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In This Issue . . .

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This *Water Operation and Maintenance Bulletin* is published quarterly for the benefit of water supply system operators. Its principal purpose is to serve as a medium to exchange information for use by Bureau of Reclamation personnel and water user groups in operating and maintaining project facilities.

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***Cover photograph** – The downstream side of the spillway radial gates at Choke Canyon Dam.*

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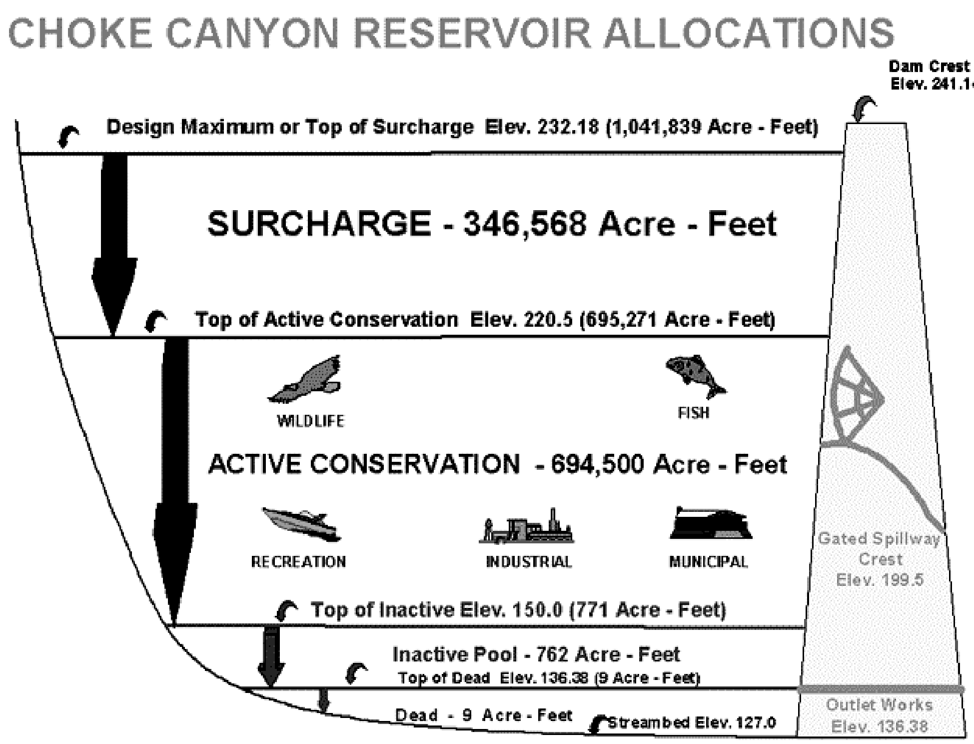
Choke Canyon Surcharge Operations Modeling Spreadsheet

by Ben Claggett[1]

Introduction

Choke Canyon, not to be confused with the 1986 United Films movie, *Choke Canyon*, is a Bureau of Reclamation (Reclamation) dam in the heart of southern Texas. This facility is a "transferred work," with operations and maintenance performed by the City of Corpus Christi (City). The 26,000-acre reservoir is fed by the Frio and San Miguel Rivers in the Nueces River Basin and is home to the American alligator, alligator gar, and other fish species.

Unlike most Reclamation reservoirs, Choke Canyon does not have an authorized flood control component. This means that while operations in the "active conservation pool" are at the discretion of the City, when the reservoir water surface rises above elevation 220.50 feet, operational decisions are the responsibility of Reclamation's Oklahoma-Texas Area Office (OTAO).



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Choke Canyon Dam has a gated spillway with seven 50- by 24-foot radial gates that can release up to 258,000 cubic feet per second (cfs) at maximum water surface.

The upstream side of the gated spillway at Choke Canyon Dam.

The downstream side of the spillway radial gates at Choke Canyon Dam.

The river channel downstream of the Choke Canyon Dam spillway.

The June 2005 article, *Modified Surcharge Operating Criteria,* described recent surcharge events and the corresponding development of enhanced surcharge operating procedures. The article stated that this reservoir was normally low until July 2002 when the reservoir rose into “surcharge pool” (above elevation 220.50 feet). Since 2002, about 15 significant rainfall events have initiated surcharge releases.

Also mentioned in the June 2005 article, the OTAO has developed a spreadsheet that is used in making surcharge release decisions at Choke Canyon Dam. The purpose of *this* article is to explain, in more detail, how the spreadsheet functions and how it assists the OTAO in the decisionmaking process.

Reservoir Modeling Spreadsheet—The “Heart of the Machine”

The *surcharge operations modeling spreadsheet,* built in Microsoft Excel, simulates theoretical future reservoir conditions based on user-entered predicted inflows and releases. More specifically, the program calculates predicted reservoir water surface (RWS) elevations at specific times in the future. Because the spreadsheet performs the calculations in a fraction of the time needed to do them by hand, we have the flexibility to quickly model various scenarios and evaluate the resulting reservoir responses. This, of course, makes the engineers happy since they had previously spent lots of time performing multiple hand calculations and wearing out their calculators. The spreadsheet is no "crystal ball"—it by no means gives all the answers. However, it is a useful mathematical model of the reservoir and a valuable tool in the decisionmaker's "tool belt."

It is important to note that this modeling method utilizes predicted riverflow data (such as from the River Forecast Center) along with real-time riverflow and reservoir elevation data. With this information, the spreadsheet is designed to model an event chronologically in 6-hour increments. Given a set of 6-hour upstream riverflow predictions in cfs, a set of 6-hour gate release rates in cfs, and a starting RWS elevation, the program produces a corresponding set of predicted RWS elevations.

How it Works

The "heart" of the spreadsheet is based on the simple "volume balance" principle:

Initial Volume Stored + Predicted Volume In – Predicted Volume Out = Predicted Final Volume Stored

The initial volume is established by entering the initial RWS elevation. The program then searches the reservoir's area-capacity table to determine the corresponding volume (in acre-feet) of water stored in the reservoir at that initial reservoir elevation. This volume is then saved as the *Initial Volume Stored*, and it represents the volume at the end of that first 6-hour period as well as the volume at the beginning of the second 6-hour period (see example below).

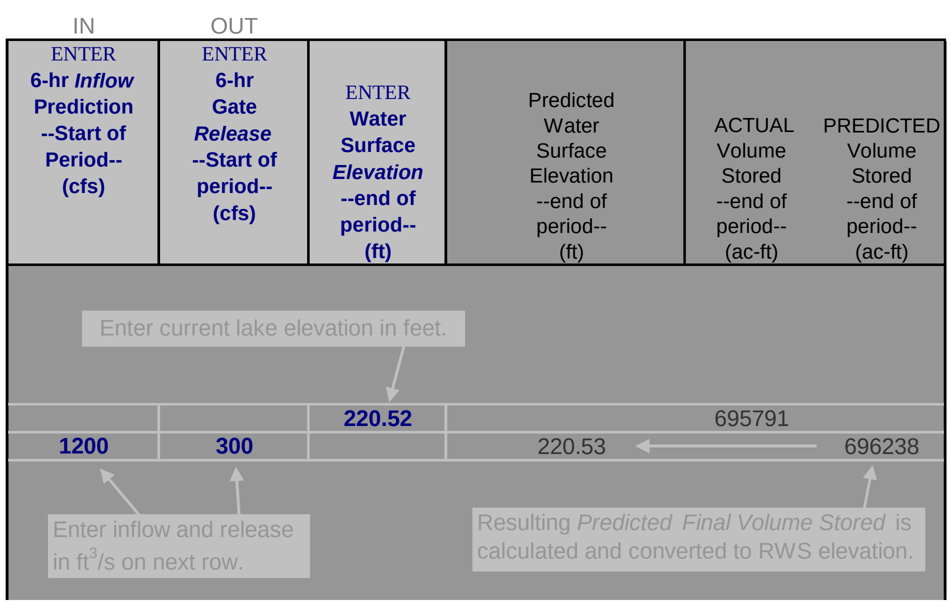
IN	OUT				
ENTER **6-hr *Inflow* Prediction --Start of Period-- (cfs)**	ENTER **6-hr Gate *Release* --Start of period-- (cfs)**	ENTER **Water Surface *Elevation* --end of period-- (ft)**	Predicted Water Surface Elevation --end of period-- (ft)	ACTUAL Volume Stored --end of period-- (ac-ft)	PREDICTED Volume Stored --end of period-- (ac-ft)
		Enter current lake elevation in feet.			
		220.52		695791	

The column headings from a portion of "the spreadsheet." The columns showing the conversions from cfs to acre-feet and vice versa are hidden from the user to simplify the user interface. The area-capacity table is also hidden from the user.

Then, on the next row, representing the second 6-hour period, the "6-hr Inflow Prediction" is entered in cfs. This inflow is assumed to remain constant for the entire 6-hour period, and the program automatically calculates the volume for this flow rate. For example, a constant flow rate of 50 cfs for 6 hours is 1,080,000 cubic feet, or approximately 25 acre-feet of volume. The calculated volume in acre-feet is saved within the program as the *Predicted Volume In*.

Next, the proposed rate of water to be released during that same 6-hour period is entered by the user in cfs. This value can be changed later in order to control the theoretical reservoir response. The flow rate is converted into a volume (acre-feet), and this value is saved within the program as the *Predicted Volume Out*.

Now, the program has all the information it needs to calculate the RWS elevation that results from these predicted changes in volume. The program takes the *Initial Volume Stored*, adds the *Predicted Volume In*, subtracts the *Predicted Volume Out*, and calculates the *Predicted Final Volume Stored*, all in acre-feet. The result is automatically converted from acre-feet back into RWS elevation via the area-capacity table (see example below).



Example input and the corresponding calculations for one 6-hour period of time.

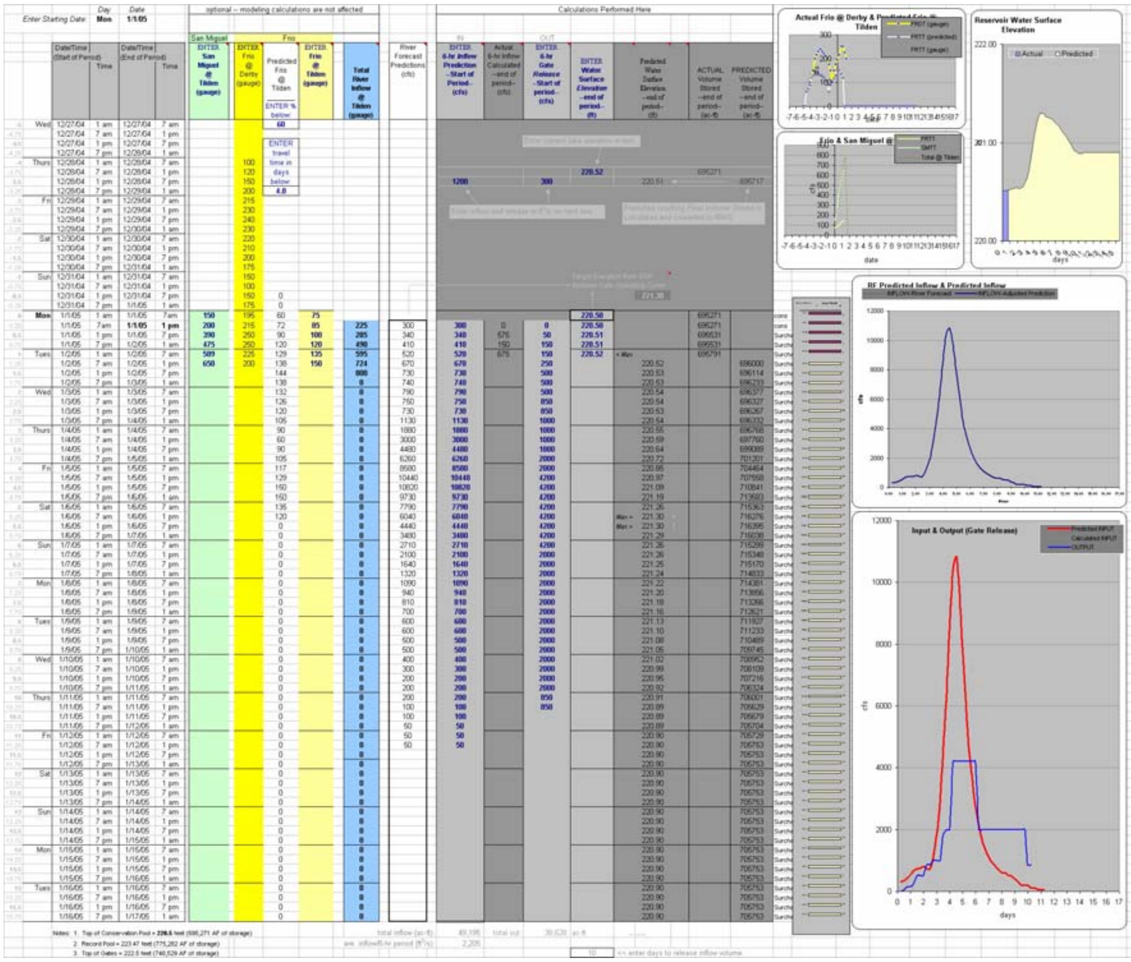
This explains the basic operation of the spreadsheet. If a series of predicted reservoir inflows are entered in the *Inflow* column and, likewise, several proposed gate releases are entered in the *Release* column, a corresponding set of predicted RWS elevations are calculated, giving us a model by which proposed gate releases can be manipulated until an acceptable reservoir response is determined (i.e., maximum RWS elevation remains below a desired level). As the event occurs, actual measured inflows, gate releases, and RWS elevations are entered in place of the previously entered predictions. The spreadsheet automatically calculates the future predicted RWS elevations based on the most recent actual RWS elevation.

It is important to realize that for any 6-hour period (a row in the spreadsheet), the flow rates represent a constant flow while the RWS elevation represents an instantaneous value at the end of the 6-hour period. In reality, the inflow rate and RWS elevation do not remain constant and then suddenly change in 6-hour increments; they are variable. Therefore, the spreadsheet has some amount of error inherit in its design, but this error is considered to be relatively insignificant. If desired, the spreadsheet can be made more accurate by breaking down the time into 3-hour or even 1-hour intervals. Also, it is recognized that other sources of inflow and water losses often exist. In our case, we consider those additional sources, such as evaporation and seepage, to be insignificant related to the rainfall and the magnitude of releases made during surcharge events. However, if so desired, these or other known contributions to or from the reservoir could simply be added to the inflows and/or gate releases and the sum entered in the appropriate *Inflow* or *Release* columns. Another option would be to modify the spreadsheet to contain an additional column for entering other inflows or losses.

Reservoir Modeling Spreadsheet—“Bells and Whistles” and Other Tools

The current version of the spreadsheet includes several “accessories” that enhance its effectiveness. The first set of columns in the following image contains the dates and times divided into 6-hour periods. Then, the next set of columns contains a log of real-time upstream riverflows that are entered manually as the event progresses. This provides a convenient “picture” of the riverflows upstream and, if needed, can be used for adjusting the River Forecast predictions. Next is a single column for entering the River Forecast Center predictions of inflow into Choke Canyon Reservoir. Then, in the center is the “heart” of the spreadsheet where the calculations are performed as described above. The graphs to the right help the user visualize the model.

The spreadsheet also indicates whether the reservoir elevation is in the conservation pool or surcharge pool. This feature also alerts the user if any of the predicted reservoir elevations exceed the maximum surcharge pool elevation or the dam crest with bright yellow or red colors, respectively. Another display always points to the maximum RWS elevation for the event, and another indicator informs the user if any of the predicted RWS elevations exceed the current “target elevation,” which is discussed in the next section.



Picture of the user interface of the Choke Canyon Reservoir Surcharge Operations Modeling Spreadsheet.

Reservoir Modeling Spreadsheet—Target Elevation

As mentioned in the June 2005 article, *Modified Surcharge Operating Criteria*, the "Spillway Gate Operating Curve" originally developed for spillway gate operations at Choke Canyon Dam was designed to pass the Probable Maximum Flood. This operating curve directs the user to open the spillway gates to specific gate openings based exclusively on the observed reservoir elevation. Today, we have access to real-time upstream river basin information and, therefore, this operating curve requires more aggressive releases than what is necessary for the vast majority of inflow events. However, the maximum RWS elevation that would be reached, if releases were to follow this operating curve, is considered a good criterion for future surcharge operations. With this is mind, we are able to consider the upstream riverflows in advance of their reaching the reservoir and we can initiate gate changes earlier. Then, with the use of the *surcharge operations* *modeling spreadsheet*, we can plan a more steady release schedule that results in a lower maximum release and yet prevents the reservoir level from exceeding the "target elevation."

The "target elevation" is the maximum RWS elevation that would be reached if the original operating curve were to be followed. This number is determined in the spreadsheet with the "target elevation calculator." This calculator, not visible to the user unless the respective worksheet is unhidden, automatically simulates an inflow event based on the initial RWS elevation and the values entered in the "River Forecast Predictions" inflow column. The releases are calculated according to the original spillway operating curve, and a resulting reservoir response is generated based on the volume balance method described earlier. The "target elevation" is then determined by simply identifying the maximum RWS elevation that results from this simulation. In other words, the reservoir response is "pre-modeled" as if releases would be made according to the release schedule of the original spillway curve, and the resulting maximum RWS elevation is acknowledged. The spreadsheet runs this model separately from the "primary" model discussed earlier.

On the user interface page of the spreadsheet, the "target elevation" is displayed so that proposed gate releases can be adjusted to result in a maximum predicted RWS elevation that does not exceed the "target elevation." However, as the event occurs, the measured inflows (instead of the predicted), as well as the actual RWS elevations, are entered, and the spreadsheet automatically fine-tunes the target elevation. The reservoir and hydrologic conditions should be evaluated, and the spreadsheet updated (at least once a day), with spillway gate changes made as necessary.

Summary

We have found the *surcharge operations modeling spreadsheet* to be a versatile means to compare potential gate releases and to have instant perspective into reservoir response. Perhaps the greatest benefit of using this spreadsheet is its ability to perform the volume balance calculations quickly, thus permitting decisionmakers to focus more on the other aspects of release decisions. Reservoir operations involve a delicate balance of technical and political factors and often impact numerous individuals and organizations. It is assumed other reservoir operators, who don't already have some similar tool, could benefit from creating their own spreadsheet specifically designed for their facility. This kind of tool requires an initial investment of time but pays returns during critical moments of decision by saving time, increasing our confidence, and by helping establish consistent operating criteria.

Mason Dam Flow Deflectors for Preventing Abrasion Damage

Purpose

A physical model study was conducted to evaluate the hydraulic characteristics of the Mason Dam river outlet works stilling basin and to design a flow deflector for the purpose of mitigating basin abrasion damage. In addition, the first prototype flow deflector design was implemented at Mason Dam in October 2002 to provide a field demonstration of this technology. Field monitoring, including dive inspections and velocity profile measurements, were included in this study to verify the effectiveness of the deflector and to refine the final design.

Introduction

Stilling basin abrasion damage is a widespread problem for river outlet works at dam sites throughout the United States. Abrasion damage occurs when materials such as sand, gravel, or rock are carried into the basin by a recirculating flow pattern produced over the basin end sill during normal operation of a hydraulic jump energy dissipation basin (figure 1). Once materials are in the basin, turbulent flow continually moves the materials against the concrete surface, causing severe damage, often to the extent that reinforcing bars are exposed. Then, when repairs are made, many basins experience the same damage again within one or two operating seasons. Research conducted by the Bureau of Reclamation's (Reclamation) Water Resources Research Laboratory (WRRL) in Denver, Colorado, has demonstrated that the installation of flow deflectors can improve flow distribution, thus minimizing or eliminating the potential for materials to be carried into stilling basins (figure 2). This can increase the life of the basins and reduce necessary repairs.

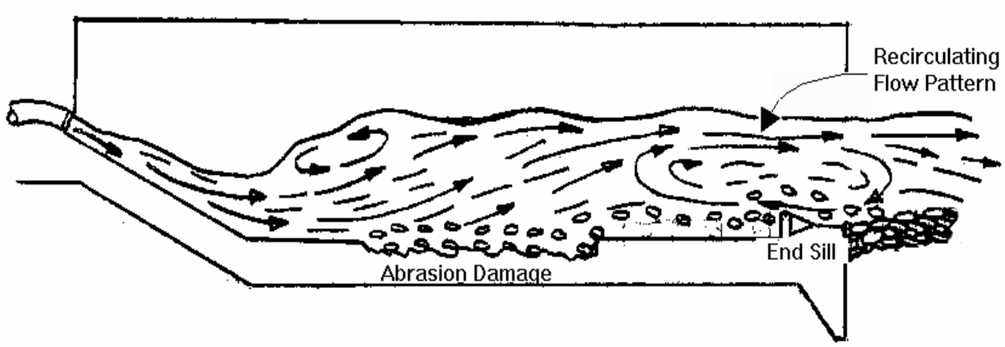


Figure 1.—A recirculating flow pattern is produced over the basin end sill during normal operations.

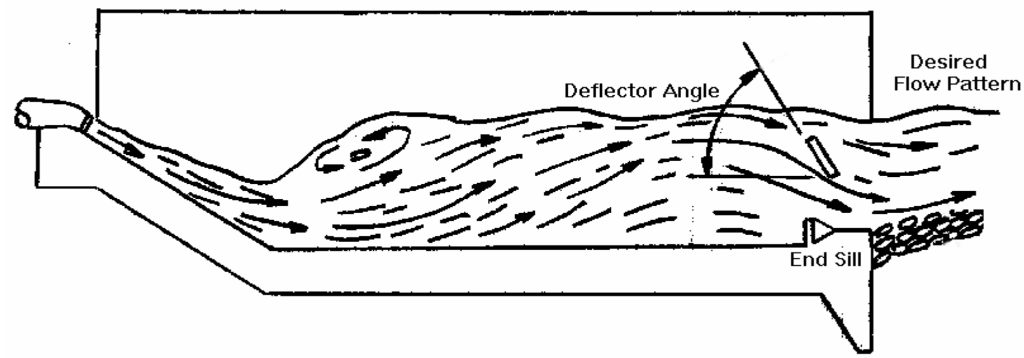


Figure 2.—Desired flow pattern with flow deflector installed.

The Mason Dam outlet works stilling basin, a typical Reclamation Type II basin with a long history of abrasion damage and repeated repairs, was determined to be an excellent candidate for a field demonstration of this technology. Mason Dam is located on the Powder River in Baker County, Oregon, approximately 17 miles southwest of the city of Baker. The dam was constructed for irrigation and for maintaining minimum flow in the Powder River. Flood control benefits are also provided for areas downstream from the dam. The dam is a 173-foot-high zoned earthfill embankment with a crest length of 895 feet. The dam forms a reservoir 4.5 miles long covering 1,962 surface acres. The tunnel outlet works and an ungated spillway are located on the left abutment. Reclamation owns Mason Dam; however, the Baker Valley Irrigation District (BVID) operates and maintains the facility under contract with Reclamation.

A physical model, constructed in the WRRL, was used to design a flow deflector for the Mason Dam outlet works stilling basin. In addition, a field evaluation was conducted after the prototype deflector was installed to verify the effectiveness of the design and to develop methodology for widespread application (a patent is pending on this technology).

Conclusions

Model Evaluation

1. Results from model investigations indicate that the installation of a flow deflector in the stilling basin can help improve flow conditions to minimize the potential for carrying materials into the basin, thereby extending basin life and reducing long-term operation and maintenance (O&M) costs.

2. Model investigations were used to design an effective flow deflector for discharges up to the maximum downstream river channel capacity of 500 cubic feet per second (ft^3/s), the maximum discharge allowed by Standing Operating Procedures (SOP).

3. The investigations determined that the optimal deflector design was a 5-foot-high deflector positioned 5 feet upstream from the end of the basin at elevation 3900 feet (referenced to the upstream lower edge of the deflector) and angled at 90 degrees (vertical).

4. The 5-foot-high deflector spanning the 17-foot-wide basin produced better performance than a 3- or 4-foot-high deflector. However, performance was acceptable for all three configurations.

5. Without a deflector in the basin, the average bottom velocities measured at the end of the basin were predominantly in the upstream direction and ranged in magnitude from -0.4 foot per second (ft/s) to -0.8 ft/s for gate openings ranging from 20 percent to 100 percent (negative values indicate velocities were upstream into the basin). Maximum upstream velocities measured were in the range of -2.0 ft/s to -3.0 ft/s. All dimensions and measurements reported here are scaled to prototype dimensions.

6. With the optimal deflector design in place, average velocities were directed downstream away from the basin. Maximum downstream bottom velocities measured at the end of the basin ranged from 3.0 ft/s to 5.0 ft/s for the range of operations tested. Velocities of this magnitude should not cause any significant erosion downstream from the basin.

7. Model investigations indicated that with a deflector installed in the basin, flow releases ranging from 30 percent to 60 percent gate opening can be used to flush materials from the basin. Without a deflector, releases at 100 percent gate opening (870 ft^3/s) are required to purge materials from the basin. However, since this exceeds the maximum downstream river channel capacity of 500 ft^3/s and SOP requirements, releases at 100 percent gate opening are not normally allowed. Therefore, the basin cannot be flushed on a regular basis without a deflector. The exact size of materials that can be flushed from the basin with the deflector in place will depend on operations and have not yet been determined.

8. The difference in water surface profiles measured along the basin walls, with and without the deflector installed, was negligible.

9. Piezometer taps were used to measure the differential loading across the deflector for model operations up to 100 percent gate opening at maximum reservoir elevation. The maximum force on the prototype deflector due to static hydraulic loading was predicted to be about 12,600 pounds.

Field Evaluation

1. Average vertical velocity profiles measured at Mason Dam at the exit of the basin without a deflector correlated well with the velocities measured in the model, especially those velocities measured near the bottom where air entrainment was minimal. This demonstrated that the physical model provided an accurate representation of prototype conditions.

2. Average velocities measured at the basin exit with the deflector in place correlated well with the model for discharge releases up to 30 percent gate opening. Velocities measured at gate openings greater than 30 percent, with the deflector in place, were inconclusive due to high air concentration in the flow that interfered with data acquisition.

3. The dive team inspecting the basin in August 2004, after two seasons of operations with the deflector in place, found only a few stones in the basin and no indications of abrasion damage. The flaking off of a thin top layer of the new concrete was attributed to other causes. In June 2005, a subsequent dive inspection was conducted and there were still no signs of abrasion damage, thereby indicating the deflector was performing as desired. In addition, divers found no signs of erosion immediately downstream from the end of the basin.

4. The high correlation between model and prototype data indicates that the installation of a deflector in the basin can help improve flow conditions significantly to minimize the potential for entraining materials in the basin, thereby extending basin life and reducing long-term O&M costs.

The Model

A 1:7 geometric scale was used to model the Mason Dam outlet works stilling basin. Froude scale similitude was used to establish the kinematic relationship between model and prototype because hydraulic performance depends predominantly on gravitational and inertial forces. Froude scale similitude produces the following relationships between the model and the prototype:

Length ratio	$L_r = 1{:}7$
Velocity ratio	$V_r = L_r^{1/2} = 1{:}2.65$
Discharge ratio	$Q_r = L_r^{5/2} = 1{:}130$

The physical model was used to investigate hydraulic conditions in the Mason Dam stilling basin and to study the effect of deflector angle and position on flow patterns over the basin end sill (figures 3 and 4).

Figure 3.—Looking through the plexiglass sidewall of the model operating at 40 percent gate opening.

Prototype features modeled included:

1. The two 33-inch by 33-inch high-pressure regulating gates and upstream bifurcation.

2. The 17-foot-wide hydraulic jump twin bay stilling basin with 2:1 sloping chutes and dentated end sill.

3. Approximately 75 feet of topography downstream from the basin, constructed on a 5:1 slope.

Velocities were measured with a SonTek Acoustic Doppler Velocimeter (ADV) probe and were measured at the downstream end of the basin at its centerline. Tailwater elevation was set for each flow condition tested using tailwater data obtained during Mason Dam outlet works operations. The deflector was modeled with a flat section of sheet metal spanning the 17-foot-wide basin and mounted on guides attached to the basin sidewalls to allow vertical movement of the deflector within the basin (figure 4).

Model Study Investigations

Model investigations were conducted to evaluate hydraulic conditions in the stilling basin and downstream apron area for the range of operating conditions expected in the prototype. The actual flow conditions tested are listed in table 1. Both high-pressure regulating gates of the twin bay design were operated symmetrically at all times as required by the SOP. Velocity data and dye streak data were collected and analyzed to define basin performance. These data were

Figure 4.—Looking upstream at stilling basin model with ADV probe and deflector installed near the end of basin.

Table 1.—Prototype flow conditions tested in model

Gate opening (%)	Prototype discharge corresponding to maximum reservoir elevation (ft^3/s)	Tailwater depth (feet)
20	230	18.2
40	420	18.8
60	575	19.5
80	735	20.0
100	870	20.7

used to determine the most effective deflector angle and the best lateral and vertical locations within the basin. Although investigations were conducted up to the maximum possible discharge of 870 ft^3/s (100 percent gate opening at maximum reservoir, elevation 4077 feet), the optimum deflector design was based only on discharges up to 575 ft^3/s (60 percent gate opening at maximum reservoir) because Mason Dam's SOP limits outlet works discharges to the maximum downstream river channel capacity of 500 ft^3/s. Velocities were measured at numerous locations within and downstream from the stilling basin to map out resulting hydraulic flow patterns for each discharge tested. Initial measurements included mapping vertical velocity profiles measured at the downstream end of the stilling basin for gate openings of 20, 40, 60, 80, and 100 percent with discharge based on maximum reservoir (figure 5).

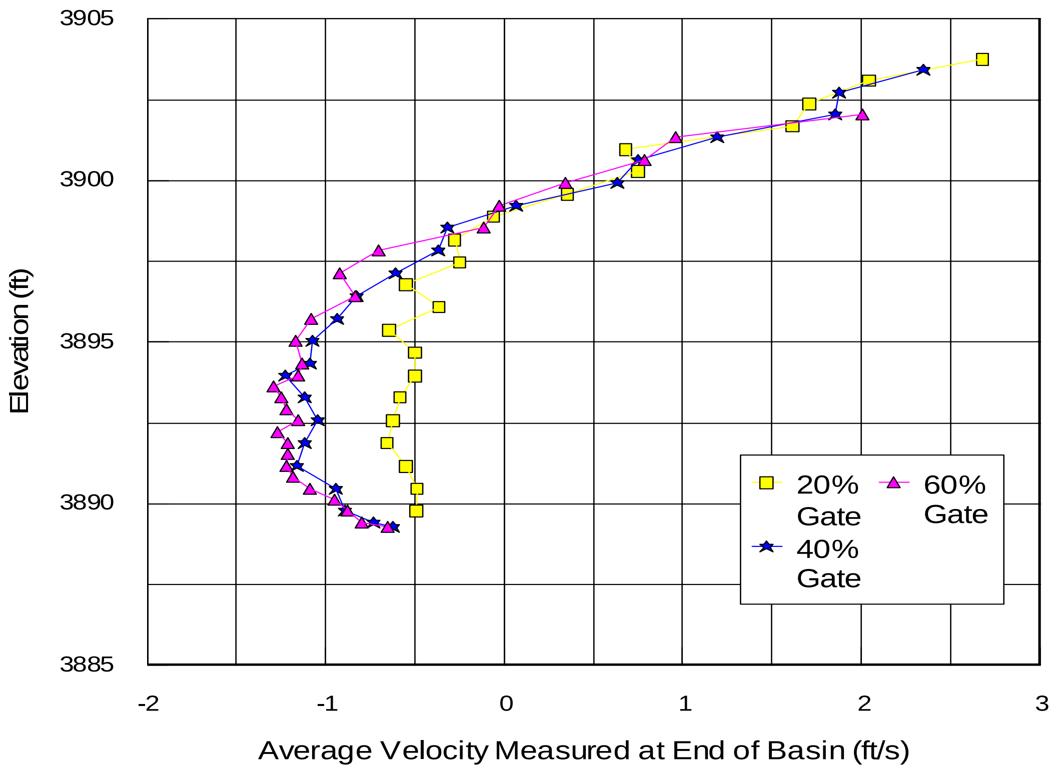


Figure 5.—Vertical velocity profiles measured at the downstream end of basin without a deflector.

Velocities were measured at approximately 0.7 foot vertical increments starting 0.29 foot above the basin invert and continuing until air entrained in the flow prevented further measurements (all dimensions are prototype). Figure 5 demonstrates that average velocities measured within the bottom 9 to 10 feet of the water column are directed upstream into the basin (negative values indicate that average velocity is directed upstream). Early investigations showed that average velocities measured at the end of the basin, at its centerline, and 0.44 foot above the invert elevation, provide a good representation of the bottom velocities that carry materials into the basin. Therefore, velocities measured at this location were used as a basis to determine deflector performance for all subsequent investigations.

In addition, eight piezometer taps were installed equally spaced across the upstream and downstream faces of the deflector. The taps were connected to a manometer board to measure differential loading on the deflector for flow rates up to a maximum discharge of 870 ft^3/s at 100 percent gate opening.

Model Results

Optimal Positioning and Size

Tests were initially conducted at 40 and 60 percent gate openings only because these conditions produced the strongest upstream bottom velocities adjacent to the riprap apron, within the maximum operating range specified by the Mason Dam SOP. Four different parameters were investigated to determine what criteria would produce best deflector performance (all parameters are referenced to the bottom upstream edge of the deflector).

1. *Lateral and Vertical Positioning* – Initial investigations were conducted with a 5-foot-high deflector, angled at 60 degrees, and spanning the width of the basin. Lateral location was defined as the distance from the downstream end of the stilling basin (defined as the downstream end of the basin sidewalls) to the deflector. Lateral locations were varied from 0 to 14 feet. The best position for the deflector laterally along the length of the basin was determined by setting the deflector a specified distance from the end of the basin and then measuring average bottom velocities at the end of the basin. For each lateral position, the deflector was moved in vertical increments so that average bottom velocities could be measured for a range of deflector elevations for each flow condition tested. Deflector elevation was varied from 4 to 15 feet above the elevation of the basin floor (floor elevation 3889 feet).

   Deflector performance was defined by comparing these velocities (i.e., the higher the velocity in the positive direction, the better the performance). Positive values indicated that average velocity was in the downstream direction, away from the basin.

   Figures 6 and 7 show average bottom velocities measured as a function of deflector elevation for each lateral position tested for 40 percent and 60 percent gate openings, respectively. The figures demonstrate that the best deflector performance occurs with the deflector located 5 to 6 feet upstream from the end of the basin walls and positioned at an elevation in the range of 3899 to 3901 feet.

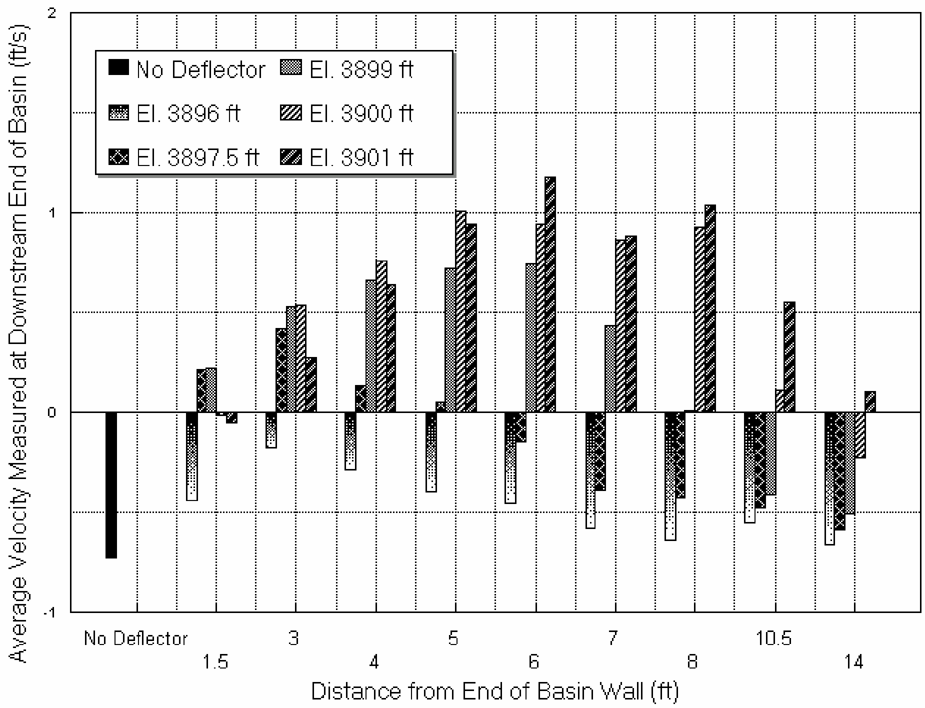


Figure 6.—Average velocity versus lateral deflector positioning with deflector angled at 60 degrees and basin operating at 40 percent gate opening.

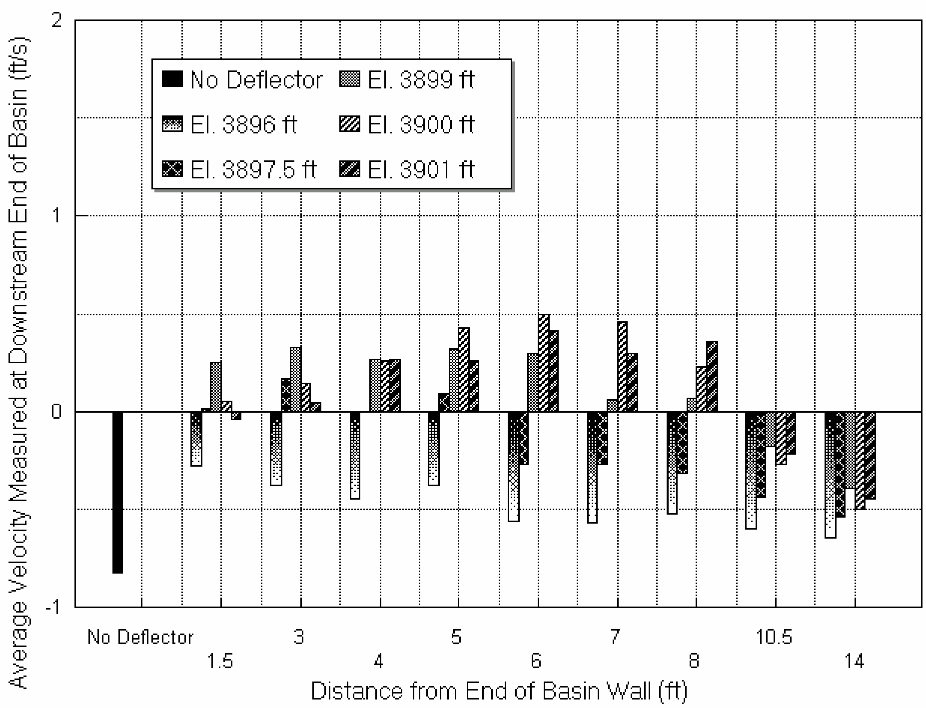


Figure 7.—Average velocity versus lateral deflector positioning with deflector angled at 60 degrees and basin operating at 60 percent gate opening.

2. *Angle* – Once the most effective range for lateral and vertical positioning was established, deflector angle was varied to determine best performance. For this case, lateral positioning was kept constant at 5 feet and deflector elevation was varied from 3896 to 3901 feet. Velocities were measured for deflector angles ranging from 40 to 90 degrees referenced from the horizontal plane as shown in figure 2.

   Figures 8 and 9 show that best performance occurs with the deflector angled at 80 or 90 degrees and with deflector elevation in the range of 3899 to 3901 feet.

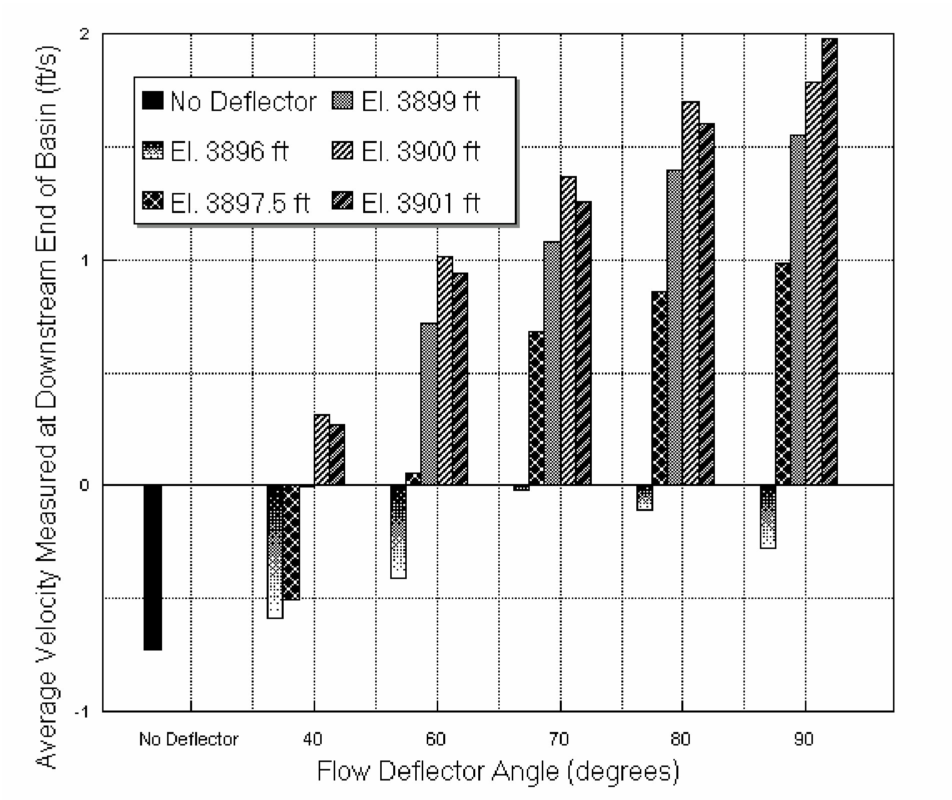


Figure 8.—Average velocity versus deflector angle with deflector positioned 5 feet laterally and basin operating at 40 percent gate opening.

3. *Size* – The next step was to determine if the deflector could be reduced in size in order to reduce costs and still maintain performance. For this set of tests, deflector lateral positioning was kept constant at 5 feet and deflector elevation was kept constant at 3900 feet. Deflectors 3 and 4 feet in height were tested at 80 and 90 degrees. Figures 10 and 11 show that although performance is still acceptable for the smaller deflectors, it is reduced compared with the performance of the 5-foot deflector. After some discussion, it was determined that the additional cost was insignificant compared to the increased confidence level in performance and, therefore, the 5-foot deflector was selected for the final design.

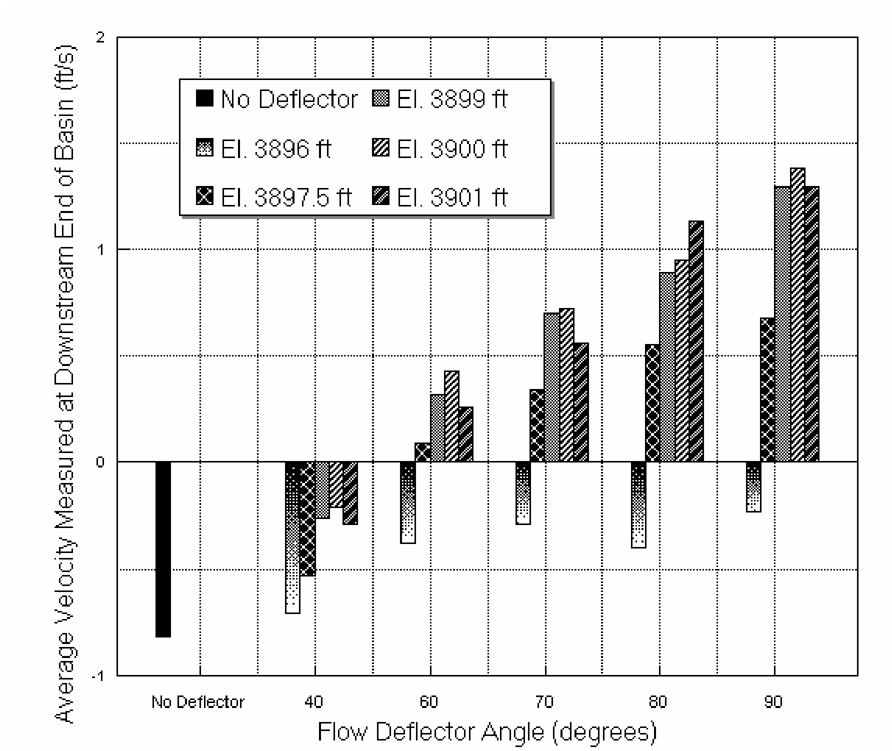


Figure 9.—Average velocity versus deflector angle with deflector positioned 5 feet laterally and basin operating at 60 percent gate opening.

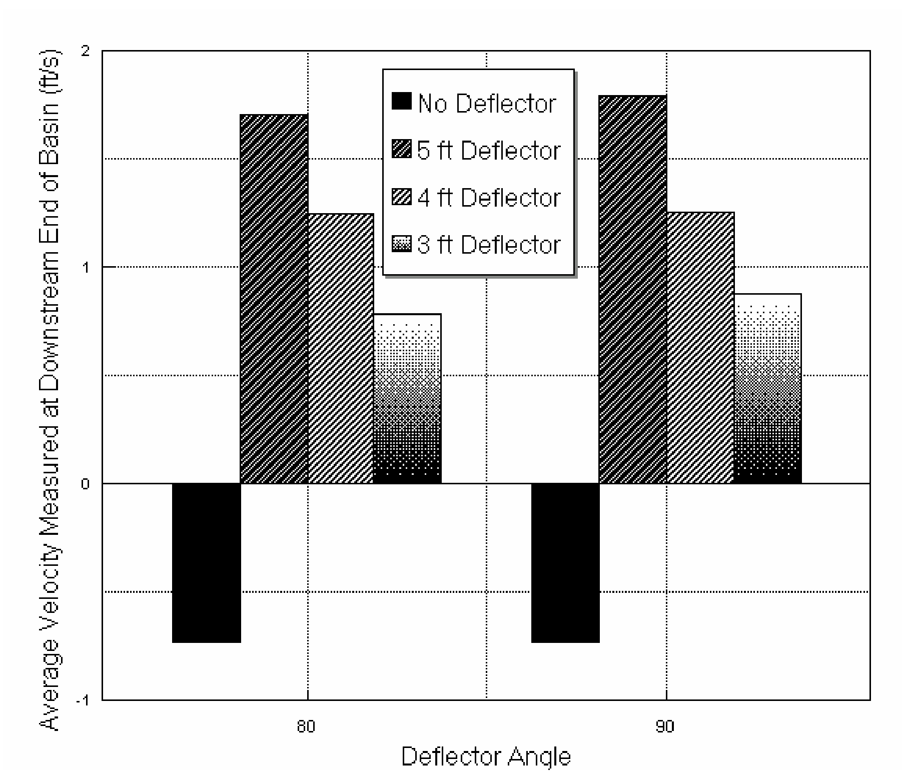


Figure 10.—Average velocity as a function of deflector angled 80 and 90 degrees for a 3-, 4-, and 5-foot-high deflector with the basin operating at 40 percent gate opening.

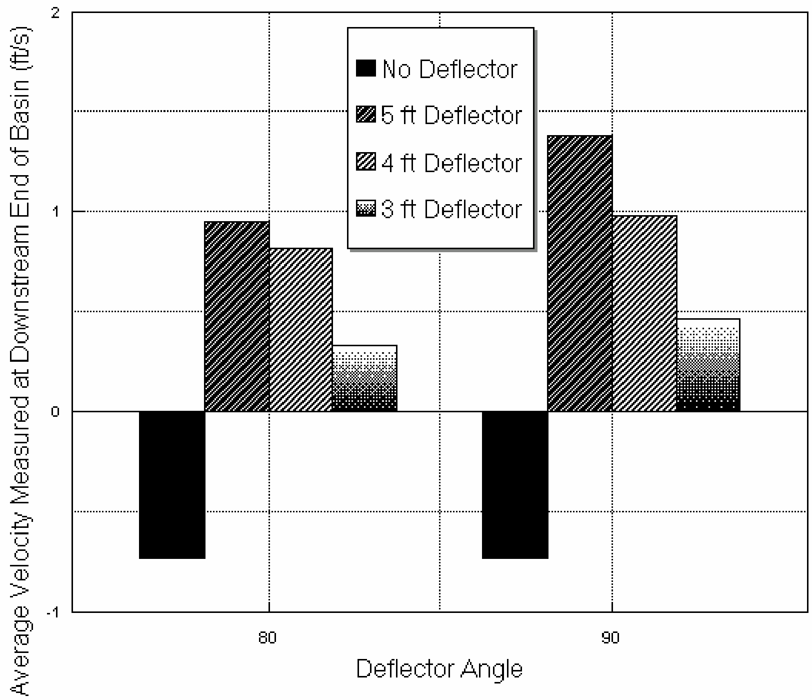


Figure 11.—Average velocity as a function of deflector angled 80 and 90 degrees for a 3-, 4-, and 5-foot-high deflector with the basin operating at 60 percent gate opening.

As a result of these investigations, it was determined that best deflector performance, based on average bottom velocities measured at the downstream end of the basin, occurred with a 5-foot-high deflector mounted 5 feet upstream from the end of the basin at elevation 3900 feet (11 feet above basin floor) and angled at 90 degrees.

Deflector Loading

Piezometer taps installed on the upstream and downstream faces of the model deflector were used to measure differential loading. The maximum loads predicted for the prototype deflector were 6,000, 12,000, and 12,600 pounds, respectively, for basin operations of 60, 80, and 100 percent gate openings.

Overall Performance

After the optimal design parameters were set, it was important to look at deflector performance with the basin operating throughout the full range of possible discharges up to the maximum flow at 100 percent gate opening in case unusual circumstances should require releases above those normally allowed while the deflector is in place. Table 2 shows the average bottom velocities measured without a deflector compared with those measured with the deflector set into

Table 2.—Basin performance with and without deflector

	Average prototype velocity measured in model at end of basin with and without deflector (ft/s)	
Gate opening (%)	No deflector	Optimal deflector at 3,900 feet and angled at 90 degrees
20	-0.44	1.3
40	-0.73	1.8
60	-0.82	1.4
80	-0.88	-0.5
100	-0.69	-0.2

optimal position for gate openings ranging from 20 percent to 100 percent. Table 2 shows that with the optimal deflector design in place, performance at gate openings ranging from 20 percent to 60 percent was very good. Average velocities for this range of discharge were greater than 1.0 ft/s and were directed in the downstream direction.

The table also shows that for gate openings of 80 percent and 100 percent, performance was reduced significantly, although still improved over having no deflector. Figure 12 demonstrates performance at higher discharges can be significantly improved by moving the deflector to a lower elevation. This could be accomplished with a mobile deflector supported on guides to allow vertical adjustments in position for operations at high and low discharges. However, since the outlet works will probably never be operated at these higher releases due to SOP limitations, the stationary deflector design positioned at elevation 3900 feet was determined acceptable.

Hydraulically Self-Cleaning Operations

Model investigations showed that without a deflector, materials can be flushed from the basin throughout the range of operations tested due to the nature of the flow occurring within the basin. This phenomenon occurs because turbulence within the basin periodically tosses materials high enough into the water column to be caught and subsequently carried out by the main jet exiting the basin. However, these suspended materials often hit their fall velocity as they are exiting the basin and are deposited back onto the basin end sill, thereby making them readily accessible to be carried right back into the basin by the upstream current. As a result, for a large range of discharges, although materials are flushed out, the inflow of materials is constant, thereby resulting in significant abrasion damage.

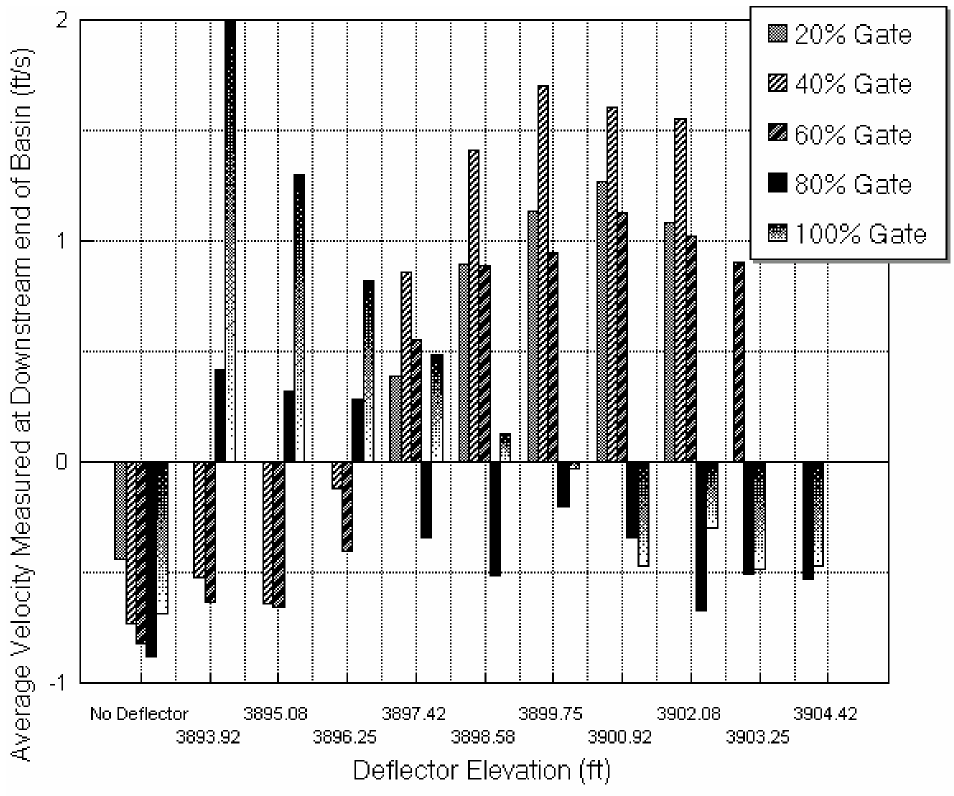


Figure 12.—Average velocity versus deflector elevation (deflector angled at 80 degrees and positioned 5 feet laterally).

With the optimal deflector design in place, model investigations demonstrated that the upstream component of velocity at the end of the basin is no longer strong enough to carry a significant amount of material back into the basin; therefore, most materials that are flushed from the basin will not be carried back in. As a result, the basin potentially becomes hydraulically self-cleaning, thereby reducing abrasion damage significantly. The range of sizes of materials that can be flushed from the basin will depend on outlet works operations and will be determined more precisely in future studies.

Field Evaluation

The final prototype deflector for Mason Dam was designed with a set of guides that would allow the deflector to be manually adjusted in angle and elevation for testing purposes. The prototype flow deflector was delivered to Mason Dam and installed by the BVID and Reclamation's Snake River Area Office in October 2002 (figure 13). In addition, basin abrasion damage was repaired with new concrete at the time the deflector was installed. In April 2003, the deflector was set to optimal position as determined from the model study before seasonal operations began.

Figure 13.—Prototype flow deflector installation at Mason Dam in October 2002.

In August 2003, after nearly 5 months of basin operations with the deflector in place, a field evaluation and dive inspection were conducted to verify the effectiveness of the deflector.

An Acoustic Doppler Profiler (ADP) probe was installed by a dive team to measure exit velocities at the downstream end of the basin. The deflector was raised above the water surface, and basin exit velocities were measured for outlet works operations ranging from 10 percent gate opening up to 60 percent gate opening at 10 percent increments. The same measurements were repeated with the deflector lowered to optimal position, with bottom elevation set to 3900 feet and angled at 90 degrees. Table 3 shows the discharge tested at Mason Dam compared with the discharge tested in the model for the same gate opening. The reason for the difference in values is because model study discharges were set based on maximum reservoir elevation, and the reservoir was actually 73 feet below that level at the time tests were conducted at Mason Dam.

Table 3.—Prototype discharges tested in the model and at Mason Dam

Gate opening (%)	Prototype discharge tested in model – corresponding to maximum reservoir (Elevation 4075 feet, ft^3/s)	Prototype discharge tested at Mason Dam at low reservoir (Elevation 4005 feet, ft^3/s)
10	N/A	85
20	230	163
30	N/A	250
40	420	330
50	N/A	400
60	75	500

Figure 14 shows the average prototype velocities exiting the basin, measured at elevation 3891 feet (2 feet above the basin floor elevation) for each gate opening tested, with and without a deflector. The figure shows significant improvement in flow conditions at the downstream end of the basin with the deflector lowered into optimal position for gate operations from 10 percent to 30 percent gate opening. Average prototype velocities are greater than 0.75 ft/s and have changed from upstream in direction to downstream with the deflector in place. However, for gate operations ranging from 40 percent to 60 percent gate opening, prototype velocities measured were inconclusive due to limitations of the ADP probe to accurately measure velocities when large quantities of air are entrained in the flow. The deflector was designed to redirect the main jet exiting the basin down toward the basin end sill. Therefore, at high discharges, when the jet is highly aerated, entrained air was also redirected downward towards the end sill where the ADP probe was located. As a result, accurate velocity measurements were not possible at the higher discharges.

Divers conducting the initial underwater inspection in August 2003 found only a few small stones in the basin and noted that the new concrete was very smooth and in excellent condition, with no signs of any erosion or wear. A second dive inspection of the stilling basin was conducted in August 2004 after a second season of operations with the deflector in place. Again, the divers found only a few small stones (total of four) throughout the entire basin. However, they discovered that a thin layer of the new concrete (used to repair the basin in October 2002) was gone, exposing aggregate at its surface.

After spending some time examining photos of the basin floor and consulting with Reclamation concrete experts and divers who had conducted similar inspections, it was concluded there was no indication that the cause of the missing layer was due to abrasion. Several factors were cited as probable causes of this phenomenon, including the fact that the concrete was exposed (despite an effort to protect it with a layer of hay) to temperatures well below freezing (5 degrees Fahrenheit) immediately following the laying of the new concrete. This likely caused the top layer to freeze before it had time to cure, thereby creating a weak

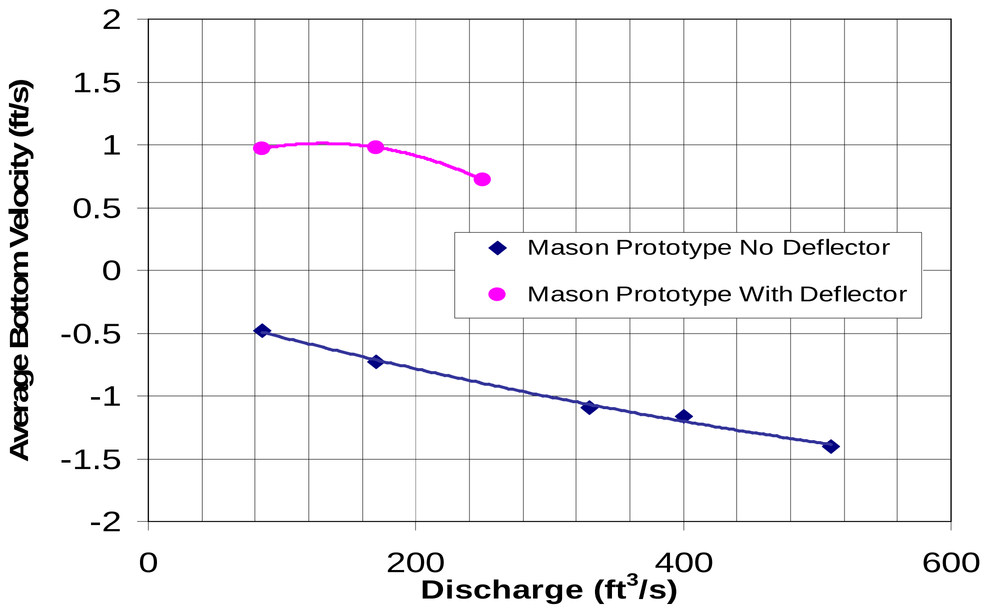


Figure 14.—Average bottom velocity measured at elevation 3891 feet at downstream end of Mason Dam tilling basin as a function of outlet works discharge.

top surface. In addition several dive team members had seen similar surfaces at Reclamation sites where there were no signs of abrasion damage or rocks in the basin, and erosion did not progress further in subsequent years.

A third dive inspection, conducted June 2005, showed no signs of abrasion damage and only a few stones in the basin, thus providing further evidence the deflector was performing as desired.

Figure 15 compares model and prototype average exit velocities, measured at elevation 3891 feet for each gate opening tested, with and without a deflector. The ADV probe used in the model study was not as sensitive to high air concentrations; therefore, velocity measurements were possible for all gate openings tested. Although model and prototype discharges are not identical (due to low reservoir elevation during prototype testing), figure 15 shows a strong correlation between model and prototype velocities measured at the same location for the same gate openings. Therefore, it would be reasonable to assume, with the field verified data already acquired, that the velocities measured in the model for gate openings ranging from 40 percent to 60 percent (with the deflector in place) are also a reasonable representation of prototype flow conditions, thereby demonstrating that the deflector is performing as desired and reducing the potential for entraining materials.

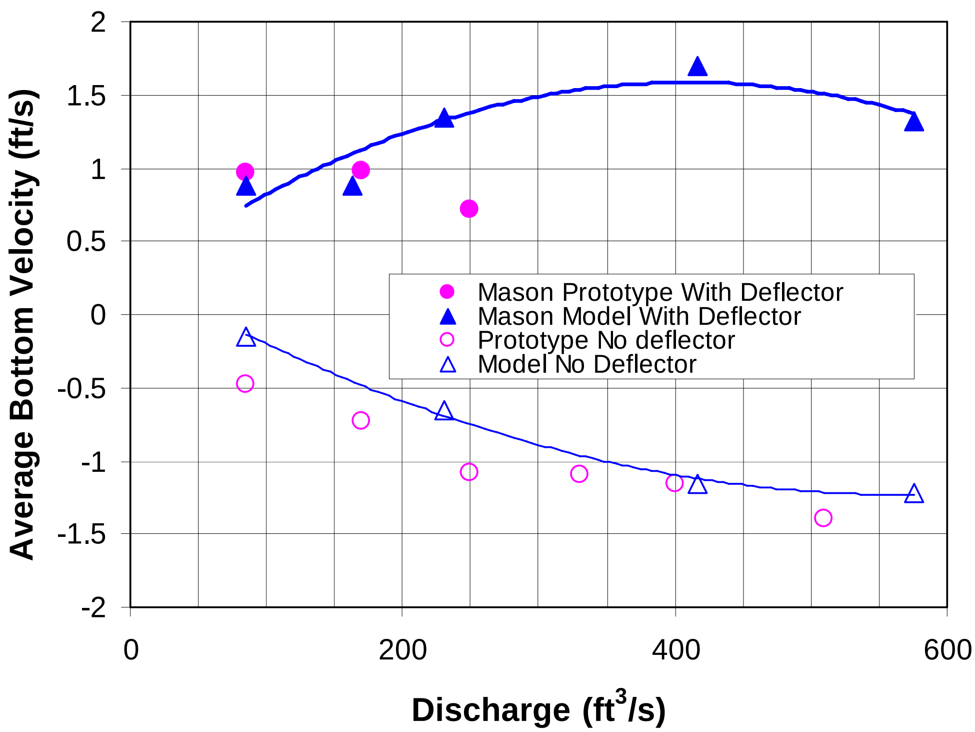


Figure 15.—Comparison of average prototype exit velocities measured in the model and in the prototype with and without a deflector.

Generalizing Deflector Design for Widespread Applications

The model investigations and field evaluation were used to develop a method for generalizing flow deflector design for Reclamation Type II stilling basins and basins of similar design based on velocity profiles measured at the end of the basin before a deflector is installed. In the future, velocity data measured on-site can be used to determine the optimal deflector design and location for a specific basin.

Optimal deflector design and position will vary over the operational range of most basins. Several practical approaches can be considered to achieve both economical and effective performance:

- One option is to design a stationary deflector to be effective for the most predominant range of basin operations. This would mean that when the basin was operated outside the deflector design range, materials may be drawn into the basin. In this case, it would be recommended that the basin be operated within the designated design range periodically to help purge materials from the basin.

- A second option would be to design a moveable deflector supported on guides so that deflector elevation could be changed for different ranges of operations. In most cases, this would require only two positions.

- A third option may be to install two separate deflectors staggered in position, both vertically and horizontally, so that flow conditions can be improved throughout the full range of operations without having to adjust deflector positioning. Preliminary research conducted by WRRL has demonstrated this may be a viable solution.

Implementation of any of the above options should significantly reduce the amount of damage caused by abrasion and the costs associated with basin repairs.

Details for determining optimal deflector design will not be released until after the patent has been awarded.

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