to 11,000 feet in the mountains of the Gila Wilderness area (New Mexico).

The U.S. Geological Survey has published stream flow records from many gaging stations located in the Gila River basin upstream from San Carlos Reservoir into New Mexico (e.g., Pope et al., 1998). There are many active gaging stations in the Upper Gila River. This study focuses on using data from long-term gaging stations located on the Gila and San Francisco Rivers, specifically these five:

1. Gila River below Blue Creek near Virden, NM
2. Gila River near Clifton, AZ
3. San Francisco River at Clifton, AZ
4. Gila River at head of Safford Valley near Solomon, AZ
5. Gila River at Calva, AZ

Pope et al. (1998) presents a list of basin, flood, and climatic characteristics for these sites.

There are two main objectives of this study: (1) estimate flood peak frequencies; and (2) estimate flow durations at selected locations within the Upper Gila River basin, for application in subsequent fluvial geomorphic and hydraulic analyses.

CONCLUSIONS

Flooding in the Gila River basin is caused primarily by rains from fall and winter storm systems. These storms are generally cold frontal systems colliding with warm, moist air or tropical storms. Extreme flood-producing storms are widespread and generally cover the majority of the Upper Gila River basin. Instantaneous peak discharge data confirm that the largest-magnitude floods occur in the fall and winter and are predominately from rainfall. The largest floods have occurred in water years 1891, 1907, 1941, 1973, 1979, and 1984.

The log-Pearson Type III distribution was fit to annual peak discharge estimates at the five gaging stations using the Expected Moments Algorithm and available historical information. The results indicated that the distribution adequately fit the data. Peak discharge probability estimates indicate the 2-year flood ranges between 5,210 ft^3/s and 9,650 ft^3/s at the five locations. The 100-year flood ranges between 44,800 ft^3/s and 175,000 ft^3/s at the five locations.

A period-of-record Flow Duration Curve for the water year indicated that mean daily flows are typically less than about 1,000 ft^3/s for 90 percent of the time at all five sites. Mean daily flows for the November-April winter season are nearly always greater than the summer July-October season. Mean daily flows are zero about 10 percent of the time in the Gila River at Calva.

GEOMORPHIC ANALYSIS – ARIZONA

The Geomorphic Analysis synthesizes geomorphic information about the Gila River and compares results of the analysis to other tasks performed for the Upper Gila River Fluvial Geomorphology Study. The goal of the geomorphic analysis is to provide an understanding of the fluvial geomorphology and to explain recent geomorphic change on the Gila River in Safford and Duncan Valleys. Methods used for the Geomorphic Analysis include geomorphic mapping, soil descriptions and laboratory analysis. Soil maps developed by Poulson and Youngs (1938) and Poulson and Stromberg (1950) for Safford Valley and Duncan Valley, respectively, provided critical information for developing the Geomorphic Map. In addition to soil surveys, soil and stratigraphic characteristics were described for 30 sites with actively eroding banks along the Gila River in Duncan and Safford valleys. The delineation of the geomorphic features used this information, along with radiocarbon analysis, aerial photography, and soil surveys.

CONCLUSIONS

In Safford and Duncan Valleys, the most substantial geomorphic changes in the Gila River in recent decades are due to changes in the magnitude and frequency of annual peak floods, as well channel straightening and flood interaction with levees and diversion dams. Using soil/stratigraphic information and lab analyses, geomorphic mapping in these valleys indicates that the Gila River has migrated within the Pima Soil Boundary for the last several hundred years and within the Geomorphic Limit for at least the last 1,000 years. Areas of lateral change are indicated where historical floods have eroded banks that are mapped as part of the Geomorphic Limit or Pima Soil Boundary.

The majority of property loss has occurred in areas of young alluvium, which is part of the active channel migration zone. Within this zone, lateral migration is common and it is not unexpected for areas to be eroded during large floods. Several areas with unusual channel geometries and erosion of banks older than several hundred years are clues that other factors are important in creating the current (year 2000) channel morphology. The Catalog of Historical Changes and the Geomorphic Map reveal the close correlation between the construction of man-made features and subsequent property loss during large floods along the Gila River in Arizona. Human factors that cause lateral instability include levee encroachment into the flood or active channel, diversion dams, and channel straightening. Vegetation and alluvial fan development may also act as controls on channel position in these reaches. The Catalog of Historical Changes shows that the majority of erosion occurs during high flow events such as the flood of October 2-3, 1983, and that channel widening is a geomorphic response to large floods. The local factors mentioned above appear to cause minimal geomorphic change during low to moderate flows but are the catalysts of substantial geomorphic change during large floods of recent decades.

STABLE CHANNEL ANALYSIS – ARIZONA

This report presents an analysis of the stability of the Gila River between the San Carlos Reservation and the lower end of the Gila Box, and between the upper end of the Gila Box and the Arizona-New Mexico state line. Stability, in an alluvial channel, according to Mackin (1948), "occurs when, over a period of time, the slope is adjusted to provide, with available discharge and the prevailing channel characteristics, the velocity required to transport sediment supplied from the drainage basin." Lane (1953) defines alluvial stability as "an unlined earth channel which carries water, the banks and bed of which are not scoured objectionably by the moving water, and in which objectionable deposits of sediment do not occur." Chien (1955) contends that "…the equilibrium state of an alluvial channel is attained by adjusting the dimensions of the cross section and the slope of the channel to the natural conditions imposed on the channel by the drainage basin."

This analysis utilizes an analytical tool named RISAD, a module of SAM, developed by the US Army Corps of Engineers, to analyze the channel roughness, sediment transport, and discharge in four reaches of the Gila River in the study area. Input into RISAD includes hydraulics produced by the HEC-RAS backwater model, bed material gradation data gathered during the field data collection portion of the Upper Gila River Fluvial Geomorphology study, and hydrology analyzed for this report based upon US Geological Survey stream gaging data collected at several gaging stations in the study area. The analysis uses hydrological data from water years 1965-2000.

CONCLUSIONS

This analysis indicates that the results of the stable channel modeling are consistent with the geometry of the Gila River in the study area. The modeling indicates that the river is moderately unstable at the effective discharge in many sub-reaches, mostly in the area downstream of Safford and upstream of

Sheldon. The modeling shows that the river is stable in a few sub-reaches, mostly between York and Sheldon, possibly due to bed-rock controls in the area. The instability is greatest with respect to the width and sinuosity of the stream. In general, the channel has widened in response to an increase in the magnitude and frequency of floods since 1965. Without large floods in the future the channel will narrow and may locally aggrade, similar to the 1935-1965 period.

For the purpose of the stability analysis, the study reach was broken into four sub-reaches. Lower Reach 1 extends from the San Carlos Reservation upstream to Emery. Lower Reach 2 extends from the Fort Thomas low water crossing upstream opposite the Ashurst Cemetery. Lower Reach 3 extends from below the Eden Bridge upstream to the Dodge-Nevada canal diversion dam. Lower Reach 4 extends from the Graham canal diversion dam upstream to the San Jose canal diversion. The Upper Reach extends from below Sanders Wash, below Sheldon, upstream to the Arizona-New Mexico state line.

Lower Reaches 1 & 2

Model results show that Lower Reach 1 and Lower Reach 2 are relatively unstable. Some sections in Lower Reach 2 might be stable. The channel width in the Safford Valley is nearly the same as in 1935, the widest measured over the period of 1935-1997 (Klawon, 2001). Model results indicate that if the channel trends towards the minimum slope on the stable channel curve, Lower Reach 2 will experience the most channel narrowing. The process may include an increase in sinuosity causing widespread bank instability and retreat. Hypothetically, and separate from the stable channel analysis, a typical geomorphic response might include invasion of non-native vegetation, followed by bank encroachment and channel narrowing. The stable channel analysis indicates that Lower Reach 1 may be overly steep. If the channel reduces its slope by increasing sinuosity, bank instability and retreat will result. However, local observations indicate that the channel may be aggrading in the reach below Fort Thomas. More modeling and geomorphic investigation is necessary to determine the channel trends in this area.

Lower Reaches 3 & 4

Model results show that both Lower Reach 3 and Lower Reach 4 are relatively stable by virtue of the distribution of points about the stable channel curve. There has been significant lateral movement of the stream in several areas caused by both channel straightening projects, the hydraulic response to channel straightening projects, and the overall cycle of hydrologic regime since the mid 1960's. Lower Reach 3 may undergo the most channel narrowing following invasion by non-native vegetation resulting in bank encroachment.

Upper Reach

Model results show that most of the sections in the Upper Reach are in the degradational range of the stable channel plot. Geomorphic evidence indicates that the river is in a period of degradation following a period of aggradation. There are ample observations of that phenomenon in the Virden and Duncan areas. There are several bedrock areas and hydraulic controls that are not alluvial in nature, invalidating the stable channel analysis in those reaches.

STREAM CORRIDOR ASSESSMENT

The Stream Corridor Assessment synthesizes findings of the Background Information report, Catalog of Historical Changes, Flood Frequency and Flow Duration Analyses report, Geomorphic Map, Geomorphic Analysis, and Stable Channel Analysis. Combined, these studies provide a framework for understanding the physical processes that shape the Gila River upstream of the San Carlos Reservation.

CONCLUSION

Systemically, the Gila River active channel widens and narrows on a decadal time scale in response to changes in basin hydrology, sediment flux, and riparian vegetation life cycles, as well as other factors. The widening and narrowing process is partly a natural response to basin hydrology. However, encroachment into the active channel by agriculture and invasive riparian vegetation accelerates channel narrowing, while widening appears to be in response to increases in frequency and magnitude of annual peak flows. The combined analyses of this study indicate that, on a local basis, constriction of the channel by levee construction and subsequent failure of significant lengths of levee, transformation of the flood channel into arable land, and the installation and operation of diversion dams, are the probable causes for the most significant property losses during large floods along the Gila River in the study reach. The findings of these analyses do not suggest that there is a system-wide instability in the Gila River system due to changes in sediment flux from the upper basin.

PROPOSALS FOR RIVER MANAGEMENT AND DEMONSTRATION PROJECTS

This section presents conceptual ideas for potential demonstration projects for managing the land resources adjacent to the Gila River. Reclamation, in consultation with Graham County, the former San Carlos-Safford-Duncan Watershed Group, the Gila Monster, and their successors, as well as the Bureau of Land Management, have developed these concepts based upon the causal analysis and other analyses of this study, and the field review held in February 2004. These projects demonstrate means for achieving stakeholders' goals and objectives.

LEVEES

We find that the failure of levees along the Gila River during the largest floods of recent decades caused significant erosion of agricultural land. Levees constructed near Whitefield Wash, the Lunt property, and Geronimo, and other locations, caused a disconnection of the active channel from the flood plain. This increased the flood stage, resulting in the failure of levees and greater erosion of the flood plain behind the levees. We do not specify detailed demonstration projects regarding levees here. Instead, general recommendations are included to guide any future construction, removal, or realignment of levees. Setting back levees to widths that approximate the average flood channel width will allow the river channel to accommodate floods rather than forcing floods to overtop levees and cause extensive damage to agricultural land. Figure 2 shows the locations of historical channel width measurements. Table 1 lists average channel widths at the locations shown in Figure 2 during the historical period 1935 to 2000. Similar average flood channel widths defined reaches. The table includes average flood channel widths from 1935 and 2000 for the same reaches. Measurement points correspond to the fixed points listed in Appendix B of the Catalog of Historical Changes. These measurements were made across the width of the channel and perpendicular to the flow direction from each fixed point.

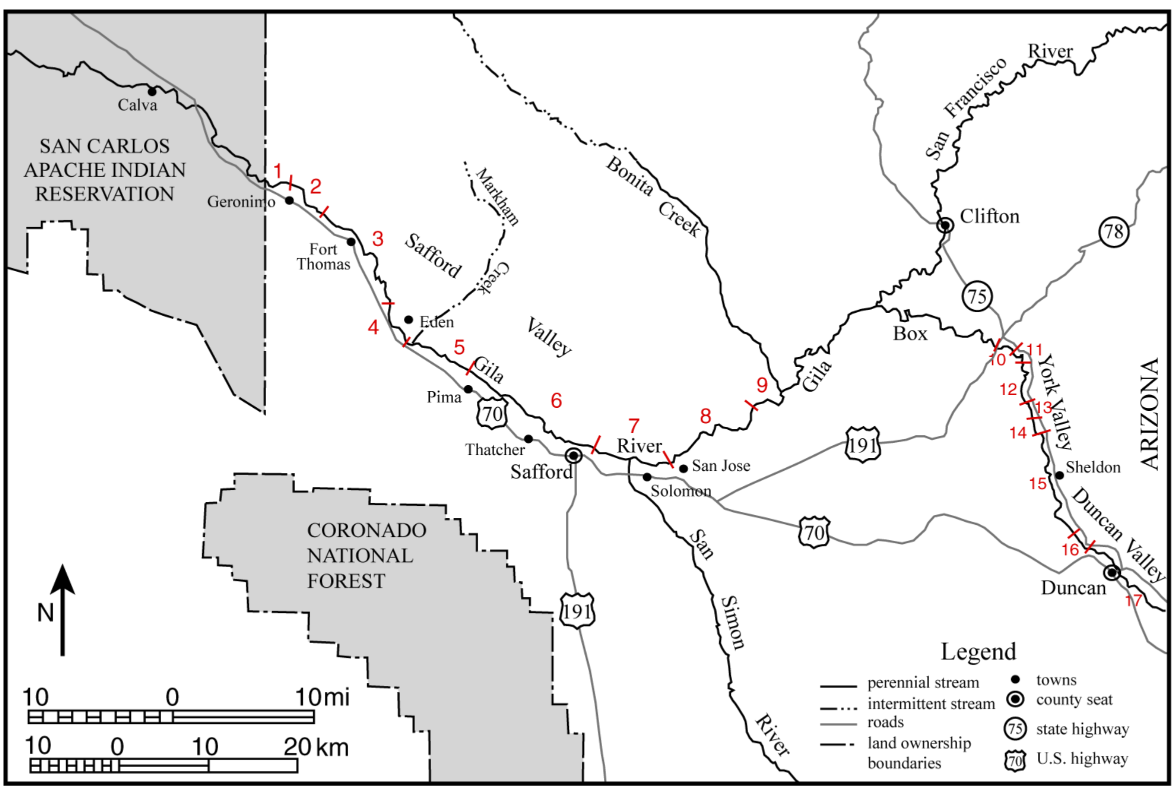

Figure 2. Reach boundaries for average channel width groupings. Red text corresponds to reach descriptions in Table 1.

Table 1. List of average flood channel widths between 1935 and 2000.

					Measurement Points
1	San Carlos Reservation to Geronimo	3200	4200	3700	1-2
2	Geronimo to Porter Wash	1700	1800	1300	3-7
3	Porter Wash to Eden North	2700	2800	2900	8-19
4	Eden North to Fort Thomas Diversion	1400	1900	1500	20-23
5	Fort Thomas Diversion to Peck Wash	2200	2000	2300	24-29
6	Peck Wash to Lone Star Wash	1700	2400	1600	30-44
7	Lone Star Wash to San Jose Wash	2100	1900	2100	45-53
8	San Jose Wash to Brown Diversion	1600	1600	1700	54-60
9	Brown Diversion to Head of Safford Valley	900	700	900	61-62
10	Route 191 Bridge to CA Bar Creek	400	700	500	63-65
11	CA Bar Creek to Cottonwood Creek	1800	2000	1500	66
12	Cottonwood Creek to Rocky John Canyon	900	700	1000	67-70
13	Rocky John Canyon to Apache Creek	300	300	300	71-73
14	Apache Creek to Kaywood Wash	1100	1400	1100	74
15	Kaywood Wash to Waters Wash	600	700	700	75-88
16	Waters Wash to Woods Canyon	1300	1200	1200	89
17	Woods Canyon to Arizona-New Mexico border	600	700	800	90-101

As discussed in the Catalog of Historical Changes, channel widths have historically been rather variable, with the greatest widths in 1935, followed by a period of decreasing channel widths during the 1940-60's due to few large floods and encroachment in the flood channel by vegetation, levees, and agriculture. Large floods in the 1970's, 1980's, and 1990's increased the width of the flood channel to the approximate width of the 1935 channel. This information indicates that although much erosion has taken place during the floods of recent decades, the average flood channel width at the present time is similar to the average width during the early part of the 20th century. As the Geomorphic Analysis describes in detail, the Gila alluvium marks the extent of channel migration for the past several hundred years. Information developed about the Gila alluvium suggests that the Gila River has been actively migrating, or eroding and depositing sediment, in this zone from approximately several hundred years ago to the present time. The Pima alluvium, or Pima soil, marks the geologic flood plain limit, where large floods from the Gila River may inundate the surface but have not in most cases exceeded its lateral extent for about the last 1,000 years.

From the hydraulic and geomorphic information developed in this study, general recommendations can be made regarding land management and the placement or removal of levees along the upper Gila River.

Land that is within the historical channel limits, or areas mapped as Gila alluvium on the Geomorphic Map, should be considered to have a high risk of erosion. It is likely that flood flows will inundate this land or that the Gila River channel will laterally migrate into these surfaces and cause substantial erosion. In some areas, this width is relatively wide while in others the width is narrow. This variability primarily reflects the natural variability in the width of the Gila River system, although in some areas where structures such as bridges have existed for the entirety or majority of the historical record, the channel width may be artificially narrow. For example, the close proximity of the Geomorphic Limit to the active channel on both sides of the river at Eden Bridge creates a natural constriction in the river. In contrast, Measurement Point 48 corresponds to an area just downstream of San Simon River that artificially constricted by earthen levees on the south side for much of the historical period (Figure 3). This reach serves to illustrate two recommended options for levee management along the Gila River:

Option 1-- setback levees to the average historical channel width of the corresponding reach in Table 1.

Option 2-- setback levees to the width of the Gila alluvium and allow floods to inundate farmlands that are located in the Gila alluvium. This setback would follow the Pima Soil Boundary.

Figure 3. Illustration of Options 1 and 2 downstream of the San Simon River.

In Figure 3, Option 1 shows the average historical channel width in Reach 7 (see Table 1) while Option 2 shows the extent required to setback levees to the width of the Gila alluvium. Figure 4, located upstream of Solomon Bridge, also demonstrates the two options. Option 1 would essentially involve maintaining the current flood channel as a corridor free of levees. Although Option 2 requires a significant portion of agricultural land to be without levee protection, we contend that many of the levees have done more harm than good in causing extensive erosion of farmlands when levees fail during large floods. The reach upstream of Solomon Bridge is a good reach to demonstrate these concepts without critical infra-structure that require flood control and protection. In other areas where critical infrastructure exists, there will obviously be a need to maintain levees within the flood channel as well as within the Gila alluvium. This point is particularly relevant in Safford Valley, where diversions, bridges and canals are important for maintaining livelihoods and transportation routes through the region. The reach down-stream of Smithville Diversion illustrates an area where levees are necessary to protect Smithville canal and Thatcher Bridge. Downstream of Thatcher Bridge, levee setbacks are a viable option (Figure 5).

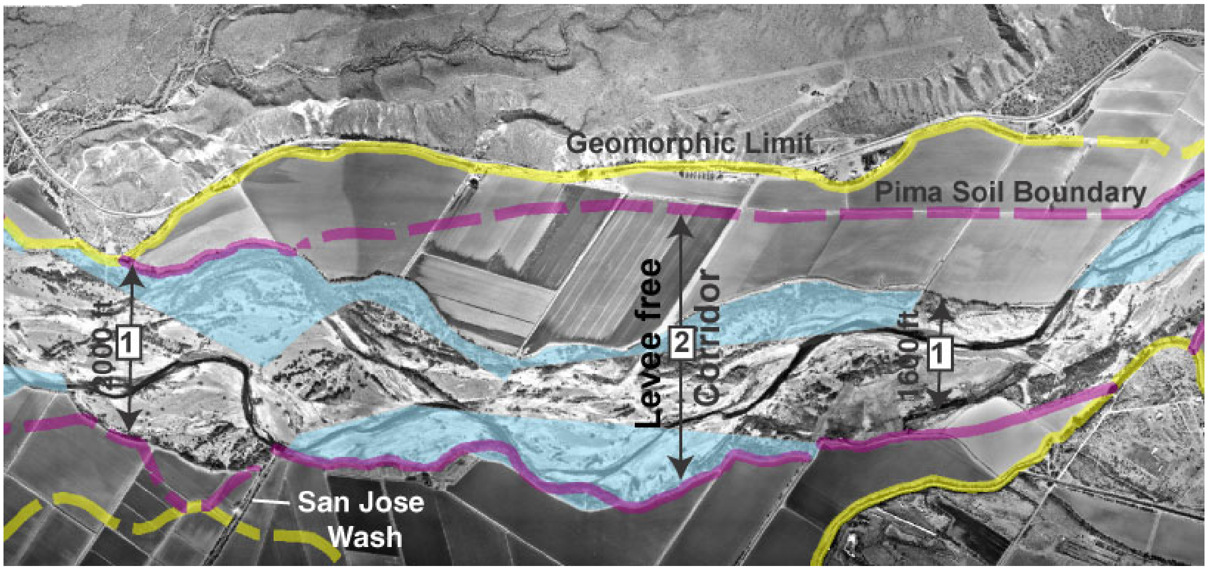

Figure 4. Illustration of Options 1 and 2 showing average historical channel widths for Option 1 and the levee free corridor for Option 2.

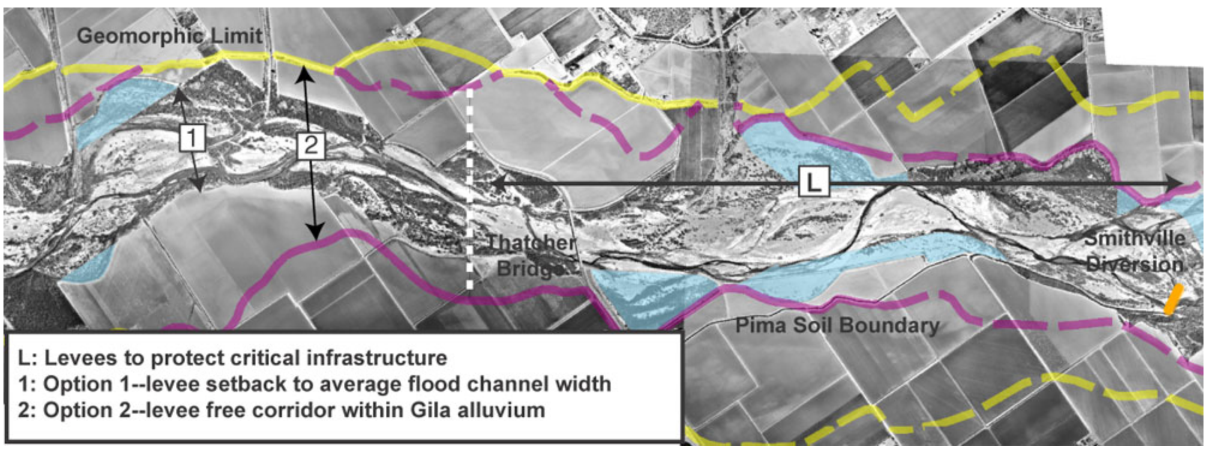

Figure 5. Illustration of land management options downstream of Smithville Diversion, where levees play an important role in flood protection for Thatcher Bridge and Smithville Canal.

In Duncan and York valleys, most of the levees constructed following the 1978 flood have since been eroded by more recent floods. Only remnants remain and do not appear to be problematic. In a few locations, the removal of levees would improve flow conveyance during large floods. Two areas in particular, near Kaywood Wash and the old bridge crossing north of Sheldon, would benefit from such a removal (Figure 6). In these areas, levee removal along the field edges and in between fields located in the Gila alluvium would allow the main channel to reattach to its former flood plain during floods. Levees along railroads would be important to retain for flood protection.

The Pima alluvium is considered to be the geologic floodplain of the Gila River that is inundated during large floods. Levees along this alluvium should remain relatively low so floods are less erosive if they do overtop the surfaces. It could be argued, in fact, that earthen levees do more damage than good on the Gila River, causing large amounts of property loss when they are compromised during large floods. If there were no levees along the Gila River, some of the lands within the Pima alluvium would be inundated but would remain relatively intact rather than lost to extensive erosion. This is an important point particularly in Duncan Valley where the Pima alluvium is closer to the active channel.

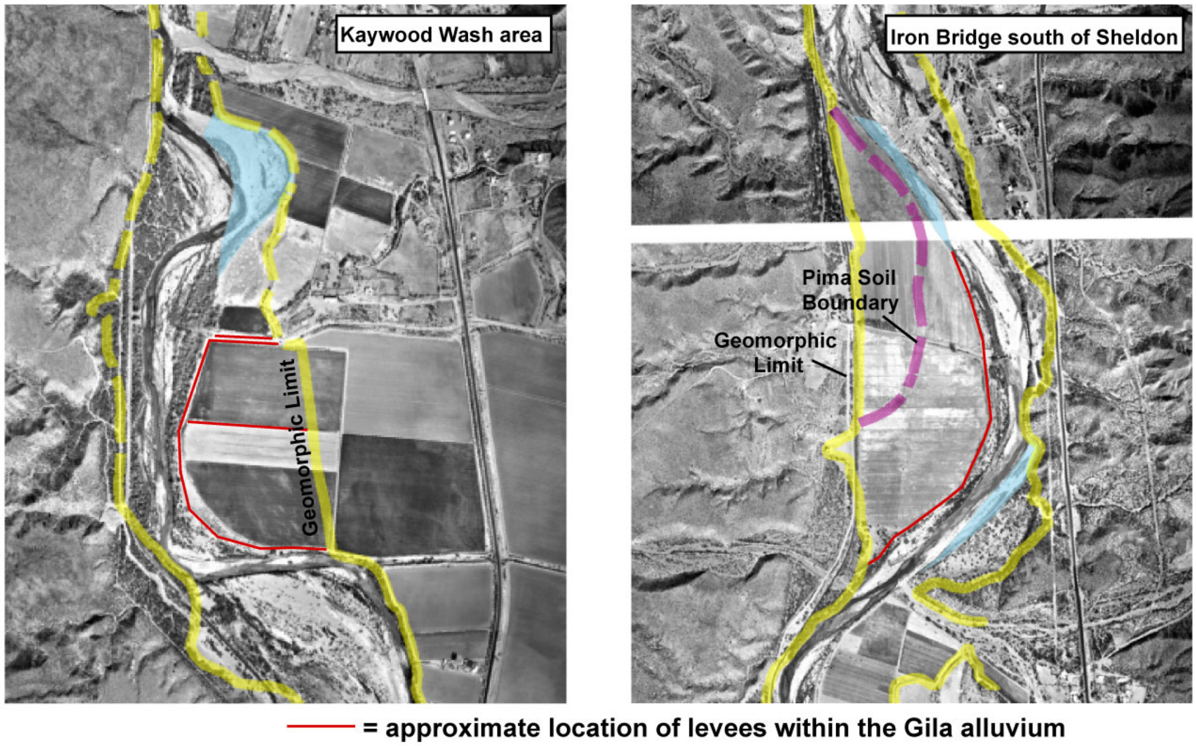

Figure 6. Levee removal recommendations near Kaywood Wash and the old bridge crossing south of Sheldon in Duncan and York Valleys.

Considering the system as a whole in the strategic removal or placement of levees is the most effective plan to reduce property loss and damage during floods along the Upper Gila River in Arizona. As the Catalog of Historical Changes, Geomorphic Map and Geomorphic Analysis show, structures emplaced along one reach may affect reaches both upstream and downstream from the new structure. These effects need to be considered so that similar property loss associated with levees is not repeated in the future. If Options 1 or 2 were to be applied to the Upper Gila River, we would recommend devising a system-wide plan for implementation.

Rivers such as the Upper Gila River are dynamic by nature, and are therefore likely to migrate laterally and change their course in response to large floods. It is thus possible that future property loss could occur despite levee setbacks and removal, especially in areas mapped as the Gila alluvium. However, future property loss should be less extensive than what has occurred in areas with many levees during the last few decades if the channel is allowed to remain at its current width (2000 A.D.) in order to accommodate large floods.

BRIDGES

Reclamation recognizes two issues associated with bridges, primarily in the Safford Valley. First, is the issue of access during floods. The left approach to the Solomon Bridge is readily inundated by relatively small floods, impairing access to this important transportation corridor. The second issue involves the relation of the Safford Valley bridges to the levees. As mentioned earlier, setback levees may be a reasonable choice for managing river flooding. Integrating the bridges into such a system is an important consideration of any levee scheme.

SOLOMON BRIDGE

Solomon Bridge provides access to both sides of the river for county residents and emergency services. It is the upstream bridge in the Safford Valley, and is therefore important to residents living east of Safford. The left abutment and approach to the bridge is a shallow swale that relatively small floods inundate, impairing access to the bridge. Figure 7 shows this approach following a recent flood that inundated the roadway. The left abutment of the bridge is in a hydraulically vulnerable location, the former outside bend of the river. This location it is subject to the brunt of flood flows. Several attempts have been made to remedy the situation, including riprap, channelization, and willow pole plantings.

Reclamation recommends a phased series of hydraulic remedies at Solomon Bridge. Figure 8 illustrates a series of seven potential actions that collectively are designed to assure bridge access and decrease the risk of transportation interruption during floods.

Figure 7. Left abutment and approach to Solomon Bridge. Note that the approach has recently been inundated (flow right to left) by floodwaters.

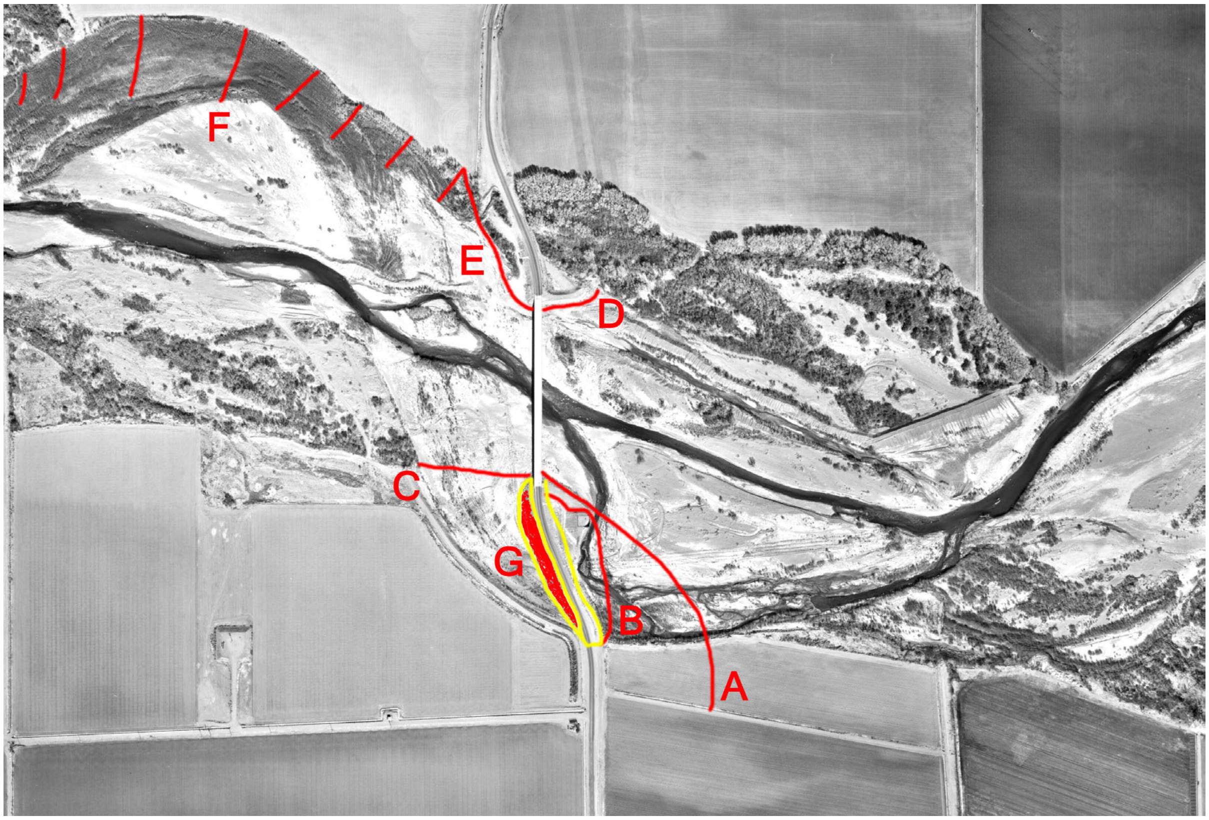

Figure 8. Potential demonstration project(s) at Solomon Bridge. (flow is right to left)

There are seven labels in Figure 8, next to two line colors, red and yellow. The red lines indicate the recommended locations of riprap berms, while the yellow lines indicate areas of fill. At location A, we propose a berm of properly sized riprap, extending from the left bridge abutment to a point well up on the floodplain. The purpose of this structure is to 'catch' the flow impinging on the left bridge approach at nearly a 90-degree angle, and direct it towards the bridge spans. We propose a second riprap berm at location B. This additional riprap should probably be at a higher elevation than the A line. It also begins at the left bridge abutment, wraps around the well structure, then ties into the flood plain near the roadway. The purpose of this riprap is to protect the upstream side of the filled roadway embankment.

Reclamation proposes raising the roadway alignment between the floodplain (location B) and the bridge deck. Furthermore, we propose armoring the downstream side of the roadway embankment with a riprap revetment at location G. Completing the left side of the river is a riprap berm at location C. This berm will contain flows exiting the bridge, reducing expansion losses, in effect 'pushing' the river downstream. This will prevent backflow that could undermine the downstream portion of the bridge abutment. We propose three treatments to the right side of the bridge. First, at location D, we propose enhancing the riprap that already protects the right bridge abutment. At location E we also propose armoring with riprap, again as at location C, to prevent back flow and to ensure good sediment transport capacity downstream of the bridge.

The product of redirecting the river flow under the bridge at an enhance incidence angle, using the methods prescribed at locations A, B, C, D, E, and G, is an increased potential for eroding the right bank of the river downstream of the Solomon Bridge. In that circumstance, we recommend installing bendway weirs at location F. These weir types are commonly used by the US Army Corps of Engineers and Reclamation to protect bendways and to increase mid-channel sediment transport capacity. Figure 9 shows a bendway weir field and the theoretical flow redirection that can result.

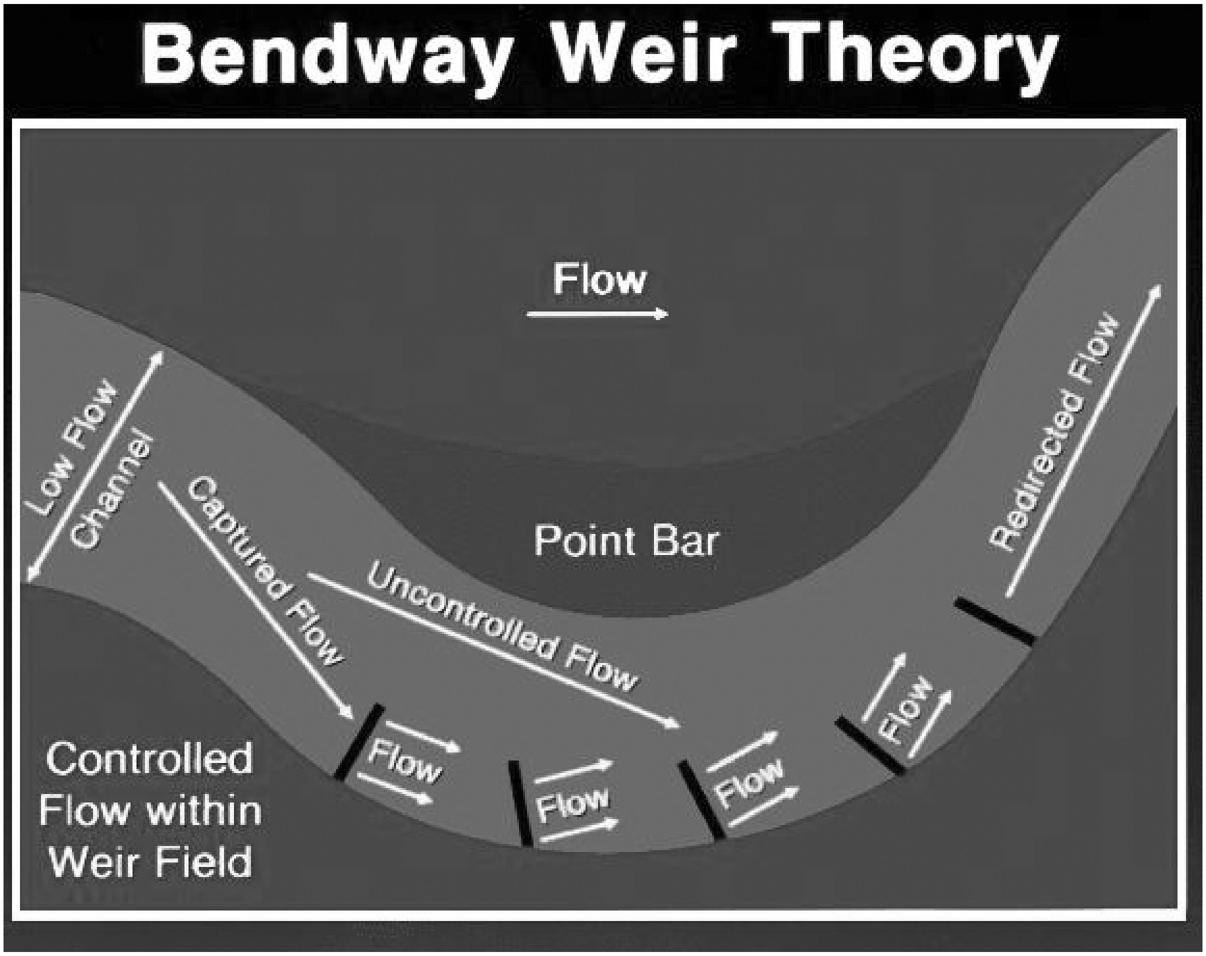

Figure 9. Bendway weir field. (http://chl.wes.army.mil/research/hydstruc/bankprotect/bendweir/work.htp)

Tools and Design Planning

This section briefly describes the tools available to the Cooperators as they move forward from this study into direct investigations and plans for demonstration projects and associated designs. The tools are publicly available and utilized heavily by the Bureau of Reclamation.

The tools fall into two categories: 1.) Data and information developed as tasks of this study; 2.) Analytical tools and references available from Reclamation, the US Army Corps of Engineers, other federal and state agencies, and private entities.

Study Data

The photogrammetry, orthophotographic mosaics, and associated digital terrain models (DTM's) are the primary physical data produced in this study that will directly aid in the planning and design of demonstration projects. The conceptual level of design in this report is intended only to outline the type, location, purpose, and general configuration of a proposed demonstration project. The responsibility for developing these conceptual projects to the point of implementation is the Cooperators. Reclamation stands ready to assist.

The DTM's currently reside in the Denver Technical Service Center (TSC) in the form of the original DTM's, that is the product of the Soft Copy photogrammetric process. The DTM's are readily uploadable into Microstation or AutoCad software for civil engineering design and manipulation. In combination with the orthophotographs, the DTM's are a powerful design base.

Tools
There are several tools to aid in the design of riprap at bridges. A general background resource is "Streambank", an interactive computer based manual produced by the US Army Corps of Engineers and published by VeriTech of Vicksburg, Mississippi. (VeriTech, 1998). The US Federal Highway Administration publishes Hydraulic Design Series Number 6, River Engineering for Highway Encroachments, Highways in the River Environment (FHWA NHI 01-004, 2001).

The US Army Corps of Engineers have two particularly useful analytical tools, HEC-RAS, and CHANNEL PRO. HEC-RAS is the industry standard step backwater program. Reclamation produced HEC-RAS models as part of this study. The computer files are included in the report package. The purpose of HEC-RAS is to calculate depth and average channel velocity for a variety of flows and channel configurations. CHANNEL PRO is another USACE computational aid, implementing riprap design formulas in EM-1110. The aid is informally available from Reclamation. CHANNEL PRO produces standard gradations for riprap designs based upon the reach hydraulics (HEC-RAS) and geometry (DTM). The design flows for a range of return intervals, i.e. 100-yr flood, can be determined from the study report "Flood Frequency and Flow Duration Analyses – Arizona."

Methods
A.) The riprap berm at location 'A' is a windrow berm as described in "Streambank" (VeriTech, 1998). The CHANNEL PRO computational aid is perfectly suited to design the riprap for the berm, once the alignment is determined. Reclamation suggests an elliptical shape with a 3:1 aspect ratio, the longer axis paralleling the desired streamlines of the flow directed under the Solomon Bridge.

B.) Likewise, CHANNEL PRO is a suitable tool for enhancing the current un-engineered riprap lining the upstream side of the left approach abutment.

C., D., & E.) Highways in the River Environment (FHWA, 2001) contains all of the procedures and techniques for designing the protection for both highway abutments.

F.) Tools for designing the bendway weirs proposed at 'F' are less developed. Appendix C contains an unpublished US Army Corps of Engineers Design Guidance for Bendway Weirs. Reclamation finds this design guidance to be useful and conservative.

BRIDGES AND LEVEES
Integrating bridges into a setback levee system is critical to the overall success of a flood control program. Bridges are necessarily narrower than the floodplain. Therefore, allowance for narrowing the floodway from the setback levee width to the bridge span width is necessary. The best method is a streamlining of the levees as they enter and exit the bridge section.

Figure 10 shows an example of how a levee setback scheme could be transitioned into and out of the Thatcher Bridge section. The contraction upstream of the bridge may be shorter than the expansion downstream. It is important to minimize energy losses in the downstream transition and maintain sediment transport capacity. Otherwise, sediment may accumulate in the main channel, causing the river to shift left or right onto the over channel and floodplain. A similar integration of levees with diversion dams is also necessary.

Reclamation recommends that plans for demonstration projects that impact the flood control scheme in the study area undergo proper conception, design, regulatory input and review, and permitting. The channel widths suggested in Table 1 should serve as the beginning point of a standard flood control levee design.

Figure 10. Potential levee realignment at Thatcher Bridge. (Flow is left to right)

DIVERSION DAMS

There are several diversion dams in the Safford and Duncan valleys. In every case sediment has accumulated upstream of the dams, causing significant geomorphic impacts. The primary impact of the diversion dams is to reduce the slope and sediment transport capacity upstream, resulting in severe lateral channel migration and subsequent loss of property. The irrigation companies expend resources in the attempt to contain the river upstream of the diversion dams. The most visible example of this is the Graham Canal diversion outside of Safford.

One option to mitigate these impacts would be to remove the diversion dams and replace them with infiltration galleries and pumps. Another intriguing option to the diversion dams is a partial replacement of the dams with inflatable gates or dams. This section provides some background on these types of dams, their geomorphic impact, and potential costs.

The figures in this section originate with the Obermeyer Gate Company, and the US Army Corps of Engineers Cold Regions Research Laboratory and Bridgestone Rubber Company. The publications that were the source of the figures are freely available on the Web. Use of these figures or the mention of the companies that provide the gates, dams, or services, is not an endorsement by the Bureau of Reclamation, or the US Government. The purpose of using these figures is to present the concept of these type of inflatable dams. The authors are grateful to the Obermeyer Gate Company and the Bridgestone Rubber Company, as well as the Army Corps of Engineers for making these materials available on the web. The brochures from both of these companies are attached as Appendix A and Appendix B.

Figure 11 shows a cutaway view of an Obermeyer gate. A cross section of the gate is visible in the cutaway section. These gates differ from inflatable dams with the inclusion of a gate flap placed upstream of and on top of the inflatable bladder.

In contrast to the Obermeyer Gate, the rubber inflatable dam is another option. Figure 12 is a conceptual drawing of a rubber inflatable dam and components. The inflatable rubber dam does not have a gate flap. Instead the rubber dam relies on the strength of the rubber alone. The Corps of Engineers tested these types of dams in an ice environment. Obviously that is not the specific problem in Arizona. However, the Gila River is heavily laden with floating debris during floods. The behavior of the rubber dams under ice loading is an indicator of their ability to withstand floating debris.

Figure 11. Obermeyer Gate.

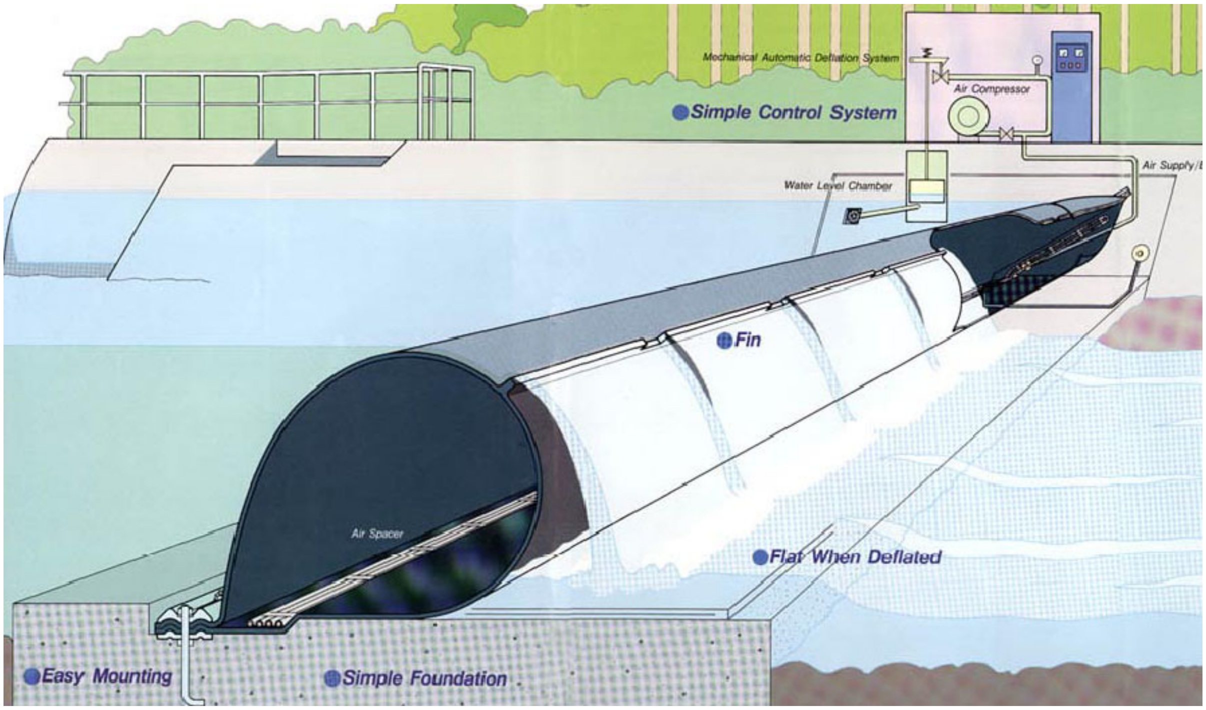

Figure 12. Inflatable rubber dam.

Figure 13 shows independent operation of Obermeyer Gate sections. This is a useful feature for maintaining head for irrigation diversion, while passing intermediate flood flows.

Figure 13. Obermeyer Gates on a concrete crest. Note independent operation of gate sections.

GRAHAM DIVERSION

The Graham Canal Diversion Dam demonstrates the geomorphic consequences of interrupting the sediment budget in the Gila River. The current dam is roughly 300 feet wide, with an estimated hydraulic height of less than 10 feet. Figure 14 shows the area upstream of the diversion dam. The amount of land lost to erosion above the diversion is large. The maintenance costs associated with the diversion, namely excavating a channel to the headworks, and attempting to keep the river from flanking the structure entirely, are significant.

Figure 14. Area view east of Safford, Arizona. Graham Canal diversion dam is the cause of property loss in this area.

A potential partial solution is to replace a section of the fixed diversion dam with either an inflatable rubber dam or multiple sections of Obermeyer Gate. Figure 15 shows the diversion dam. The three colors represent sections of the dam. The red section is close to the headworks and should not be disturbed. The green section is the preferable section for replacement. It aligns well with the center of the channel, and could easily capture a large portion of intermediate flood flows. The yellow section is also a candidate for replacement. It is furthest from the headworks, and might need to be rebuilt in any case if the diversion dam is reconstructed.

The idea of an inflatable dam or Obermeyer Gate is to facilitate both irrigation diversions during low flows and intermediate floods, as well as sediment transport during intermediate and larger floods. The inflatable operation allows the dam to divert flows at maximum head during low flows, then to deflate and allow sediment transport and maximum flow conveyance during floods. Transporting sediment past the dam reduces the potential for flanking of the diversion, especially on the descending limb of a flood.

Figure 15. Location of the Graham diversion dam, and potential sections for replacement with rubber inflatable dam or Obermeyer Gate.

The rough costs for a Bridgestone Rubber Dam – 7.5 feet tall, 500 feet long – is $2M. The rough cost for a 15 feet tall, 165 feet long rubber dam – is $5.0M. Obermeyer Gates were not submitted for a rough bid.

An economic comparison of inflatable rubber dams, Obermeyer Gates, or dam removal and replacement with infiltration galleries and pumps should form the basis for a community discussion regarding the geomorphic impact of the diversion dams.

GENERALIZED MONITORING PLAN

The purpose of this monitoring plan is to collect the data necessary to discern changes to the Gila River channel over time due to the implementation of channel and levee altering demonstration projects. The monitoring plan will also track the performance of these types of projects.

DATA REQUIREMENTS

The principle data required for this monitoring program are distance and elevation measurements for each cross section and a channel length measurement through each demonstration project reach. The distance and elevation data are best collected by a field survey between permanent monuments that have been established at the ends of the cross sections. Permanent monuments should be established at the ends of the cross sections at or near the prescribed coordinates. These monuments may consist of standard benchmarks or reinforcement bar set in concrete. The monuments should be sited in an area that is easily located, considered stable (roadway, local landmark, etc.), and is not prone to disturbance or vandalism. Detailed notes describing location and distances to nearby landmarks such as telephone poles, fence posts, bridge piers, trees, etc. should be developed for each cross section and incorporated into a permanent monitoring project database.

Project Leaders should establish a project database to gather and archive monitoring data. The development and implementation of standardized data forms should also be considered to ensure the consistency of the data collected. In addition, the location (Lat/Long) as well as the distance and direction between the endpoint monuments should also be documented so that if a monument is lost, it can be reestablished without compromising the dataset for that cross section.

Given the scope of this monitoring plan, once the monuments and the baseline conditions for each cross section have been established, the time required to acquire and process the data should not exceed more than 10 staff days annually. In order for the data collected as part of this monitoring plan to be utilized, a baseline dataset for each cross section must first be established. It is recommended that this baseline dataset be populated with data collected on a bi–monthly basis during the first year of monitoring following implementation of a demonstration project. These data should then be averaged to provide the baseline that would be considered representative of the current river conditions.

After an initial baseline dataset is established, all cross section measurements should be repeated annually, on or about the same date. It is suggested that this survey be undertaken sometime in the fall because the base flow on the Gila River at this time of the year is low and the vegetation along the river has lost its leaves. Little or no flow and dormant vegetation will facilitate data collection and improve the quality of the survey data by increasing the accuracy of the channel geometry measurements and reducing random error incurred due to foliage on the vegetation. Surveying at this time of the year should document any changes along the river that may have resulted from flooding during the previous year.

An assessment of the data collected should be performed at 5–year intervals. This assessment would establish the range of expected variability in annual measurements. Threshold criteria should be developed based on the baseline data collected during the first year and the sediment model predictions. The monitoring project should be continued for a minimum of 10 years. At the end of this period, the decision to continue monitoring of the project would be based on the result of the data assessment.

The amount of measurable stream flow in the river at the time of the cross section surveys should be recorded. These values are available from the United States Geological Survey. A record of the stream flow during the period of the survey should be included with the cross section surveys in the monitoring program database.

In addition to distance and elevation data acquired in the field survey of the cross sections, the channel length in the monitored reach should also be determined. Channel length is extremely important to the data analysis, as it is required to accurately calculate the channel slope and derive representative thalweg profiles. It is also very important to understand that the channel length is not equivalent to the distance between the measured cross sections. The measure of channel length can be collected by two different methods, field survey or from aerial photography. Gathering this information can also be complicated if there is any significant flow in the river at the time of the survey. Some of these logistical problems can be eliminated by scheduling the field survey at a time when flow is low or non–existent, the vegetation has lost it leaves, and with the use of GPS survey equipment.

It is strongly encouraged that aerial photography also be acquired on an annual basis coincident with the collection of channel geometry data (i.e., within several weeks). In addition to the invaluable record that it provides, aerial photography is more comprehensive in the sense of total data gathered and for documenting channel conditions that are not easily measured in the field. Information derived from aerial photography can add to and improve the quality of data in the database, and hence may be much more economical in terms of the incremental costs versus the data collected.

The primary purposes for acquiring aerial photography are to detect and document changes in the channel plan form associated with meandering or channelization, evaluate vegetation conditions and to identify the location and derive a length for the channel between measured cross sections. Documenting changes in these parameters cannot be determined from survey data in the monitored cross sections alone. To gather this information in the field would be very time intensive and subject to numerous errors that could not be evaluated in later analyses. Most of this change occurred between the cross sections, so these changes might not have been documented in a field survey of channel geometry. At a minimum, the aerial photography acquired as part of this monitoring plan should include uncontrolled stereo coverage of the monitored reach flown at a scale of roughly 1:12,000 at least every five years and after every flood that exceeds the twenty year flood. With the placement of some permanent monuments, the photography could be rectified and utilized in later detailed analyses, should the occasion arise. While annual aerial photographic coverage of the monitored reaches would be optimal, it could potentially increase the program costs by as much as 50%.

DATA ACQUISITION

When surveying each cross section, the maximum distance between points in a cross section should not exceed 100 feet. A minimum number of 25 points, excluding end–points, should be surveyed in each cross section. Obviously, the more survey points collected, the more accurate the cross section. Changes in elevation across the flood plain or in the channel of more than 2 feet should be included in the survey so that topographic breaks can be accurately represented in a graphical depiction of the cross section. This is accomplished by surveying a point at the top and bottom of the break. In addition, the following details must be noted during the survey and included in the monitoring database. All references to right or left should be made in the context of the feature's position while looking downstream.

- The position of the vegetation on the right and left sides of the active channel; for example, left edge of vegetation (LEV) and right edge of vegetation (REV). Figure 16A illustrates the definitions and locations of these features. When the active channel of the river consists of multiple threads, measure the position of the LEV and REV for each channel thread.
- The position of the channel bank on the right and left sides of the active channel. Because knowing the position of the top of the bank can be useful in analyzing other hydraulic characteristics of the river, the top edge of both banks should be noted. For example, top right bank (TRB) and top left bank (TLB), illustrated in Figure 16A. When the active channel of the river consists of multiple threads, measure the position of the TRB and TLB for each channel thread. Most banks will represent a topographic break in the cross section (see preceding paragraph), therefore a survey point should be measured at the base and top of each bank.

- The position of the left edge of water (LEW) and right edge of water (REW) when there is flow, illustrated in Figure 16A. When the active channel of the river consists of multiple threads, measure the position of the LEW and REW of each channel if flow is present.
- The position of the channel thalweg, the lowest point in the active channel, as illustrated in Figure 16A. Measure the thalweg in each channel of a river with multiple threads or channels.
- The position of any boundaries or essentially permanent features in the cross section such as roads, fence lines, levee crests, bedrock outcrops, large trees, etc.

Similarly, when surveying the channel length in the monitored reach, the maximum distance between points should be less than 100 feet. The channel length measurements should be collected as close to the thalweg as possible. Obviously, it would be advantageous to collect these data when there is little or no flow in the channel. Finally, each cross section should be photographed from both endpoints. Each pair of photographs should be annotated with the time, date, and cross section number and included with their respective cross section datasets in the monitoring program database.

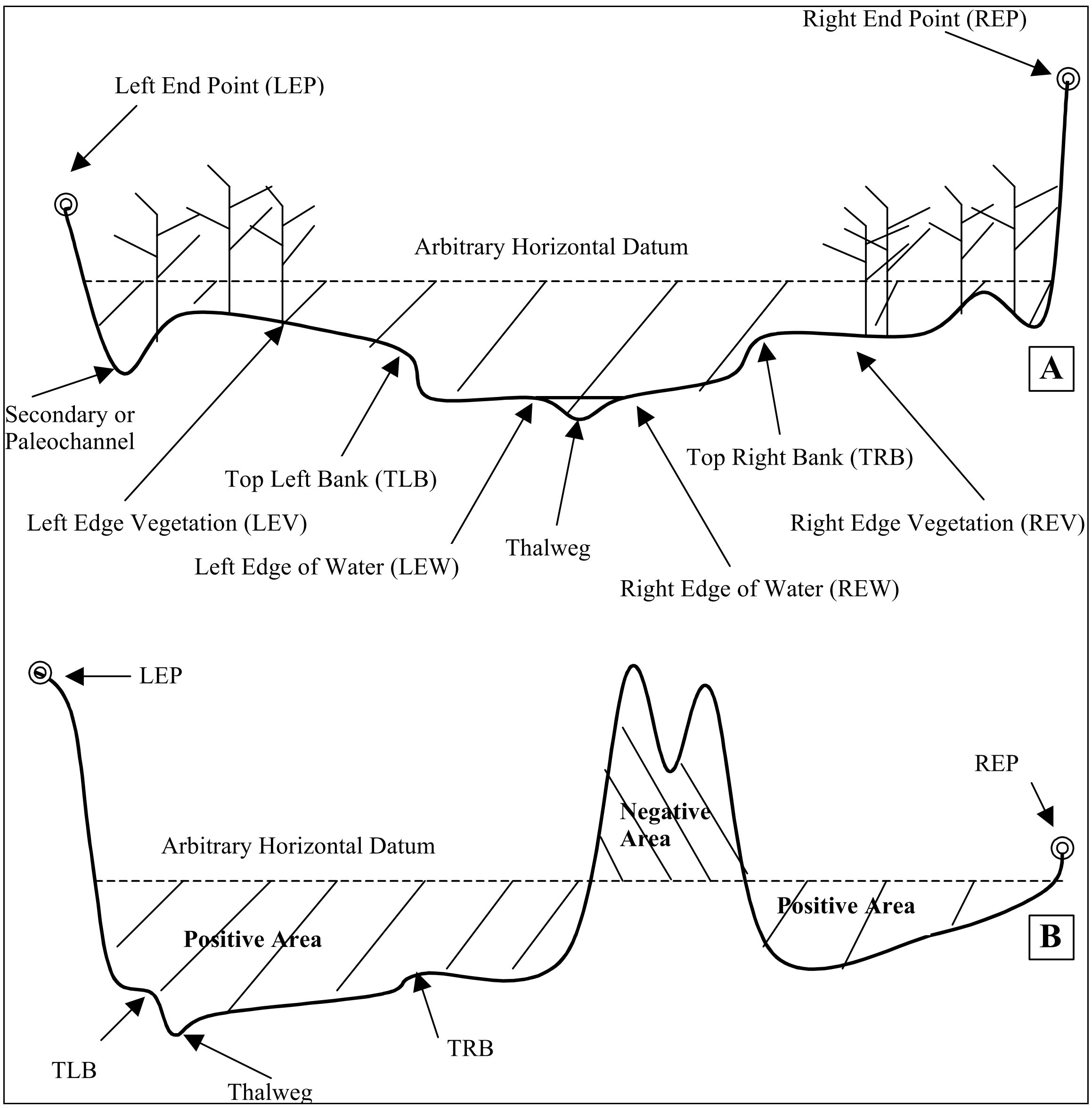

Figure 16. ***A.*** *Diagram showing typical cross section and placement of arbitrary horizontal datum.* ***B.*** *Diagram showing a cross section with a portion of the cross section above the datum.*

DATA ANALYSIS

The distance and elevation measurements from the cross section surveys and the channel length measurements collected from either the field survey or aerial photography will be used to assess the river conditions. Three basic parameters, the thalweg elevation, the cross sectional area, and the channel slope, developed using these data will be analyzed. These parameters are sensitive indicators of changes on the river that result from aggradation or erosion. The first parameter to be analyzed from these data is the thalweg elevation. The thalweg in a river channel is defined as a line connecting the lowest points along the channel bed. In this case, the thalweg elevation is defined as the lowest point in the active channel within each cross section. It is possible that the thalweg elevation will not coincide with the lowest point in the cross section. At several locations along the Gila River, the active channel of the river is perched so the bed elevation in the active channel is actually higher than in isolated or abandoned channels on the flood plain. In many cases, these abandoned channels or isolated back channels may only convey flow during large magnitude floods. Thus, it is extremely important that the position of the thalweg in the active channel be clearly noted in the cross section during the field survey and distinguished from secondary or paleochannels that may be present in the cross section. Figure 16 shows examples of this type of channel morphology. The position of the thalweg and secondary channels can also be determined on the aerial photography, thereby verifying field measurements and eliminating potential error resulting from field personnel unfamiliar with specific river characteristics and terminology.

Monitoring changes in the thalweg elevation can be helpful in detecting increases or decreases in the bed elevation resulting from aggradation or erosion. Therefore, thalweg data is best evaluated in a time series analysis. However, numerous years of data need to be gathered before the analysis will be meaningful. Each year data can be compared to previous data sets to evaluate systematic changes or trends in the bed elevation that may result from either erosion or aggradation. The thalweg elevation data in a given cross section can also be compared to the thalweg elevation data in adjacent cross sections. A comparison of these data in each cross section in a given reach could indicate if changes are localized or reach–wide.

The second parameter, the cross sectional area, can be evaluated using distance and elevation measurements in each cross section. The cross sectional area provides a means of measuring changes in the stored sediment in a given cross section. This value acts as a proxy for volume and is independent of such complicating factors as multi–thread channels, stream terraces of different ages, sand dunes, and vegetation encroachment. In this case, the cross sectional area simply represents the available space in the cross section measured between the ground surface in the cross section and a previously established horizontal datum for each particular cross section, as shown in Figure 16A. If the river aggrades in a particular cross section, the available space will decrease; if the cross section experiences erosion, the available space will increase.

It is important to note that each cross section has its own unique horizontal datum and that all cross sectional areas calculated in a given cross section must utilize the horizontal datum established for that cross section. The datum is established at an arbitrary elevation in the cross section that is located as close to the ground surface as possible yet allows for all of the measured points in the cross section to fall below the datum, as shown in Figure 16A. This minimizes the area in the cross section to the point that small changes in the area from year–to–year are readily detected in the analysis. In some cross sections, the flood plain may be covered by high dunes or a channel may have migrated from one side of the cross section to the other leaving a higher isolated portion of an abandoned terrace in the cross section. In order to locate the datum at a minimal elevation and facilitate the area computations in the monitoring program, some areas of the cross section may lie above the datum, as shown in Figure 16B. In these particular cases, the area of the cross section above the datum is considered negative area. In the analysis, the negative area of the cross section would then be combined with the positive areas to derive the cross sectional area.

The computed cross sectional area derived from the above analysis is used to evaluate river conditions in two different ways. First, compare this value to previous area measurements at the same location to evaluate the magnitude of change within the cross section. Second, compare this value statistically to cross sectional areas measured in adjacent cross sections in the reach to detect any deviation in trends within a reach. Figure 16 illustrates how the cross sectional area simply represents the available space in the cross section. Therefore, if the river aggrades in a particular cross section or through a particular reach, the available space will decrease. Conversely, if the channel experiences any degradation as the result of either bank erosion or bed scour in the cross section, the available space will increase.

The third parameter to be analyzed is channel slope or the thalweg profile. The channel slope is simply the change in the bed elevation over some distance along the channel. Changes in channel slope are closely related to the capability of the river to move sediment. The channel slope decreases as the channel aggrades and increases as it degrades. This is a broad generalization as channel slope is also dependent on other channel characteristics and stream flow. Therefore, it is important to understand which parameters are influencing the channel geometry and river behavior. The data required to calculate the channel slope includes channel length derived from either field survey or aerial photography and the thalweg elevations through the entire monitored reach. There is a variety of methods that may be employed to analyze this data. In this particular case, a time series comparison of thalweg profile or channel bed elevation plotted against the main channel distance should prove adequate. Again, it is important to recognize that the distance between cross sections is not necessarily equivalent to the channel length.

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Appendix A

OBERMEYER HYDRO, INC.
SPILLWAY GATES BROCHURE

OBERMEYER HYDRO, INC.

P.O. Box 668 Fort Collins, Colorado 80522 USA
Tel 970-568-9844 Fax 970-568-9845
Email: hydro@obermeyerhydro.com www.obermeyerhydro.com

Thank you for your interest in Obermeyer Spillway Gates. Obermeyer gates offer an economical and technologically superior method of spillway control. Some of the features include:

1. Obermeyer Spillway Gates conform to almost any spillway shape without costly changes to the existing spillway profile.

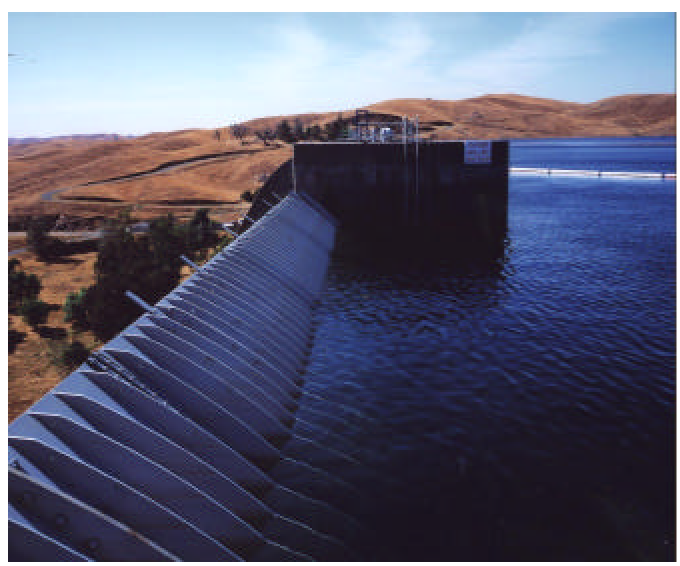
2. The rugged steel gate panels overhang the reinforced air bladders in all positions. The gate panels protect the air bladders from damage due to ice, logs, or other debris.

3. The Obermeyer Spillway Gates are very controllable. Our gates can be set at an infinite number of positions between fully raised and fully lowered. Our standard pneumatic controller provides accurate upstream pond control, and discharges water appropriately to maintain upstream pond elevation through a full range of flow conditions.

4. Obermeyer Spillway Gates use no high precision parts or bearings. This allows for easy installation and long service life.

5. Obermeyer Spillway Gates use clean, dry, compressed air for actuation. No hydraulic fluid or other contaminates are used.

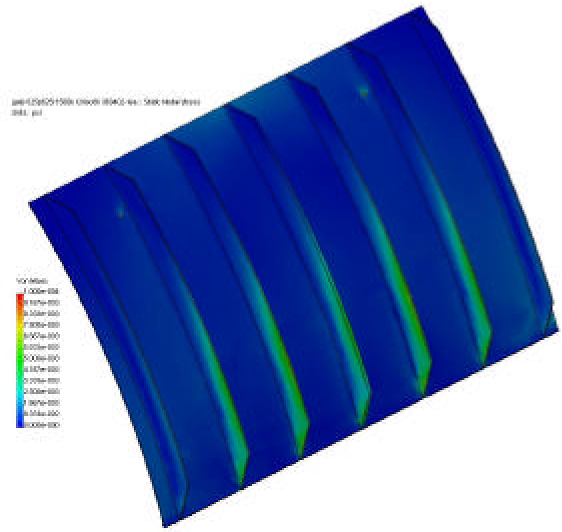
6. The modular design of Obermeyer Spillway Gates creates a very safe operating system. For large gate systems, each air bladder is isolated from the other by means of a check valve. If one air bladder becomes damaged, the rest of the gate system will not deflate through the damaged section.



7. The modular design of Obermeyer Spillway Gates simplifies installation and maintenance. The use of individual air bladders and gate panels minimizes the lifting capacity required for installation. This saves significant time and money by reducing the size of equipment and manpower needed to install the system.

8. Obermeyer Spillway Gates are very vandal and damage resistant. From the upstream side, steel panels protect the air bladders in all positions. Damage due to ice, trees, or other debris is nearly impossible from the upstream side. The air bladders are reinforced by multiple plies of polyester of aramid tire fabric. The use of these types of fabrics, in combination with generous thickness of rubber, creates a very bullet and vandal resistant air bladder.

9. Obermeyer Hydro utilizes state of the art engineering and software packages to insure that each gate system design will be safe and reliable. Gate panels and other steel components are designed using the latest finite element analysis programs.



We hope this package answers the questions you have regarding Obermeyer Spillwa y Gates. If you have any other questions, please don't hesitate to contact our head office by phone or email. If you desire a site-specific price quote, please refer Page 4, Site Specific Details, which lists questions asked by our applications engineers when designing a project.

Once again, we appreciate your interest in Obermeyer Spillway Gates and we look forward to hearing more about your project.

Sincerely,
Rob Eckman
Vice President
Obermeyer Hydro, Inc.

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Obermeyer Spillway Gates are manufactured under license under one or more of the following patents:
U.S. PATENT 4,780,024 • U.S. PATENT 5,092,707 • U.S. PATENT 5,538,360 • U.S. PATENT 5,642,963
U.S. PATENT 5,709,502 • U.S. PATENT 5,713,699 • OTHER U.S. AND FOREIGN PATENTS PENDING

Introduction

Obermeyer Spillway Gates are most simply described as a row of steel gate panels supported on their downstream side by inflatable air bladders. By controlling the pressure in the bladders, the pond elevation maintained by the gates can be infinitely adjusted within the system control range (full inflation to full deflation) and accurately maintained at user-selected set points.

Obermeyer Spillway Gates are patented bottom hinged spillway gates with many unique attributes that include:

- Accurate automatic pond level control even under power failure conditions.
- Modular design simplifies installation and maintenance.
- Unlike torque tube type spillway gates, Obermeyer gates are supported for their entire width by an
  inflatable air bladder, resulting in simple foundation requirements and a cost effective, efficient gate structure.
- Thin profile efficiently passes flood flows, ice, and debris.
- Unlike rubber dams, the steel gate panels overhang the air bladder in all positions, protecting the bladder from floating logs, debris, ice, etc.
- No intermediate piers are required.
- Obermeyer Spillway Gates are a great investment due to increased revenue, decreased maintenance, and low cost of installation.

These features are the result of combining rugged steel gate panels with a resilient pneumatic support system.

The Spillway Gates are attached to the foundation structure by anchor bolts which are secured with epoxy or non-shrink cement grout as design dictates. The required number of air bladders are clamped over the anchor bolts and connected to the air supply pipes. When the air bladder hinge flaps are fastened to the gate panels, the installation of the strong, durable and resilient crest gate system is complete.

View of Gate from Downstream

The individual steel gate panels and air bladders are fabricated in widths of five or 10 feet, (1.5 meters or 3 meters for metric installations) for systems up to 6.5 (2 meters) high. Systems higher than 6.5 feet (2 meters) use various standard width air bladders such that the height/length ratio is less than approximately 1.0.

The gaps between adjacent panels are spanned by reinforced interpanel seals clamped to adjacent gate panel edges. At each abutment, a robust, low-friction lip seal is affixed to the gate panel edge. This seal moves along the abutment plate, keeping abutment plate seepage to a minimum. For installation in cold climates the abutment plates are provided with heaters to prevent ice formation. Alternatively, rubber seals may be fixed to the abutments or piers which engage when raised.

Hydraulic Performance

Obermeyer Spillway Gates provide excellent controllability over a full range of flow rates, water elevations and gate positions.

All gates operating on the same air supply line maintain a uniform crest height. This is because any differential lowering of a gate panel relative to others on the same air supply manifold causes said gate panel to develop more contact area with its respective air bladder than other gate panels. The extra contact area produces a restoring moment that returns said gate panel to the same position as the others.

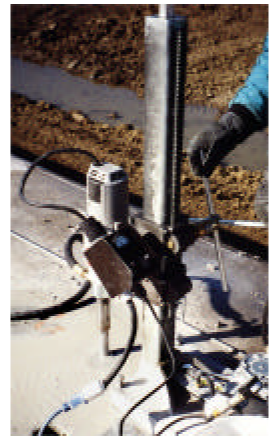
Vibration due to von Karman vortex shedding does not occur with Obermeyer spillway gates. The shape of the system when raised or partially raised causes flow separation to occur only at the downstream edge of the gate panels. This favorable condition also occurs when the system is operating in a submerged or high tailwater condition; in contrast, rubber dams which due to their rounded shape can vibrate destructively as the line of flow separation moves cyclically back and forth across the rounded surface of the inflated structure.

Obermeyer Spillway Gates provide very repeatable positioning relative to inflation pressure and headwater level and can be used to precisely measure the flow, as well as control flow.

Obermeyer Spillway Gates can be operated continuously over a full range of gate positions, headwater elevations and tailwater elevations and may be installed within siphon spillways subject to extreme water velocities.

Installation

Installation of Obermeyer Spillway Gates is quick and easy. For systems up to approximately 4 meters high, the air bladders are secured to the spillway with a row of anchor bolts. For system heights above 4 meters, an embedded clamp is used to secure the gate system to the spillway. The anchor bolts may be embedded in a new spillway or may be secured in holes drilled into an existing spillway. The air supply lines, which connect to each individual air bladder, can be embedded or grouted into a saw slot in the spillway. Surface mounted air supply lines may also be used. A typical installation sequence is as follows:



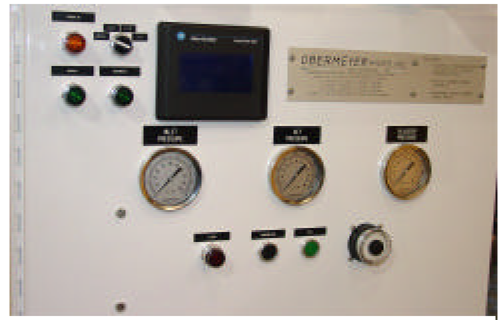
Drilling of Anchor Bolt Holes

1. Place anchor bolts
2. Install air supply lines
3. Install abutment plates, if used
4. Place air bladders over anchor bolts
5. Secure air bladders to spillway with clamp bars
6. Connect air supply lines to underside of air bladders
7. Attach steel gate panels to each air bladder
8. Attach interpanel seals
9. Attach restraining straps if used
10. Attach nappe breakers
11. Adjust and grout abutment plates or install J seals
12. Install compressor, drier and controls
13. Start up system

Installation of Gate Panels

Start of Installation – Installing Gate Panel – Completed Gate

Types of Control Systems



Control System with Touch Panel

Obermeyer Spillway Gates are supplied with control systems in accordance with customer requirements. Each control system includes a controlled source of compressed air and a means for controlled venting of air from the air bladders. All automatic systems also include provision for local manual control. Each system includes an air compressor, a receiver tank, and required control valves. Most systems, especially those subject to freezing conditions, include air driers.

Pneumatic Water Level Control
The most basic control system uses an all-pneumatic water level controller to automatically regulate air bladder pressure in inverse proportion to upstream water level. This system requires no electrical power to accurately maintain a constant upstream pool elevation over a full range of gate positions and spillway flow rates. This controller is ideally suited to hydroelectric projects where a turbine load rejection is often associated with loss of electrical power. This control system is also ideal for safety critical flood control projects where flood conditions and extended loss of electrical power often occur simultaneously. A bubbler line senses upstream water level. The minute amount of air required for the bubbler system is supplied from the air receiver with the air stored within the air bladders connected as a backup supply.

Programmable Controllers
In many applications, it is desirable to control Obermeyer Spillway Gates with a Programmable Controller. A Programmable Controller is ideal for complex schemes such as maintaining precise environmentally mandated spillway flows under varying head pond elevation at hydroelectric peaking plants. Pre-existing programmable controllers at numerous hydroelectric plants have been used to control Obermeyer Spillway Gates, thus reducing the overall cost of the gate installation. Conversely, at new projects, an Obermeyer supplied Programmable Controller can also serve other control requirements not related to the spillway gates. Programmable Controller based systems can be provided with Pneumatic Water Level Controllers as a mechanical backup.

Solar Powered Controls
Obermeyer Spillway Gates can be supplied with solar powered compressors and control systems. Obermeyer Spillway Gates are well suited to solar powered operation because no large electric motors are required even on quite large gate installations. Solar powered systems normally use 12-volt solar panels, battery and compressor. A programmable controller with optional radio modem operates the compressor or vent valves in accordance with water level readings or remote control signals.

Safety Critical Applications
For relatively small gate installations on large rivers, it is usual to operate all of the air bladders on the same pipe or pressure manifold. For large gate installations on narrow populated river channels, check valves are used on each air bladder to insure that damage to any one air bladder cannot release air from any of the other air bladders. This feature is an important safety advantage of Obermeyer Spillway Gates over rubber dams.

Independent Operation of Groups of Gates

At many projects it is desirable to control various sections of the spillway independently. This can be accomplished by simply providing separate pipes to each independent section. No intermediate piers are required. Applications for this scheme include:

- Releasing floating debris from near a power plant intake.
- Concentrating flows to discharge upstream sediment.
- Minimizing tailwater elevation by releasing excess flow away from the power plant.
- Providing fishway attraction water in the precise amounts and locations needed.
- Diverting flows to allow inspection access to the raised portion of a gate system.

Flow Measurement and Control

Obermeyer Spillway Gates respond to changes in headwater elevation and internal air pressure in a precise and repeatable manner. For any particular gate installation, the flow rate and gate crest elevation can be calculated on the basis of the measured up stream pond elevation and the controlled air bladder pressure. Flow rates for submerged installations, i.e., installations with high tailwater, can be calculated on the basis of upstream and downstream levels and air bladder pressure.

Gate Panels

Gate panels are made from high strength steel plate that is epoxy coated or galvanized in accordance with customer preference. Stainless steel gate panels may be supplied on request. Gate panels for systems less than 1 meter high are made from a flat plate that is bent to conform to the spillway shape when in the lowered position. A small amount of additional curvature of the gate panel profile is provided to allow space for the deflated air bladder when the gate panels are fully lowered. Gate panels for systems higher than 1 meter are provided with stiffening ribs running parallel to the direction of flow. The ribs provide strength without obstruction of flow. A high degree of torsional rigidity is not required because of the uniform support of the gate panels by the air bladders. For the same design stress level, the gate panels are much lighter, less costly and less restrictive to water flow compared to gate panels for hydraulically or mechanically operated gates.

Gate panels are provided with a row of threaded studs near the pivot edge to which the hinge flap is clamped. Similar threaded studs are provided at the right and left edges of each gate panel for sealing to the adjacent gate panels or to the abutments.

The outermost ribs on each gate panel are provided with lifting holes. The upper/downstream edge of each gate panel features holes or studs for the attachment of nappe breakers. For installations that utilize restraining straps, holes or studs are provided for attaching the restraining straps to each gate panel.

The upstream/lower edge of each gate panel features a smooth rounded surface for transferring a reaction load to the air bladder and hinge flap.

Air Bladders

Air bladders are designed and manufactured by methods similar to those used in the manufacture of automotive tires. A butyl rubber inner liner provides excellent air retention characteristics. A intermediate layer of high tensile strength rubber compounds containing multiple plies of polyester or arimid tire cord reinforcement, e.g. DuPont KEVLAR ® fiber, provide the mechanical strength needed to contain the internal pressure. A cover compound utilizing aging and ozone resistant polymers such as EPDM is used to protect the bladder from wear and weathering.

Air bladders for systems of less than 2 meters in height incorporate integral hinge flaps to which the gate panels are attached. Systems higher than 2 meters utilize separate hinge flaps which utilize the same high strength tire cord construction as the inflatable portion of the air bladders. No mechanical hinges are used.

Comparison Chart

Obermeyer Spillway Gates vs. Rubber Dams

Advantages of Obermeyer Spillway Gates:	*Disadvantages of Rubber Dams:*
Precise control of upstream elevation over a full range of headwater elevations and gate positions	*The inflatable membrane is exposed directly to ice and debris*
Unlimited spans can be installed without intermediate piers	*Allowable overtopping is limited by vortex shedding induced by vibration*
Steel panels provide robust protection from debris damage	*Replacement at an entire span is required if damage cannot be repaired*
Vertical abutments provide maximum discharge capacity and reduced civil costs	*Discharge along crest is non-uniform when partially inflated*
Modular design reduces maximum required crane capacity	
Modular design allows change out of any damaged components without requiring whole system replacement. This dramatically reduces life cycle cost and limits any downtime	
Check valve isolation of individual air bladders maximizes public safety by dramatically limiting unintended flows which could result from air loss	
Obermeyer Spillway Gates can provide precise flow data and flow control	

Site Specific Details Questionnaire

The following information should be supplied to Obermeyer Hydro, Inc. to facilitate the design of a Spillway Gate System:

1. Is the proposed gate installation on an existing dam or a proposed dam?

   What is the proposed: Length? ______________________________

   Height? ______________________________

   Fixed crest elevation? ______________________________

   Top of Gate elevation? ______________________________

   Tailwater Rating Curve? ______________________________

   Upstream streambed elevation? ______________________________

   Downstream streambed elevation? ______________________________

2. If this is a new dam, is it founded on bedrock or sand, gravel, clay, etc.?

3. What existing features such as piers, abutments, intakes, exist?

4. What is the desired function and purpose of the proposed gate structure?

5. Local Regulations, such as national electrical codes:

6. Anticipated debris flow:

7. Climate description including minimum and maximum temperature and humidity. Ice conditions if applicable.

8. Control System functions required? Automatic upstream level control, diversion flow control, etc.

9. Control system power source, 1 phase, 3 phase, solar, etc.?

10. Required inflation and deflation time of bladders:

Appendix B

PERFORMANCE SURVEY OF INFLATABLE DAMS IN ICE-AFFECTED WATERS
ICE ENGINEERING, NUMBER 30, OCTOBER 2001
US ARMY CORPS OF ENGINEERS, COLD REGIONS RESEARCH & ENGINEERING LABORATORY

US Army Corps of Engineers®
Cold Regions Research & Engineering Laboratory

Ice Engineering

U.S. Army Cold Regions Research and Engineering Laboratory, Hanover, New Hampshire

Performance Survey of Inflatable Dams in Ice-Affected Waters

Since their first appearance in the mid 1950s, inflatable dams have gained increasing acceptance. There are now more than 2000 of these structures in use worldwide,[1] with an increasing proportion in ice-affected waters. The purpose of this survey is to document the performance of existing inflatable dams in rivers with ice, and outline potential expanded uses in the field of river ice control.

Inflatable dam applications include headgates for irrigation, water supply and hydropower, flashboard replacement, raising the crest of an existing dam or reservoir spillway, tidal barriers, sewage treatment lagoons, sediment discharge gates, and groundwater recharging. Although inflatable dams have many advantages in ice-affected waters, none have been built for the specific purpose of ice control.

An inflatable dam consists of an air-filled tube clamped to a concrete sill (Fig. 1). The tube is made of a laminated rubber and nylon material that ranges in thickness from 10 to 25 mm, depending on the height of the dam. Inflatable dams range in height from about 0.4 to 4.6 m (1.3 to 15 ft) and the individual span lengths range from about 6 to 89 m (20 to 290 ft). The structures are best suited to situations where the width-to-length ratio is relatively high, typically greater than five.

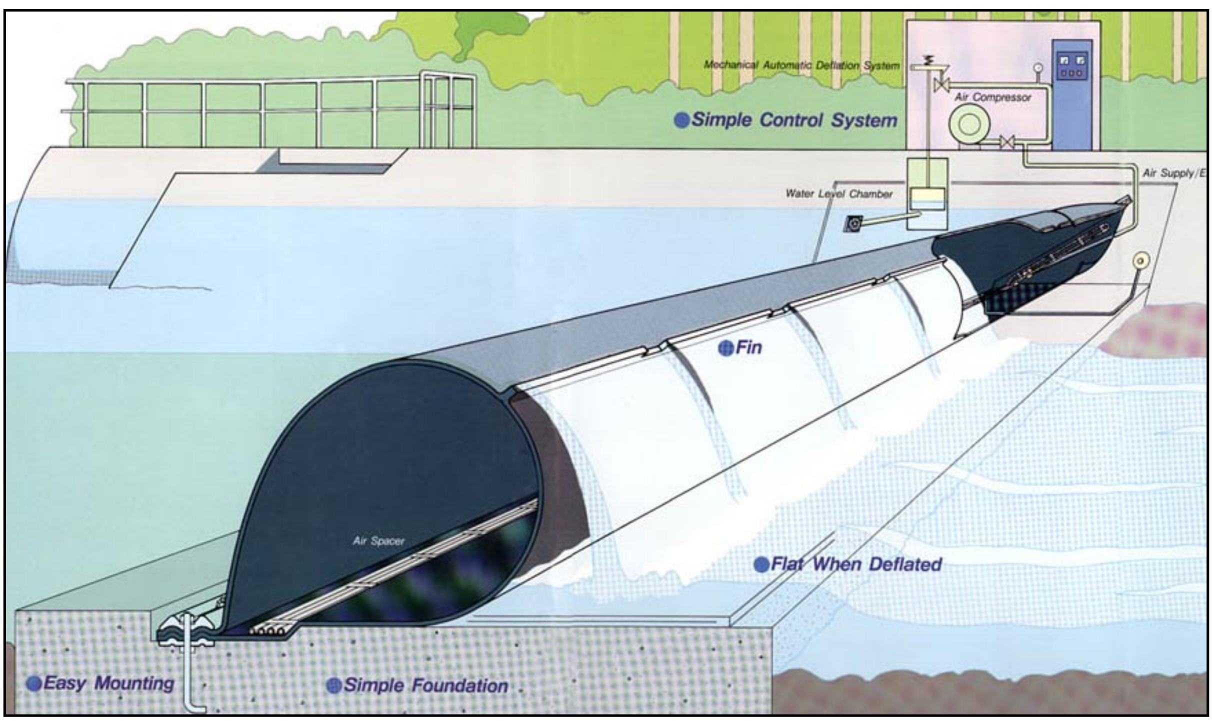


Figure 1. Cross section of an inflatable dam.
(Diagram courtesy of www.bridgestoneindustrial.com/RubberDam/design.htm.)

[1] Personal communication, Roger Campbell, Bridgestone Industrial Products America, Inc., New York, 18 October 1999.

The dams are inflated by high-volume, low-pressure compressors called "blowers" and are emptied through exhaust valves. The dam can be deflated to pass flood flows, to drain the pool, or to bypass flow during turbine shutdowns. An automatic control system operates the blowers and exhaust valves to maintain a set pool elevation or a set air pressure inside the dam. The blowers and exhaust valves also can be operated manually. When fully deflated, the rubber tube lies flat on the concrete foundation. Internal air pressure is low, varying from about 7 to 55 KPa gage (1 to 8 psi), depending on dam height. Side bulkheads and intermediate piers can be either vertical or sloping to conform to the natural side slope of the river.

The first inflatable dam, an Imbertson Fabridam composed of rubberized canvas, was built in California in 1956 and was manufactured by Firestone. Some of these early structures were inflated by water rather than air. Because they did not lie completely flat when deflated, the dams would oscillate with the river current and abrade against the concrete sill, eventually developing holes. In the 1970s, Bridgestone developed an air-inflated dam made of a tougher ethylene propylene diene monomer (EPDM) rubber compound. The dam laid flat on the foundation when deflated, avoiding abrasion. A fin on the downstream side of the dam provided nappe separation, preventing oscillation-induced vibration of the dam when inflated.

Advantages of inflatable dams

Advantages of inflatable over conventional concrete dams with metal gates include a lower initial cost and lower maintenance costs due to the lack of gate mechanisms and the need to paint. Because the sills for inflatable dams can be constructed to conform to the existing channel, the dam's environmental impact when deflated is minimal. Depending on sill geometry and water level, fish passage may be possible over the deflated dam. This low profile also allows passage of flood flows with a minimal increase in upstream stage. If the concrete sill is low enough, bed load and suspended sediment can also pass over the deflated dam, reducing deposition and loss of storage capacity in the upstream impoundment. Finally, the capability for wide pier spacing and absence of a superstructure optimizes debris and ice passage and improves aesthetics.

The initial cost of an inflatable dam, including its sill, blowers, and control system, is significantly lower than the equivalent steel-gated, concrete structure. As an example, in 1986, the Corps constructed a steel-gated, concrete ice control weir on Oil Creek in Pennsylvania at a total cost of $2.2 million. A similar-sized inflatable dam structure is estimated to cost about $1.5 million.[2]

An inflatable dam has no gate-lifting mechanisms to be maintained or kept from freezing in winter. With the dam fully inflated, there is no seepage through side or bottom seals, as is often the case with conventional steel gates. In the winter, this dry downstream condition eliminates icing of sideseals and the apron; in the summer, weed growth is minimized. Ice that does adhere to the inflated dam easily breaks off when the dam is deflated, even under extremely cold temperatures.

Disadvantages and concerns regarding inflatable dams

Some disadvantages of inflatable dams are a shorter design life, vulnerability to vandalism, and uncertainty due to the newness of the technology. Also, when spilling, a low area or "vee notch" tends to form, concentrating the flow in that region. A final drawback is that most types of inflatable dams are manufactured overseas, making it more difficult for federal agencies such as the Corps of Engineers to purchase the products.

The first Bridgestone inflatable dams came on the market in 1978 with an estimated design life of 30 years. However, two spans of a Bridgestone dam installed in 1986 on the Susquehanna River at Sunbury, Pennsylvania, developed air bubbles in the corners of the outer protective layers and had to be replaced in 2000, at half their estimated design life.[3] A possible reason is that the dam was completely deflated during the winter months and may have been damaged by debris and ice.

A high-powered rifle round will penetrate an inflatable dam, but at the relatively low internal pressures the resulting air loss is slow enough that the blowers can compensate until the bag is repaired. This occurred at the Broadwater Dam on the Missouri River at Townsend, Montana.[4] A Bridgestone dam at a water supply reservoir near Norwich, Connecticut, failed completely during the summer of 1999 when some youths kindled a large campfire on the downstream side of the airbag. Although the 180-hectare (450-acre) reservoir quickly lost 1.5 m (5 ft) of pool, there were no injuries or significant damage to downstream property.

[2] Estimate by Ed Foltyn, Hydraulic Engineer, USACRREL (retired), 1989.
[3] Personal communication, Mary Lorah, Manager, Shikellamy State Park, Sunbury, Pennsylvania, January 2000.
[4] Personal communication, Brian Carroll, Plant Manager, Broadwater Hydroelectric Station, Townsend, Montana, 25 January 2000.

Vee-notch formation while spilling water has potential drawbacks. The first is that concentration of flow downstream of the dam may result in scour and possible foundation damage. Also, with an uneven crest height, it is difficult to estimate the depth of flow over the dam, and the water discharge being spilled.

Table 1. Examples of inflatable dams in ice-affected waters.

Project/ location	Year built	Manufacturer/ dimensions *(m)*	Owner/ operator	Use	Comments	Points of contact
Palmer Falls Hudson River Corinth, NY	1987	Bridgestone 1.83 _ 45.5 1.83 _ 61.6	International Paper	Hydro (50 MW)	Performs well during ice season. Passes ice and debris without problems.	Tom Ucher 518-654-3440
Susquehanna River Sunbury, PA	1984–1988	Bridgestone Six 2.44 _ 88.7 One 2.44 _ 50.6	Pennsylvania State Bureau of Parks	Recreational lake	Replaced Fabridam bags. Deflated all winter. Two bags now leak and must be replaced.	Mary Lorah 570-988-5557
Broadwater Station Missouri River Townsend, MT	1988	Bridgestone Seven 3.4 _ 16.5	Montana Power	Hydro (10 MW)	Performs well during ice season. Passes ice and debris without problems. Small leaks in creases near bulkheads.	Brian Carroll 406-266-3869
Rainbow Falls Missouri River Great Falls, MT	1989	Bridgestone Two 3.5 _ 67.67	Pacific Power and Light	Hydro (35 MW)	Performs well during ice season. Passes ice and debris without problems.	Rich Halverson 406-266-3869
Bolton Falls Winooski River Bolton, VT	About 1990	Bridgestone About 1.5 _ 30	Green Mountain Power	Hydro (8.8 MW)	Withstands severe breakup ice runs without problems.	William Conn 802-864-5731
Highgate Falls, Missisquoi River, Highgate, VT	1992	Bridgestone 4.57 _ 67	Village of Swanton, VT	Hydro (9.8 MW)	Highest inflatable dam in the world. Excellent performance in ice. Eliminated freezeup and breakup ice problems at project.	Alan Mosher 802-868-4200
Silvian Station Mississippi River Brainerd, MN	1992	Bridgestone 1.3 _ 6.1	Minnesota Power	Hydro (2 MW)	Performs well in extreme cold. Solved downstream icing and weed problems.	Dave Nixon 218-722-5642
Stoney Brook Reservoir Norwich, CT	1996	Bridgestone 1.53 _ 15	City of Norwich, Department of Public Utilities	Increases spillway crest height of reservoir	Fully inflated except during floods. Failed in 1999 as a result of vandalism.	John Bilda 860-823-4192

Inflatable dam applications in ice-affected waters

The main use of inflatable dams in ice-affected waters has been for small run-of-the-river hydroelectric plants at sites in the northern United States. The inflatable dams are often installed to replace older flashboard systems, or to increase the depth of an impoundment and provide crest control. At these facilities, the dam is fully inflated most of the time while pool elevation is controlled by turbine settings.

In the event of a large runoff event or a turbine shutdown, pool elevation is regulated by changes in air pressure and the height of the dam. During spring ice breakup, it may be possible to lower the inflatable dam sufficiently to avoid a large upstream water level rise and maintain an intact sheet ice cover. Otherwise, inflatable dams perform well at passing ice and debris. Most operators agree that passing debris with sharp steel protrusions—old refrigerators, bridge planks with spikes sticking out, etc.—pose a greater threat to the dam than ice and trees.

Table 1 lists eight examples of inflatable dams in the northern United States. All but two are at hydroelectric projects. Reportedly, many small hydroelectric projects in Canada are also switching to inflatable dams for crest control or flashboard replacement.[5] To illustrate performance in ice conditions, several of the projects listed in Table 1 are described in greater detail below.

Broadwater Dam, Missouri River, Townsend, Montana

Broadwater Station is a run-of-the-river hydro plant, with a head of 6.7 m (22 ft) and a capacity of 10 MW. Under normal flow conditions of 110–170 m^3/s (4000–6000 cfs), all discharge goes through the turbines. In 1988, the flashboards were replaced with seven 3.4-m _ 16.5-m (11-ft _ 54-ft) Bridgestone inflatable dams between vertical-sided concrete bulkheads (Fig. 2). In the event of a turbine shutdown, all flow passes over the dam. Except during high-flow periods, the air bag inflation/deflation system and turbines are operated to maintain a constant pool level year-round.

Typical winter discharge is fairly constant at about 110 m^3/s (4000 cfs), and the sheet ice on the pool can reach thicknesses in excess of 0.6 m (2 ft). Ice breakup usually occurs over a two-day period in late February, during which time the river flow usually increases 50% to around 170 m^3/s (6000 cfs). During breakup, large ice floes, trees, pieces of washed-out bridges, and telephone poles (wires and all) pass over the air bags on the dam. Long pieces of debris sometimes lodge between the concrete piers and must be dislodged using long poles, or cut in half with chainsaws. The ice-out is usually followed by a rainy season with higher open water flows. During the maximum flow experienced since installation, 1080 m^3/s (38,000 cfs), the air bags were completely deflated.

Some of the air bags leak in the crease areas near the concrete bulkheads. Also, leaks in the upstream and downstream faces of one airbag resulting from a high-powered rifle round were mended using a tubeless tire repair kit. Brian Carroll, who has been at the project since the installation of the inflatable dam, is very pleased with its performance.

Figure 2. Ice and debris passing over Broadwater Dam on the Missouri River at Townsend, Montana. Note the vee notch in the airbag near the land-side bulkhead.

5 Personal communication, William Conn, Green Mountain Power, Burlington, Vermont, 25 January 2000.

Highgate Falls Power Dam, Swanton, Vermont

A 4.6-m-high by 67-m-long (15-ft by 220-ft) Bridgestone inflatable dam regulates pool elevation at a 9.8-MW hydroelectric plant owned by the village of Swanton, Vermont, on the Missisquoi River at Highgate Falls. Figure 3 shows the dam in its fully inflated configuration. Plant Manager Alan Mosher describes the new dam as a godsend in terms of reducing ice problems.

The 1992 construction of the Highgate Falls inflatable dam, on a 5-ft-high concrete sill, raised pond elevation by a total of 6.1 m (20 ft). It is one of the highest in existence, and it is possible to walk inside for inspection and repair purposes. When fully inflated, the inside air pressure is 52 KPa gage (7.5 psi). The dam is constructed of 18-ply rubber about 25 mm (1 in.) thick.

Figure 3. Highgate Falls Dam, fully inflated, 21 January 2000.

Although vandalism has not been a problem to date, Mosher believes that it would be difficult for anyone to cause a catastrophic failure. Bullet holes would result in slow leaks that could be easily repaired from the inside. In a test, a 30.06 steel jacket bullet went through a sample piece of the air bag material, but 22 caliber and 30.06 soft point bullets failed to penetrate. Mr. Mosher recollects that the cost of the inflatable dam and associated equipment was about $1.2 million.

The higher pool allowed the area of the hydroelectric intakes area to be doubled, reducing water velocity near the trash racks. The frazil ice blockage problems that existed before the raising of the pool were solved, and debris collection problems minimized. Breakup ice runs passed over the old dam, often destroying the wooden flashboards. Setting and maintaining the flashboards was time-consuming and, to some extent, risky. With the inflatable dam, flashboards are no longer necessary. Before the inflatable dam was installed, breakup ice runs were a problem. In addition to damaging the flashboards the hydroelectric plant downstream of the dam was inundated in 1979 as the result of a breakup ice jam. In March 1992 a severe breakup ice run damaged the structural steel in the footings to the new dam while under construction. With the inflatable dam, it is possible to minimize stage rise in the head pond during breakup by progressively lowering the crest height of the dam as river flow increases. This practice helps preserve the intact sheet ice cover, often allowing it to melt in place rather than break up and move downstream as in the past. Mosher has witnessed ice blocks as thick as 1.2 m (4 ft) passing over the dam crest. He said that at times large ice pieces or floes will hang up on the dam, requiring a temporary lowering of the crest height to get the ice moving again.

Silvian Hydro Station, Minnesota Power, Mississippi River near Brainerd, Minnesota

A 1.3-m _ 6.1-m (4.3-ft _ 20-ft) Bridgestone inflatable dam was installed about nine years ago at this 2-MW run-of-the-river hydro project. Operators are very happy with the new dam's performance to date. The inflatable dam solved the constant leakage of the previous flashboard system, eliminating icing problems in the winter and weed buildup in summer. The concrete apron is now completely dry when the dam is up. The dam is fully inflated except for when the turbines go offline. The dam has been lowered in the dead of winter as a test, and the rubber surface easily broke free of the 1.2-m- (4-ft-) thick ice cover on the pool.

Although the dam's width-to-height ratio is five, which is near the maximum for an inflatable dam, it has worked well to date. There have been no leakage problems at the edges (perhaps because the bag is not deflated that often). The dam's cost, including some rehabilitation to the concrete piers, was $60,000. Minnesota Power would like to replace flashboards with inflatable dams at many more of its sites, but is forced to move slowly because of the cost. Dave Nixon in engineering mentioned that Obermeyer gate systems, which use air bladders to lift hinged steel gates, are also being considered. The Obermeyer gates fit into narrower bays, but have the side leakage and icing problems of conventional mechanical gates.

Conclusions

Based on survey results, inflatable dams perform well in ice-affected rivers and they have a number of advantages over conventional, mechanically operated gates. To date, the primary use of inflatable dams in the northern United States and Canada has been for flashboard replacement and crest control at small hydroelectric projects. In addition to the advantages of lower initial cost and minimal moving parts, inflatable dams appear to be well adapted to winter operations. Ice adhesion, seal leakage and freezing, as well as ice and debris passage, have not been problems.

Inflatable dams can be operated to maintain a constant pool level, delaying or preventing breakup of the upstream ice cover and protecting downstream locations from ice jam flooding. On steeper streams and rivers, low-profile inflatable dams would be ideal for creating shallow pools or series of pools to speed ice cover formation and reduce frazil production and subsequent freezeup or breakup ice jam flooding. The airbags would rest deflated on the riverbed when unneeded, allowing fish migration and natural movement of sediment.

Finally, inflatable dams might provide an attractive option in the recent trend towards removal of dams to return rivers to their natural condition. In most cases, the lowering or removal of a dam changes the ice regime on a river with possibly negative effects. For example, frazil ice that once collected behind the dam might move downstream to form a freezeup ice jam and flooding at an undesirable location. As insurance against unforeseen ice problems associated with the dam removal or lowering, the project could incorporate the construction of a sill to accept an inflatable dam if needed.

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Appendix C

BENDWAY WEIR DESIGN GUIDANCE

REVERSE SILL (BENDWAY WEIRS) GENERAL GUIDANCE

The information contained herein is based on a cursory review of current construction practices of "Bendway Weirs" and "Bank Barbs" by the professionals cited in the section entitled "Acknowledgements" and is provided for general guidance only. The term "Reverse Sill" will be used in this guidance to refer to the type of structures known as either a Bendway Weir or Bank Barb. The formulas provided were developed to consolidate the many "rules of thumb" that currently exist in the field. The formulas are not based on exhaustive research but appear to match well to current practices. The formulas should be used by a qualified designer as general guidance and not as an absolute answer.

(a) DESIGN CONCEPT

Reverse sills are similar to stone jetties in plan view appearance but have significant functional differences. Jetties are generally visible above the flow line and are designed to move the river flows around the structure. Reverse sills are normally not visible because they are below the water line and are intended to reduce the waters impact on the bankline both upstream and downstream of the structure. Downstream, the flow lines turn as they break perpendicular to the upstream sill surface. The reverse sills also decrease the erosion along the upstream banks by creating a reverse eddy which reduces the secondary currents. The reverse sills are designed to be overtopped during normal and high flows. By their nature the reverse sills are not visible during normal flows and therefore are ideal where esthetics are paramount. Large reverse sills are commonly used for developing and maintaining river channels. Short reverse sills -- also commonly called reverse sills or barbs -- are used for bankline protection.

(b) GENERAL MATERIAL SPECIFICATIONS

(1) Stone should be angular, and not more than 30 percent of the stone should have a length exceeding 2.5 times its thickness.

(2) No stone should be longer than 3.5 times its thickness.

(3) Stone should be well graded but with only a limited amount of material less than half the median stone size. This is because the stone will most often be placed in moving water and the smaller stone will be displaced during placement..

(4) Construction material should be quarry-run stone or broken, clean concrete.

(5) Material sizing should be based on standard riprap sizing formulas for turbulent flow. Typically, the size should be approximately 20-percent greater than that computed from nonturbulent riprap sizing formulas. The most current sizing formulas are based on river depth and velocity. Results should typically be between 1-foot and 3 feet and should be in the 150- to 3,500- pound weight range.

(6) Broken concrete should be derived from material previously designed for ground exposure with thickness greater than 6 inches. No broken concrete piece should have a maximum dimension greater than 5 times the thickness. No exposed rebar should be present in the material. If proper material is used and the exposed key covered with soil and seeded, broken concrete is acceptable by the environmental community in many states.

(c) GENERAL DESIGN GUIDANCE

This section requires the use of sound engineering judgement.

(1) HEIGHT - The height of the sills is determined by analyzing the depth of low flow, high flow, and sedimentation bed formation depths at the project site. This can be done in several different ways but it is critical that the flow depths be well established. The bendway sill height should be above the bed formations which are typically between 30 to 50 percent of the mean annual high water level. The height of the structure should be below the normal or average seasonal water level and can be equal to or below the mean low-water level. Construction should be conducted during low flows. Large sills, longer than 30 feet, are most easily constructed by barge. If a barge is not used, equipment can be driven out on top of the structure. The height of the structure can be over built as a construction platform while construction is moving riverward. The structure can then be lowered as the equipment backs off the completed structure. Plate 1 shows the design height guidance.

(2) ANGLE - Reverse sills are designed to be flat or nearly flat with the structure projected upstream 15 to 30 degrees, 60 to 75 degrees from the bankline and/or flow lines. The angle of projection is determined by the location of the sill in the bend, the angle at which the flow lines approach the structure, and the angle required to turn the flow towards the middle of the downstream channel. Ideally, the angle should be such that the high-flow streamline's angle of attack is not greater than 30 degrees and the low flow streamline's angle of attack is not less than 15 degrees. If the angle required to divert the flow towards the middle of the downstream channel is greater than 30 degrees, then the angle should be such that the perpendicular line from the midpoint of an upstream sill points to the midpoint of the following downstream sill. Plate 2 shows a typical plan view for a system of reverse sills.

(3) CROSS SECTION - The flat sill section transitions into the bank on a slope of 1V:1.5H to 1V:2H. The structure height at the bankline is the height of the maximum design high-water level. This level is chosen based on sound engineering judgment. The key must be high enough to prevent water from flowing around it and flanking the structure.

(4) LENGTH - The **reverse sill length (L)** should not exceed 1/3rd the **mean channel width (W)**. Sills with lengths at or greater than one-third of the width of the channel tend to alter the channel flow and meander patterns which can impact the opposite bankline. Sills designed for bank stabilization need not exceed one-fourth the channel width and can be significantly shorter. The length of the sill will, however, impact the spacing between the sills. In circumstances where bed degradation is also a concern, the sill may cross the entire channel.

Maximum L = W/3 Typically W/10< L < W/4

(5) LOCATION - Reverse sill location guidance is shown on Plate 2. Ideally, a short sill should be placed a **distance (S)** upstream from the location where the **midstream tangent flow line (midstream flow line located at the start of the curve) intersects the bankline (PI)**. The following reverse sills are then located based on the site conditions using sound engineering judgment. Typically, the sills are evenly **spaced a distance (S)** apart.

(6) SPACING - Reverse sill spacing is influenced by several site conditions which include soil type, bank stability, vegetation, upper bank use and climate. The following guidance is based on a cursory review of the tests completed by the Waterways Experiment Station (WES) on reverse sills and on tests completed by the Missouri River Division (MRD) Mead Hydraulic Laboratory on underwater sills. Based on the review, reverse sills should be spaced similarly to typical hardpoints and jetties. The **length of the sill (L)** influences the spacing. The **spacing (S)** is also influenced by the length of the sill in proportion to the **channel width (W)** and **channel radius of curvature (R)**. Spacing can be computed based on the following guidance formulas where:

S = 1.5L *(R/W)^0.8 * (L/W)^0.3 (LaGrone, D. L. 1995)

Maximum spacing (Smax) is based on the intersection of the tangent flow line with the bankline assuming a simple curve. Note that on very straight reaches where "R" is large the spacing is related to the length of the key and the sin of 20 degrees as discussed later:

Smax = R*(1-(1-L/R)^2)^0.5 (LaGrone, D. L. 1995)

(7) LENGTH of KEY - Reverse sills like all bankline protection structures should be keyed into the bankline. The purpose of the key is to prevent the water from getting behind the structure and flanking it. The **length of the key (LK)** should be based on the **length of the sill (L)**; the **channel width (W)**; the **spacing between the sills (S)**, the **channel radius of curvature (R) and bank height**. Typically the key length is about half the length of short sills and about one-fifth the length for long sills. Test results conducted by the MRD Mead Hydraulic Laboratory found that the **lateral erosion (E)** between jetties on nearly straight reaches could be estimated by using a 20-degree angle of erosion expansion. The following guidance formulas for **LK** were therefore developed. These formulas compute minimum **LK** and should be extended in critical locations.

When the channel radius of curvature is large and $S > L/\sin 20$

$LK = E - L$ where $E = S \sin 20$ (LaGrone, D. L. 1995)

When the channel radius of curvature is small and $S < L/\sin 20$

$LK = L/2 * (W/L)^{0.3} * (S/R)^{0.5}$ (LaGrone, D. L. 1995)

NOTE: LK should neither be less than 15 feet nor less than 1.5 times the total bank height.

(8) TOP WIDTH - The top width of the sill may vary between 3 feet and 10 feet with side slopes no steeper than 1V:1.5H. Sills over 30 feet in length will have to be built either from a barge or by driving equipment out on the structure during low flows. Structures built by driving equipment on them will need to be at least 10 to 15 feet wide.

(9) NUMBER OF SILLS - The fewest number of sills necessary to accomplish project purpose should be constructed. It is recommended that, not less than three sills be used together in an eroding bend. The length of the sills and the spacing can be adjusted to meet this requirement. On very straight reaches one or two sills may be used.

(10) CONSTRUCTION - Construction of the reverse sills should be conducted during the lowest flow period for the affected river. Construction methods will vary depending on the size of the river. Construction on larger rivers may be conducted using a barge which would allow the rock to be placed without disturbing the bankline. Again, to restate a point made earlier, for rivers where a barge is not available and where the reverse sills will be longer than 30 feet, access will need to be made from the bank and equipment may need to be driven out on the sill as it is being constructed. This will require a wider sill than typically necessary. A view of this type of construction is shown in Plates 3 and 4. On small streams construction may be completed by using small equipment.

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