RECLAMATION

Managing Water in the West

Hydraulic Laboratory Report HL-2007-02

Fontenelle Dam Flow Deflectors for Mitigating Stilling Basin Abrasion Damage

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

January 2007

REPORT DOCUMENTATION PAGE	Form Approved OMB No. 0704-0188

1. REPORT DATE *(DD-MM-YYYY)*	2. REPORT TYPE	3. DATES COVERED *(From - To)*
11-01-2007	Technical	Oct 2005 – Dec 2006

4. TITLE AND SUBTITLE	5a. CONTRACT NUMBER
Fontenelle Dam Flow Deflectors for Mitigating Abrasion Damage	5b. GRANT NUMBER
	5c. PROGRAM ELEMENT NUMBER
6. AUTHOR(S)	5d. PROJECT NUMBER
Hanna, Leslie J.	5e. TASK NUMBER
	5f. WORK UNIT NUMBER

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)	8. PERFORMING ORGANIZATION REPORT NUMBER
Bureau of Reclamation Water Resources Research Laboratory PO Box 25007 Denver, CO 80225	HL-2007-02
9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)	10. SPONSOR/MONITOR'S ACRONYM(S)
Colorado River Storage Project (CRSP)-Seedskadee Project and the State of Wyoming Water Development Commission, Reclamation's Flaming Gorge Field Division, and the Science and Technology Research Program.	11. SPONSOR/MONITOR'S REPORT NUMBER(S)

12. DISTRIBUTION/AVAILABILITY STATEMENT

Available from: National Technical Information Service, Operations Division, 5285 Port Royal Road, Springfield Virginia 22161 http://www.ntis.gov

13. SUPPLEMENTARY NOTES

14. ABSTRACT

A physical hydraulic model study was conducted to evaluate the hydraulic characteristics of the Fontenelle Dam river outlet works stilling basin and to design a flow deflector or a series of flow deflector panels for the purpose of mitigating basin abrasion damage. The Fontenelle basin has a width of 62 feet that spans a distance much greater than any basin previously studied. As a result, the project presented the additional challenge of designing a deflector for the nonuniform flow conditions that often occur in wide-span basins at flows less than the design flow.

15. SUBJECT TERMS

Abrasion, Erosion, Stilling Basins, Energy Dissipation, Terminal Structure, Fontenelle Dam

16. SECURITY CLASSIFICATION OF:			17. LIMITATION OF ABSTRACT	18. NUMBER OF PAGES	19a. NAME OF RESPONSIBLE PERSON
			SAR	28 plus appendices	Cliff Pugh
a. REPORT UL	b. ABSTRACT UL	THIS PAGE UL			19b. TELEPHONE NUMBER *(Include area code)* 303-445-2151

Hydraulic Laboratory Report HL-2007-02

Fontenelle Dam Flow Deflectors for Preventing Stilling Basin Abrasion Damage

Leslie J. Hanna

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

January 2007

Mission Statements

The mission of the Department of the Interior is to protect and provide access to our Nation's natural and cultural heritage and honor our trust responsibilities to Indian Tribes and our commitments to island communities.

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

Acknowledgments

Support provided by Reclamation's Flaming Gorge Field Division, especially Chuck Green and Warren Blanchard, was invaluable. Thanks to Vinny Castillo and Bill Baca for their efforts in model data acquisition. Thanks to Marty Poos, Dane Cheek, Neal Armstrong, and Rudy Campbell for their attention to detail in the construction of the model. Thanks to Kathy Frizell for the peer review of this document.

Hydraulic Laboratory Reports

The Hydraulic Laboratory Report series is produced by the Bureau of Reclamation's Water Resources Research Laboratory (Mail Code 86-685600), PO Box 25007, Denver, Colorado 80225-0007. At the time of publication, this report was also made available online at:
http://www.usbr.gov/pmts/hydraulics_lab/pubs/HL/HL-2007-02.pdf

Disclaimer

This hydraulic model study was funded by the Colorado River Storage Project (CRSP)-Seedskadee Project and the State of Wyoming Water Development Commission, Reclamation's Flaming Gorge Field Division, and the Science and Technology Research Program. (Project ID No. 82.)

Table of Contents

List of Figures

Figure *Page*

List of Figures

Figure *Page*

List of Figures

Figure *Page*

List of Figures

Figure *Page*

List of Tables

Table *Page*

Purpose

A physical hydraulic model study was conducted to evaluate the hydraulic characteristics of the Fontenelle Dam river outlet works stilling basin and to design a flow deflector or a series of flow deflector panels for the purpose of mitigating basin abrasion damage. The Fontenelle basin has a width of 62 feet (ft) that spans a distance much greater than any basin previously studied. As a result, the project presented the additional challenge of designing a deflector for the nonuniform flow conditions that often occur in wide-span basins at flows less than the design flow.

Background

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. Unfortunately, when repairs are made, many basins again experience the same damage within one or two operating seasons. Research conducted by Reclamation's Water Resources Research Laboratory (WRRL) in Denver has demonstrated that the installation of flow deflectors (figure 2) in basins less than 25 ft in width can improve the flow distribution in existing basins. Improving the flow distribution can minimize or eliminate the potential for materials to be carried into stilling basins, thus increasing the life of the basins and reducing necessary repairs [1]. A patent is pending on this technology.

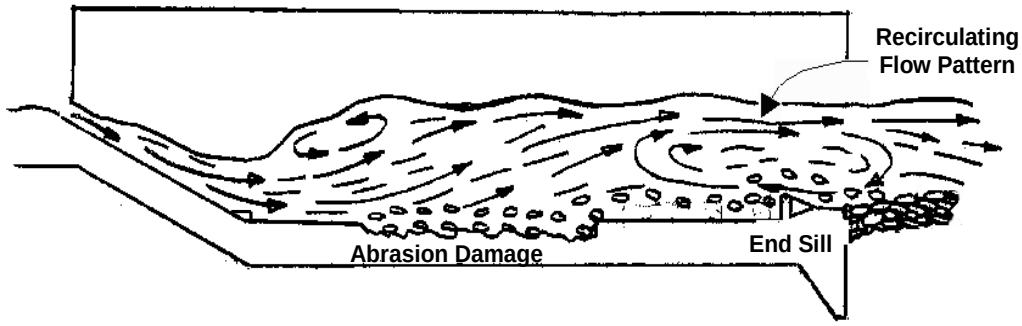

Figure 1. A recirculating flow pattern is produced over the stilling basin end sill during normal operations, creating the potential for abrasion damage.

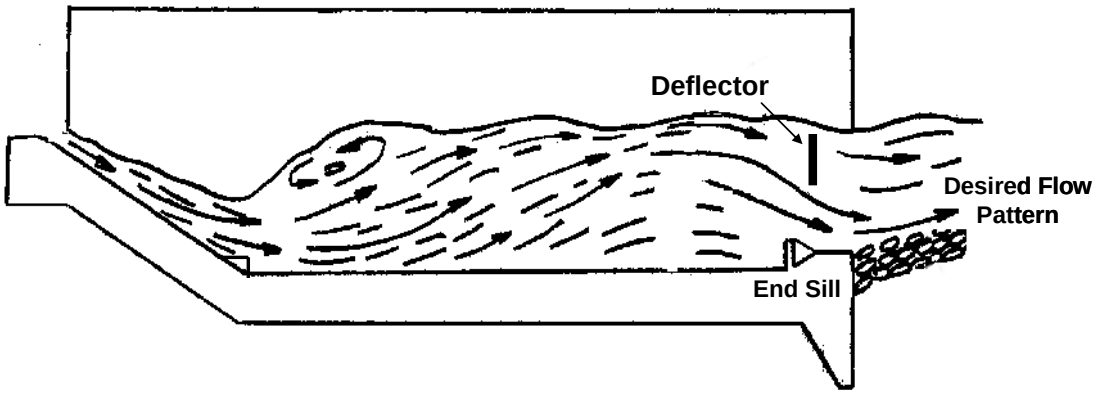

Figure 2. Desired flow pattern with a typical flow deflector installed.

Stilling basins wider than about 25 ft present an additional challenge due to the tendency of the incoming jet or hydraulic jump to attach to one side of the basin, thus creating nonuniform flow conditions at discharges less than the maximum design flow for the basin. In addition, the hydraulic jump will often oscillate from one side of the basin to the other without changes in gate operations.

Introduction

The Fontenelle Dam outlet works stilling basin is a typical Reclamation type II basin with a width spanning 62 ft and a long history of abrasion damage [2]. Fontenelle Dam is located on the Green River in Lincoln and Sweetwater Counties, about 24 miles southeast of La Barge, Wyoming. Fontenelle Reservoir has a total capacity of 345,000 acre feet. The reservoir provides water for municipal and industrial use, the Seedskadee Wildlife Refuge, and minimum flows for fish, and recreation. The dam is an earth and gravel structure approximately 6,000 ft long and rises about 127 ft above the riverbed. The river outlet works and powerplant are located near the center of the embankment adjacent to the toe of the dam, with the power penstock branching from the river outlet works (figure 3). The river outlet works consists of two 8-1/2-ft by 11-ft top-seal regulating gates discharging into two 14-ft diameter horseshoe conduits into a curved chute and stilling basin. The third tunnel consists of a 10-ft diameter pressure conduit that branches off to the powerplant penstock 20 ft upstream from the tunnel portal. The end of the third conduit feeding into the outlet works is terminated at an 8-½ ft by 8-½ ft slide gate controlled turnout at the upstream end of the stilling basin chute.

A physical hydraulic model of Fontenelle Dam river outlet works was constructed in the WRRL to study flow patterns associated with the basin and to determine if a flow deflector or series of deflectors could be designed to mitigate entrainment of materials into the basin (figure 4).

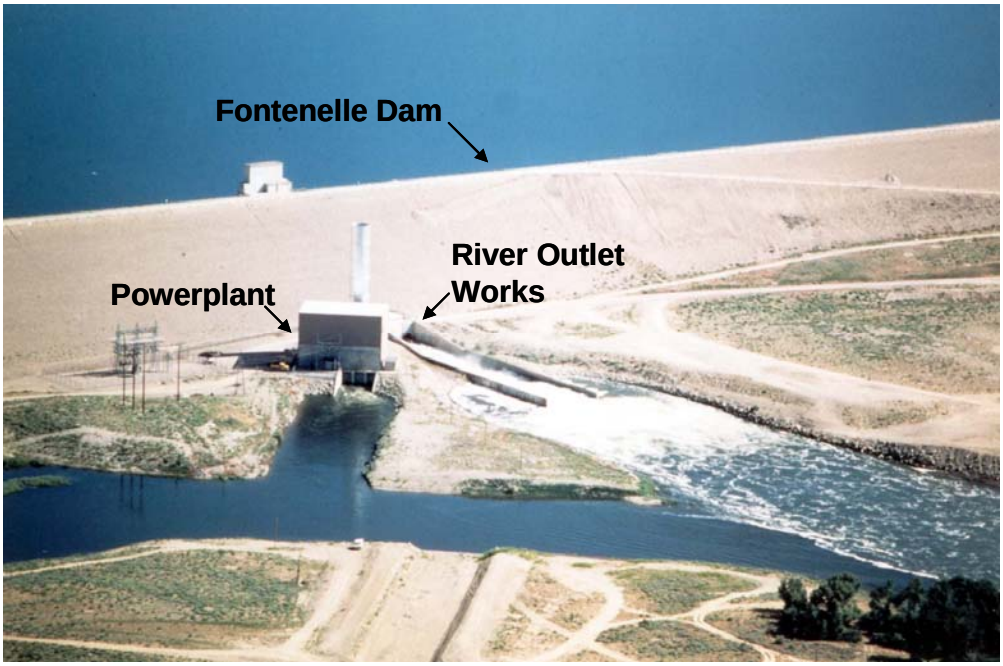

Figure 3. View looking upstream at Fontenelle Dam, the river outlet works, and the powerplant.

Conclusions

The studies began with evaluating the existing conditions for a range of operations up to 14,200 cubic feet per second (ft^3/s), then progressed with testing a series of different configurations using one or more deflectors through the same range of operations, until an optimal deflector configuration was determined. Optimal is defined as producing the maximum possible downstream average velocity exiting the stilling basin over the largest range of operations without producing significant erosion immediately downstream from the stilling basin. All dimensions and measurements reported are scaled to prototype dimensions and are referenced to the bottom upstream edge of the deflector.

The following conclusions are based upon the results from the hydraulic model testing:

(1) Flow conditions were first evaluated for the existing basin without a deflector. Average bottom velocities measured at the end of the basin without a deflector were predominantly in the upstream direction and were as high as -4.0 feet per second (ft/s) with a maximum velocity of about -10 ft/s measured over the range of operating conditions tested (negative values indicate velocities were directed upstream into the basin).

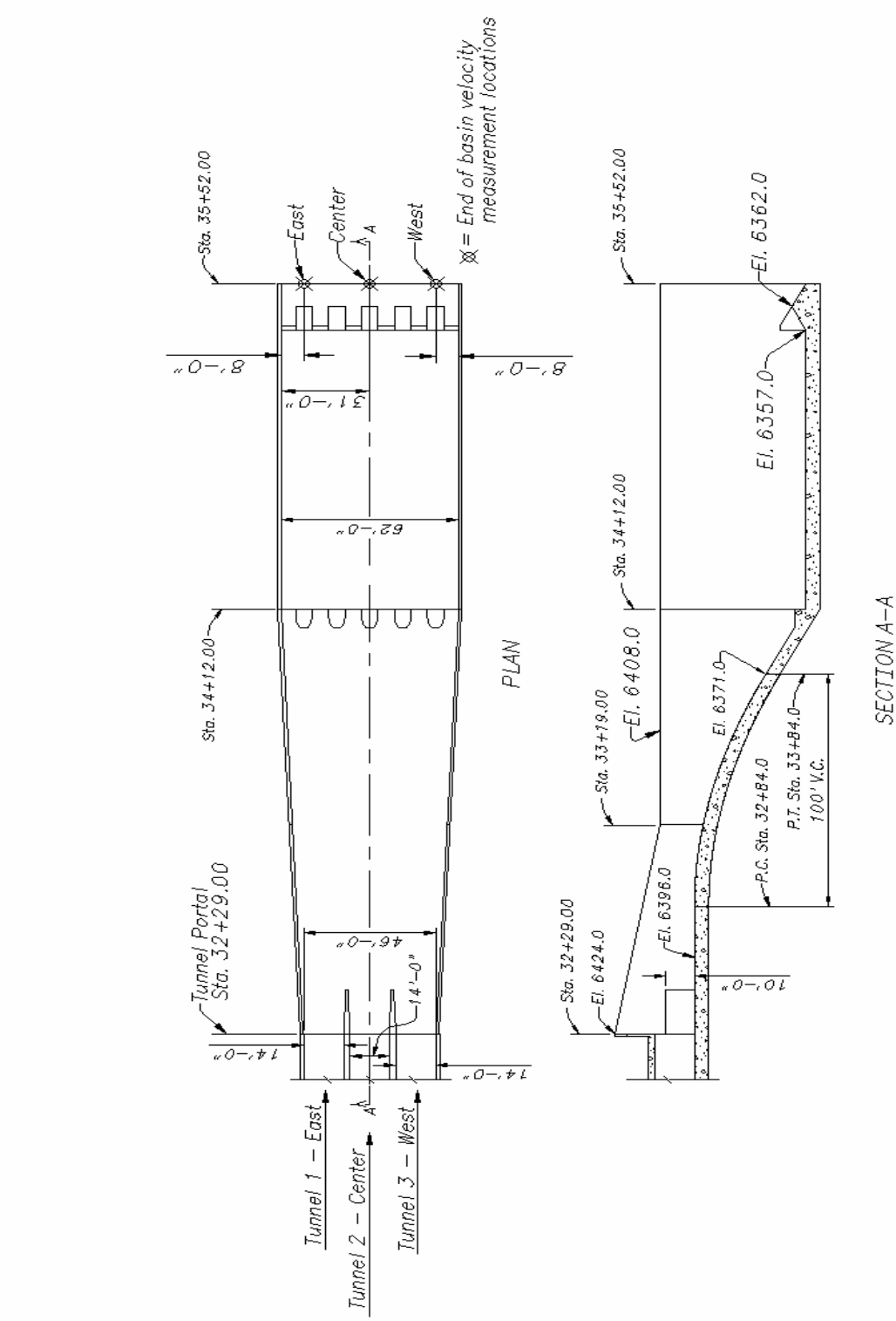

Figure 4. Plan and section of the Fontenelle Dam river outlet works stilling basin. (Locations for velocity measurements in the model are also shown.)

(2) The final optimal deflector design consists of two stationary deflectors oriented vertically and staggered in position within the stilling basin walls. The vertical dimension of the upper deflector is 11.33 ft. The upper deflector is positioned 25 ft above the basin invert elevation and 13.33 ft upstream from the end of the basin walls. The lower deflector has a vertical dimension of 5.67 ft and is positioned 13.75 ft above the basin invert elevation and 6.65 ft upstream from the end of the basin walls. With this arrangement, flow conditions should be improved significantly to minimize the potential for carrying materials into the basin, thereby extending basin life.

(3) With the final optimal deflector design in place, average velocities were redirected downstream away from the basin. Average downstream bottom velocities measured at the end of the basin for discharges ranging from 1,126 ft^3/s to 14,200 ft^3/s were as high as 9.5 ft/s, with a maximum velocity from occasional flow surges as high as 15 ft/s to 18 ft/s. Velocities of this magnitude could cause some shifting of riprap; however, no significant erosion is expected for riprap greater than 18 inches in diameter [3].

(4) The optimal, two deflector staggered configuration produced better overall performance than a single moveable deflector, although flow conditions with the vertically mobile deflector were significantly improved over having no deflector within the basin. The mobile deflector tested was positioned vertically with a vertical dimension of 11.33 ft and 13.33 ft upstream from the end of the stilling basin walls. However, when the mobile deflector was positioned for best performance for discharges above 6,010 ft^3/s (elevation 6368 ft), average velocities were as high as 18 ft/s and could produce significant erosion for riprap as large as 3 ft or 4 ft in diameter [3].

(5) A single stationary deflector was tested and proved successful in significantly improving flow conditions up to a discharge of 3,062 ft^3/s, with some improvement in flow conditions for discharges up to 6,010 ft^3/s. However, for flows above 6,010 ft^3/s, although there would be some improvement in performance, it would be insignificant compared to performance without a deflector. Best performance for a single stationary deflector occurred with the deflector positioned vertically at elevation 6387 ft, with a vertical dimension of 11.33 ft, and 13.33 ft upstream from the end of the stilling basin walls. With a single deflector in place, although there may be some downstream movement of materials, no significant erosion would be expected.

(6) Differential loading across the upper and lower deflectors for model operations up to 14,200 ft^3/s was determined by measuring pressures with piezometers. The greatest pressure drop measured across the upper deflector was about 4.6 pounds per square inch (lb/in^2) and occurred at a location near the east wall, at a discharge of 14,200 ft^3/s. The largest pressure drop

measured across the lower deflector was about 8 lb/in^2 and again occurred at a location near the east wall at a discharge of 14,200 ft^3/s.

The Model

A 1:16 geometric scale was used to model the Fontenelle Dam river outlet works stilling basin (figure 5) to study the effect of deflector positioning on flow patterns over the basin end sill. 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 $L_r = 1:16$

Velocity ratio $V_r = L_r^{½} = 1:4$

Discharge ratio $Q_r = L_r^{5/2} = 1:1024$

To accommodate the substantial drop in Reynold's number that occurs at the end of the stilling basin for flows below design flow, a range of discharge was evaluated in the model for each prototype discharge. The range tested for each discharge was based on results from previous stilling basin research and allowed for a conservative design of the deflector to provide effective performance over the range of prototype discharges represented [4].

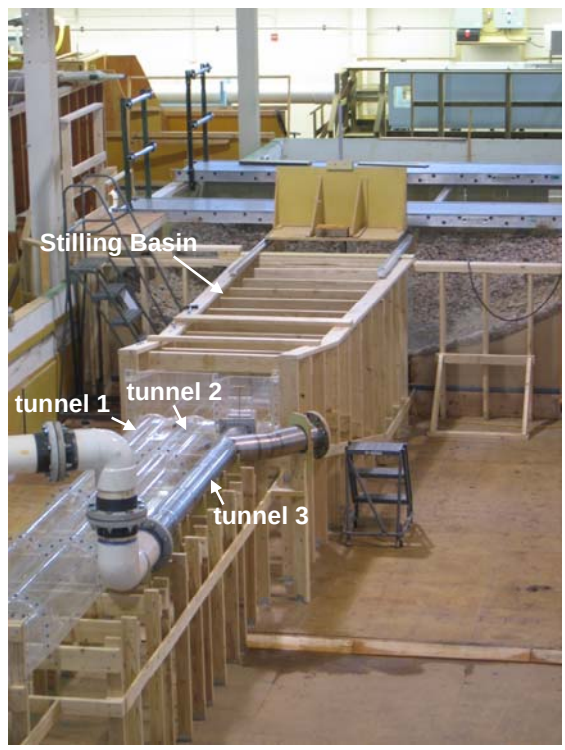

Figure 5. Overview of the 1:16 scale Fontenelle Dam river outlet works model, looking downstream.

Model Description

The model included the following prototype features:

(1) The two 8-½ ft-by 11-ft high pressure regulating gates and 14-ft diameter horseshoe conduits (referenced as tunnel 1 for the eastern conduit and tunnel 2 for the center conduit).

(2) The 8-½ ft-by 8-½-ft regulating gate (referenced as tunnel 3) and upstream bifurcation to the powerplant stub.

(3) The 62-ft-wide hydraulic jump stilling basin with curved chute, chute blocks, and dentated end sill (figure 4).

(4) Approximately 200 ft of topography downstream from the basin, constructed on a 5:1 adverse slope up from the end sill.

Water was supplied from the permanent laboratory venturi system and routed to the model through the pipe chase surrounding the perimeter of the laboratory. The prototype radial gates in tunnels 1 and 2 were represented with slide gates in the model (figure 6) because the conduit length was enough to dampen any significant effects associated with differences in gate geometry, and to reduce costs. Hoist travel for the radial gates was converted to percent slide gate opening and is given in table 1. Then discharge corresponding to each slide gate opening, at 20% increments, was calculated based on the radial gate discharge curves provided in the Fontenelle Dam Designer's Operating Criteria (DOC) and is given in tables 2 and 3 [5].

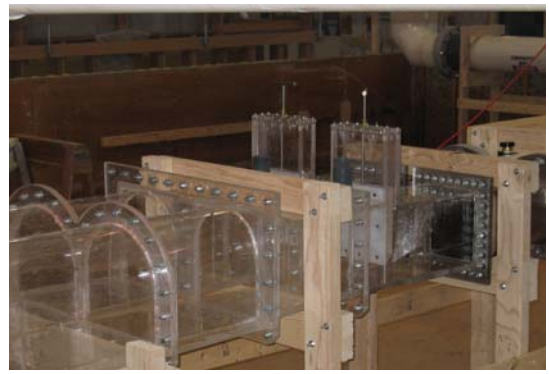

Figure 6. High pressure regulating slide gates in the model for tunnels 1 and 2.

Table 1. Representative slide gate opening as a function of radial gate hoist travel for tunnels 1 and 2

Radial gate hoist travel (ft)	Slide gate opening (ft)
1	7.2
2	13.6
4	27.3
6	43.3
8	59.6
10	78
11	87.7
12.125	100

Table 2. Test flow conditions investigated with the center tunnel, tunnel 2, operating

Tunnel 2 Operating Alone		
Gate opening (%)	Prototype discharge represented (ft^3/s)	Prototype tailwater elevation (ft)
20	1,126	6396.4
40	2,120	6397.1
60	3,062	6397.7
80	4,301	6398.4
100	6,010	6399.3

Table 3. Test flow conditions investigated with multiple tunnels operating

Multiple tunnels operating	Prototype discharge (ft^3/s)	Prototype tailwater elevation (ft)
Tunnel 1 and Tunnel 2 at 80% gate opening	8,601	6400.4
Tunnel 1 and Tunnel 2 at 100% gate opening	12,020	6401.5
Tunnel 1 and Tunnel 3 discharging equally at 4,100 ft^3/s; Tunnel 2 discharging at 6,010 ft^3/s	14,200	6401.9

Tailwater elevation was set for each flow condition, using tailwater data obtained during Fontenelle Dam outlet works operations. Tailwater was set based upon the powerplant operating under a continuous discharge of 1,700 ft^3/s, except when all three tunnels were discharging together at 14,200 ft^3/s. For this condition it was assumed that the powerplant would not be operating. Slide gate discharge for tunnel 3 was set according to the discharge curve provided in the DOC [5].

Model Study Investigations

Model investigations were conducted to evaluate hydraulic conditions at the stilling basin exit for the range of operating conditions expected in the prototype. Figure 7 shows a view of the model looking through the plexiglass side wall. The model was operated to match the Fontenelle Dam DOC [5]. According to the DOC, the center tunnel, tunnel 2, will be operated alone until it is fully opened and discharging at about 6,010 ft^3/s. If more flow is required, tunnel 1 is added and both are operated symmetrically. If the magnitude of the flood forecast requires greater releases, then the powerplant bypass gate can be opened to 8 ft for a discharge of about 4,100 ft^3/s and tunnel 1 should be opened to provide 4,100 ft^3/s for a balanced flow. This gives a discharge capacity of 14,200 ft^3/s, when the water surface elevation is at the spillway crest elevation 6401.9. Several representative flow conditions within this range of operation were tested and are listed in tables 2 and 3.

Figure 7. View looking through the plexiglass side wall of Fontenelle Dam outlet works model discharging at 6,010 ft^3/s.

Velocities in the model were measured with a SonTek Acoustic Doppler Velocimeter probe and were measured near the bottom at the downstream end of the basin at three separate positions across the width of the basin (figure 4). Best deflector performance was defined as the configuration that produced the highest maximum possible downstream average velocity exiting the basin over the largest range of operations.

Five separate deflector configurations, with varying dimensions and elevations, were tested in the model over the same range of operating conditions.
The initial deflector configuration consisted of three separate panels to span the 62-ft-wide basin. Each panel was mounted on a set of guides attached to the basin sidewalls or dentates to allow independent vertical movement of the three deflector panels within the basin. For the next four test configurations, a second deflector panel was added, spanning the width of the basin and staggered in location, both vertically and horizontally, from the first set of panels.
Figure 8 shows a typical deflector arrangement and dimension description.

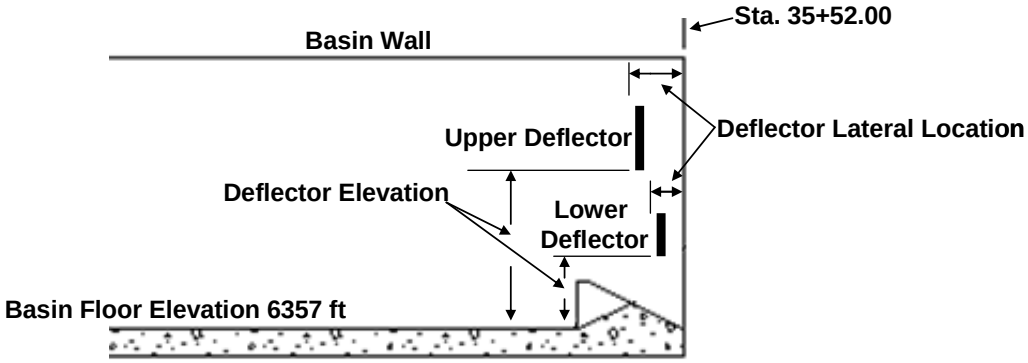

Figure 8. Typical deflector arrangement tested with an upper and lower deflector staggered vertically and horizontally in position.

In addition, deflector loading was investigated using 24 piezometer taps that were installed equally spaced across the upstream and downstream faces of each deflector. The taps were connected to a manometer board to measure static hydraulic differential loading on each deflector, for the final recommended geometry, for flow rates up to a maximum discharge of 14,200 ft^3/s.

Model Results and Discussion

No Deflector

Initial investigations began with collecting and analyzing velocity data to define basin performance of the existing basin without a deflector. For each flow condition tested, velocities were measured at three locations across the width of the basin and in 1.5-ft vertical increments to map out resulting flow patterns at the stilling basin exit. Figures 9-15 show average velocities as a function of elevation for vertical profiles measured at the east, center and west locations over the basin end sill.

The figures also show average velocity magnitudes and directions with flow from left to right. Velocity data to the left, or negative, of the vertical elevation scale are upstream, while those to the right, or positive, are downstream. Basin invert elevation is 6357 ft. The figures show that strongest upstream velocities occur at an elevation about 6 ft above the basin floor elevation where flow can be pulled over and into the stilling basin just above the dentated sill. Initial investigations showed that average velocities measured at this elevation (6363 ft) at the end of the basin and at three positions across the width of the basin provided a good representation of bottom velocities that either carry materials into or flush materials out of the basin. Therefore, velocities measured at this location were used as a basis to determine basin or deflector performance for all subsequent investigations with deflectors in the basin. Note that negative average velocities

indicate flow is directed into the stilling basin (with the potential of moving materials into the basin) and positive average velocities indicate flow is downstream away from the stilling basin, thus indicating good performance. The strongest average velocities measured in the upstream direction for the existing basin were as high as -4 ft/s, with maximum upstream velocities as high as -10 ft/s, indicating the strong potential for pulling materials into the basin.

The elevation for each profile where velocities cross over the axis from negative to positive is usually a good indication of the location of the most concentrated portion of the flow exiting the basin and was used to determine the initial elevation for each deflector. However, some interpretation of data is required for each of the figures because the upper portion of the water column is usually highly aerated and may cause some error in velocity measurements, resulting in erratic values. However, the general trend in values or flow patterns is thought to be representative of the flow conditions in the prototype. This has been confirmed with correlation of previous research during field evaluations [4]. In addition, air concentration is relatively small near the bottom and at the end of the basin where velocities were measured to define basin performance.

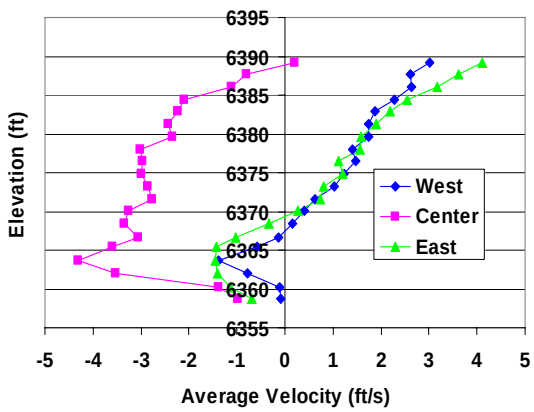

Figure 9. Vertical velocity profiles for tunnel 2 discharging at Q = 1,126 ft^3/s.

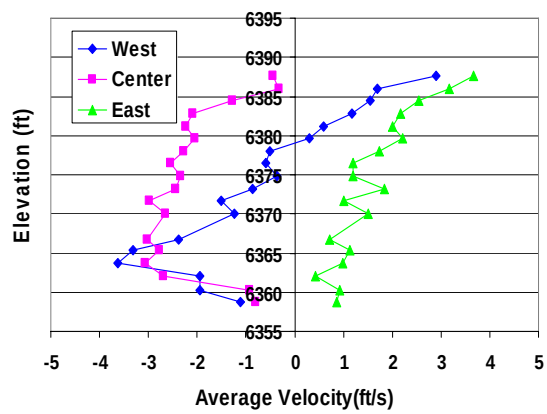

Figure 10. Vertical velocity profiles for tunnel 2 discharging at Q = 2,120 ft^3/s.

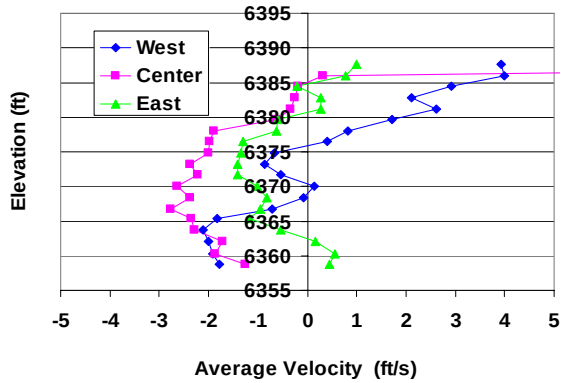

Figure 11. Vertical velocity profiles for tunnel 2 discharging at Q = 3,062 ft^3/s.

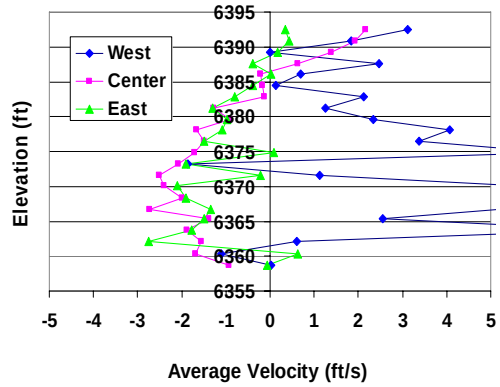

Figure 12. Vertical velocity profiles for tunnel 2 discharging at Q = 4,301 ft^3/s.

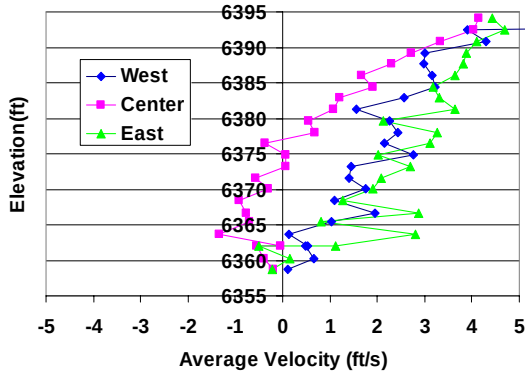

Figure 13. Vertical velocity profiles for tunnel 2 discharging at Q = 6,010 ft^3/s.

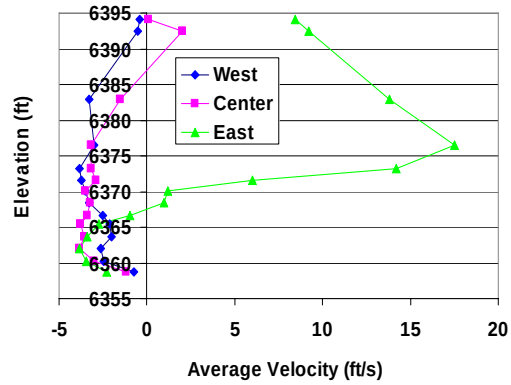

Figure 14. Vertical velocity profiles for tunnels 1 and 2 discharging equally for a total Q = 8,601 ft^3/s.

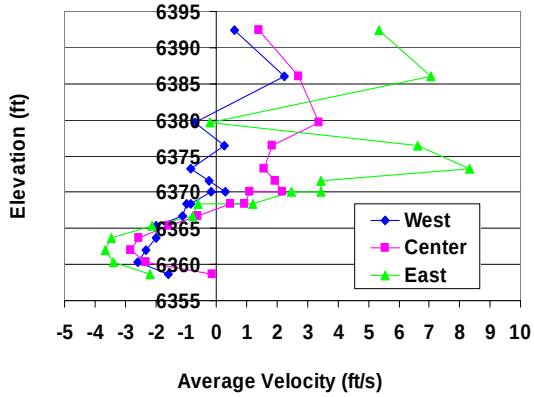

Figure 15. Vertical velocity profiles for tunnels 1 and 2 discharging equally for a total Q = 12,020 ft^3/s.

With Deflectors

Deflector performance is usually best when the deflector is positioned at an elevation corresponding to the bottom of the most concentrated portion of the downstream jet exiting the basin. Although this location can often be identified with vertical profile data, it changes with discharge. So best performance occurs when the deflector can be positioned to be effective over the largest or most predominant operating range expected in the prototype.

When evaluating deflector performance, relative deflector performance was determined by comparing the average velocities measured at the lowest elevation 6363 ft over the basin end sill. Higher average velocities in the positive direction indicate better performance. Figure 16 shows an example of a histogram developed by analysis software from a typical test run of 3,000 data points. Figure 16 shows data distribution for a case where the average velocity measured was near zero. The figure shows that individual velocity measurements range from 5 ft/s to - 5 ft/s; therefore, some materials may be carried into the basin during upstream flow surges. Therefore, although performance with an average velocity near zero may be significantly better than performance without a deflector with negative average velocities, it does not mean materials won't be pulled into the basin. Figure 17 shows the data distribution for a case where the average velocity measured was 2.3 ft/s and directed downstream. This figure shows that although the average velocity is positive, some flow velocities in the upstream direction are as high as those in the histogram of the previous example. However, in this case, because the majority of the velocity samples measured are positive, the potential for moving materials into the basin is much smaller than that of the condition where the average velocity was near zero. Therefore, higher average velocities indicate better performance.

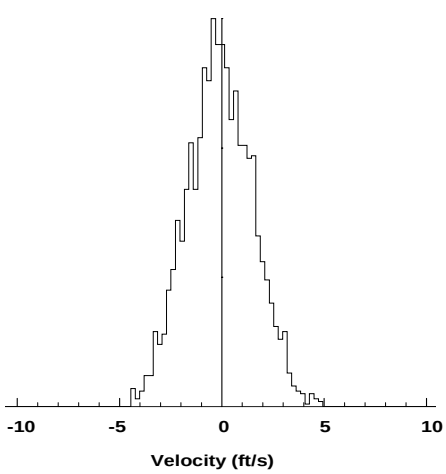

Figure 16. Example histogram for data distribution of 3,000 samples. Average velocity is near 0.0 ft/s.

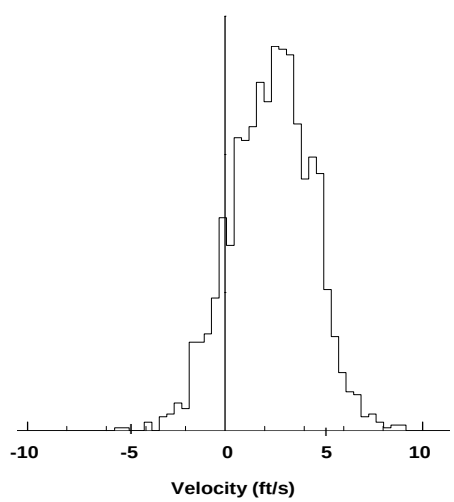

Figure 17. Example histogram for data distribution of 3, 000 samples. Average velocity is 2.3 ft/s.

Single Deflector Span

The vertical profile data from the existing basin tests were used to determine the initial vertical location for each of the three deflector panels for each flow condition tested. These initial investigations with the model showed that there was no significant advantage in moving the three panels independently, so all three panels were moved as a single unit for all subsequent investigations (figure 18). The deflector lateral location, height, and orientation were determined from guidelines developed from previous site- specific model studies and research [4]. The three-panel unit was oriented perpendicular to the basin floor, with a vertical dimension of 11.33 ft, and positioned at a lateral location 13.33 ft (referenced to the upstream face of the deflector) upstream from the end of the stilling basin. As the panel unit was moved up vertically in increments of 1.5 ft, bottom velocities were measured to determine deflector performance. Deflector elevation, referenced to the bottom edge of the deflector, ranged from 6365 ft to 6387 ft. Actual elevations that were tested varied for each flow condition depending on several factors, including analysis of initial vertical velocity profiles measured at the end of the basin. Figures 19-26 show average bottom velocities at elevation 6363 ft for each discharge, measured without a deflector compared to velocities measured at the same location for each deflector elevation tested.

Figure 18. View looking upstream at the deflector panels installed at the end of the basin.

Figures 19-21 demonstrate that for discharges ranging from 1,126 ft^3/s to 3,062 ft^3/s, performance improves as the deflector is moved higher in elevation. Figures 24-26 show that for discharges ranging from 8,601 ft^3/s to 14,200 ft^3/s, deflector performance improves as the deflector is moved lower in elevation. This difference in deflector performance with discharge is due to changes in the hydraulic jump characteristics with flow and tail water. At low discharges, the hydraulic jump is fairly weak and the most concentrated portion of the jet immediately rises off the basin floor and is high in the water column by the time it reaches the end of the basin. Therefore, the deflector must be positioned at higher elevations to redirect flow downward.

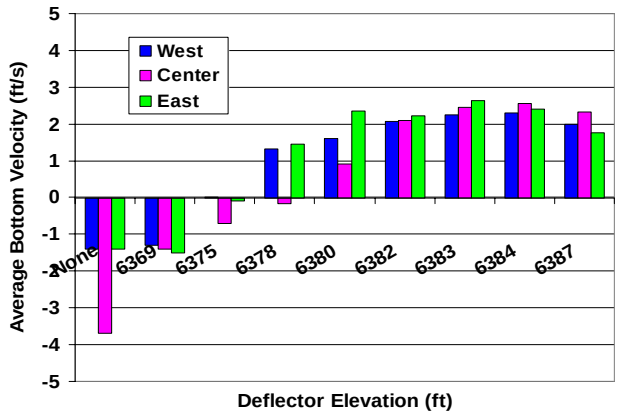

Figure 19. Average bottom velocity measured at El. 6363 ft at the basin exit for Q = 1,126 ft^3/s.

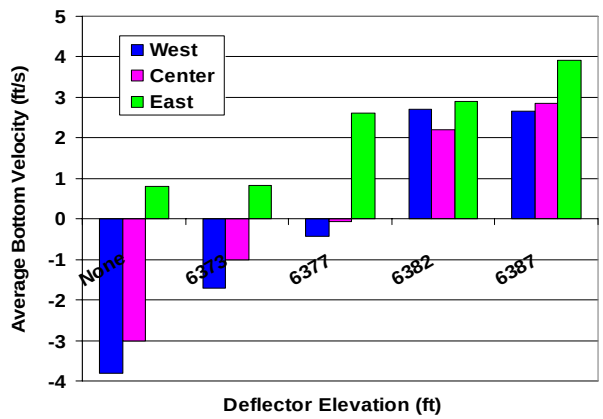

Figure 20. Average bottom velocity measured at El. 6363 ft at the basin exit for Q = 2,120 ft^3/s.

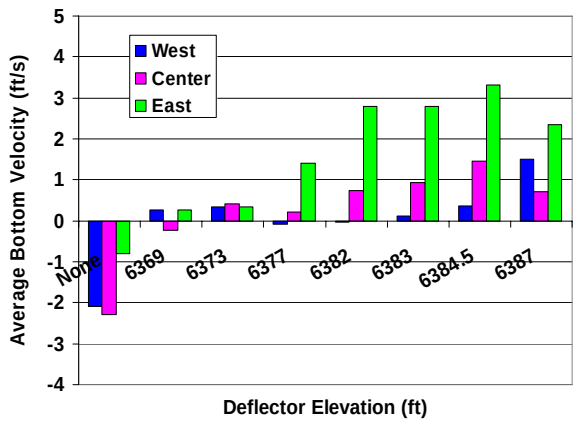

Figure 21. Average bottom velocity measured at El. 6363 ft at the basin exit for Q = 3,062 ft^3/s.

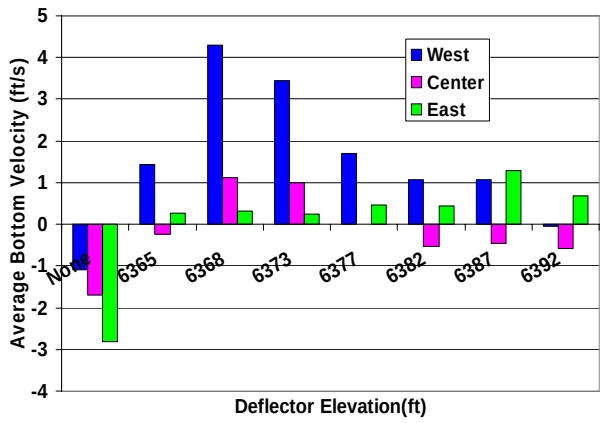

Figure 22. Average bottom velocity measured at El. 6363 ft at the basin exit for Q = 4,301 ft^3/s.

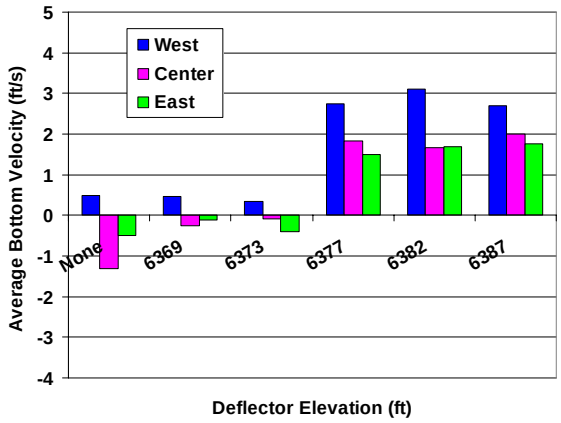

Figure 23. Average bottom velocity measured at El. 6363 ft at the basin exit for Q = 6,010 ft^3/s.

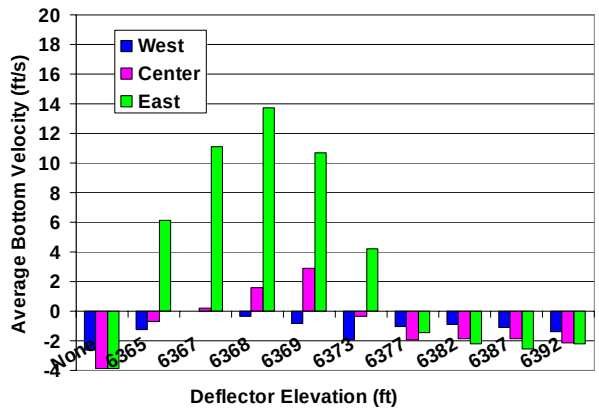

Figure 24. Average bottom velocity measured at El. 6363 ft at the basin exit for Q = 8,601 ft^3/s.

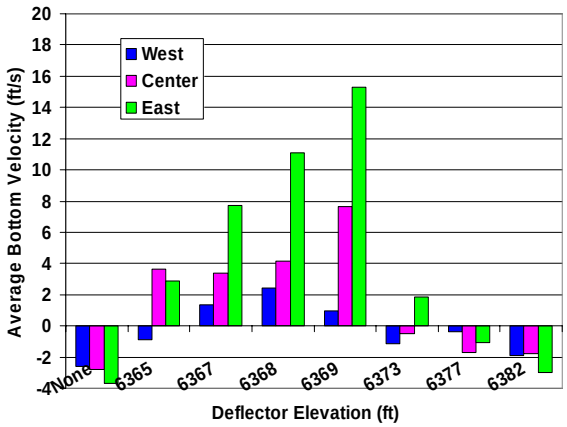

Figure 25. Average bottom velocity measured at El. 6,363 ft at the basin exit for Q = 12,020 ft^3/s.

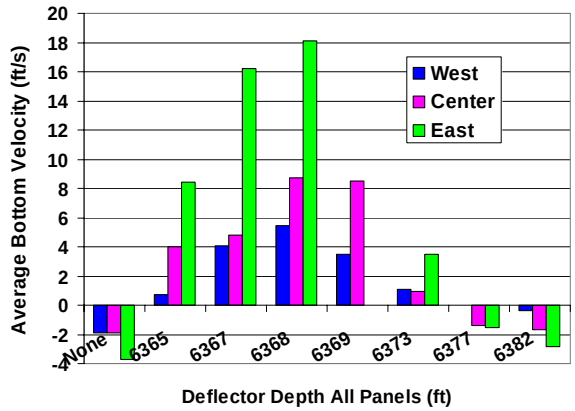

Figure 26. Average bottom velocity measured at El. 6363 ft at the basin exit for Q = 14,200 ft^3/s

At high discharges, the hydraulic jump is strong and a concentrated jet remains near the basin floor; thus, a low deflector elevation is needed to redirect flow.

Discharges in the range between 3,062 ft^3/s and 8,601 ft^3/s (figures 22 and 23) are in a transition zone where flow currents are difficult to define and deflector positioning is more difficult to determine. This transition zone occurs because flow magnitude increases from being small relative to the width of the basin to a magnitude that fills the width of the basin. As a result, the hydraulic jump transitions from being extremely strong and concentrated on one side of the basin into a jump that is weaker in strength as it spreads more uniformly across the width of the basin. At a flow of 4,301 ft^3/s, the hydraulic jump is strong but is predominantly concentrated on the west side of the stilling basin. Therefore, although the jump does not fill the basin in width or length and is concentrated on only one side, it acts similar to the higher flows with the jet remaining strong and low in elevation. As a result, the deflector must be low in elevation to redirect flow and produce good performance. At a discharge of 6,010 ft^3/s, the incoming jet begins to spread more uniformly across the width of the basin and the jump becomes weaker so that the downstream portion of the flow rises higher into the water column. Therefore, the deflector must be high in elevation to redirect flow and produce good performance. For discharges of 8,601 ft^3/s and above, the uniformly distributed jet is strong enough to remain low in elevation near the end of the basin, again requiring a low deflector elevation.

The data from figures 19-26 were used to derive the effectiveness of deflector performance for a moveable deflector that uses two positions to maximize performance over the operating range tested. Figure 27 shows deflector performance with the deflector positioned at elevation 6387 ft for discharges up to 6,010 ft^3/s and positioned at elevation 6368 for discharges greater than 6,010 ft^3/s. While operating at discharges above and below the transition zone, the moveable

deflector is effective in mitigating entrainment of materials, since flow patterns are fairly consistent and well defined. However, while operating within the transition zone, there is no guarantee that the deflector is positioned where it will be most effective. In addition, when the deflector is positioned at elevation 6368 ft for the higher discharges, downstream velocities are quite high. Average velocities at 14,200 ft^3/s are above 18 ft/s with maximum instantaneous downstream velocities as high as 23 ft/s. According to Reclamation's Engineering Monograph No. 25, velocities of this magnitude could cause significant movement of riprap, up to 4 ft in diameter, immediately downstream from the end of the stilling basin [3].

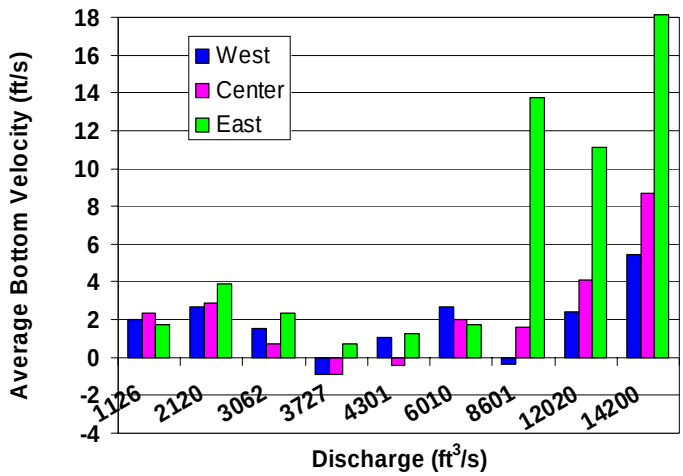

Figure 27. Performance of a single moveable deflector set at El. 6387 ft for discharges less than or equal to 6,010 ft^3/s and at 6,368 ft for discharges greater than 6010 ft^3/s.

If a single stationary deflector were used, best performance would be achieved with the deflector set at elevation 6387 (figure 28). This design would provide good performance for discharges up to 3,062 ft^3/s and would provide some effectiveness for discharges up to 6,010 ft^3/s. However, no significant improvement in performance would be provided for discharges above 6,010 ft^3/s. As a result, several other options, using two deflectors, were evaluated to see if overall performance could be improved.

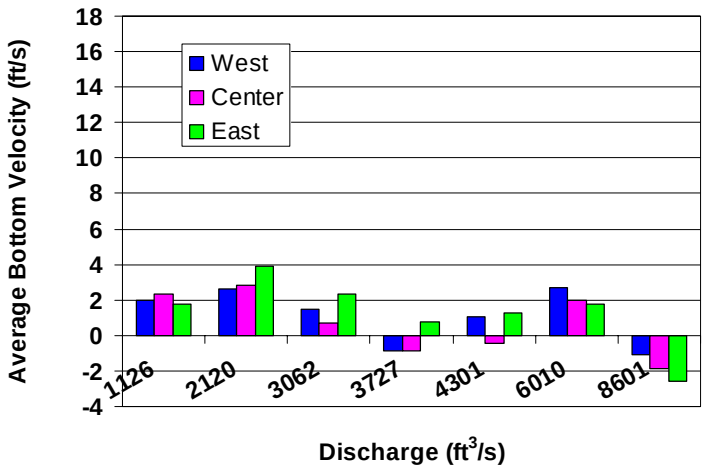

Figure 28. Single stationary deflector performance at elevation 6387 ft for discharges up to 8,601 ft^3/s.

Staggered Deflector Configurations

Four separate configurations were tested, using a two deflector staggered arrangement, to see if performance could be improved over that of a single span deflector (figure 29); however, only the two most effective configurations are presented here in detail.

Figure 29. View looking upstream at the end of the basin with a typical staggered deflector arrangement.

Option 1- Staggered Deflectors

The three original deflector panels were left in place and an additional flow deflector was installed, staggered both horizontally and vertically in position from the original panel unit.

The position for the new deflector panel was determined based on the results from the vertical velocity profiles collected for the existing basin and from guidelines generated previously from research using a staggered deflector arrangement [4]. The new panel was positioned vertically, 11 ft above basin floor elevation 6357 ft, and 6.65 ft upstream from the end of the stilling basin walls (figure 8). The new panel remained stationary and had an 8.5-ft vertical dimension and spanned the 62-ft-wide basin as a single unit. The original upper panels were then moved as a single unit in vertical increments, and bottom velocities at elevation 6363 ft were measured to determine best deflector performance (figures 30-37).

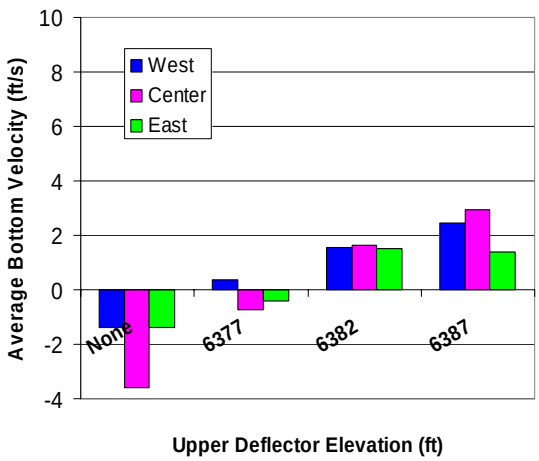

Figure 30. Option 1 - staggered deflectors — average bottom velocity measured at El. 6363 ft at the basin exit for Q = 1,126 ft^3/s.

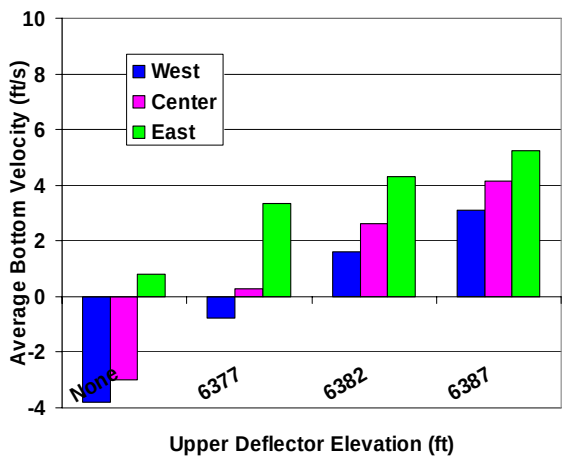

Figure 31. Option 1 staggered deflectors — average bottom velocity measured at El. 6363 ft at the basin exit for Q = 2,120 ft^3/s.

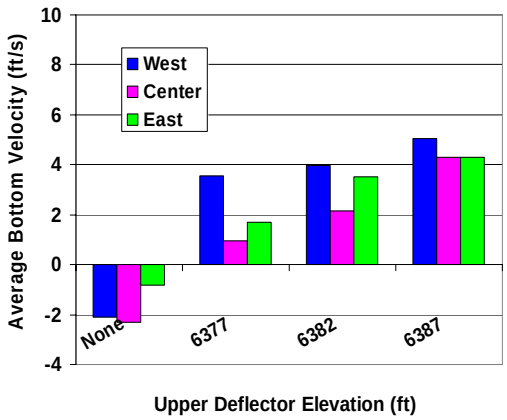

Figure 32. Option 1 - staggered deflectors — average bottom velocity measured at El. 6363 ft at the basin exit for Q = 3,062 ft^3/s.

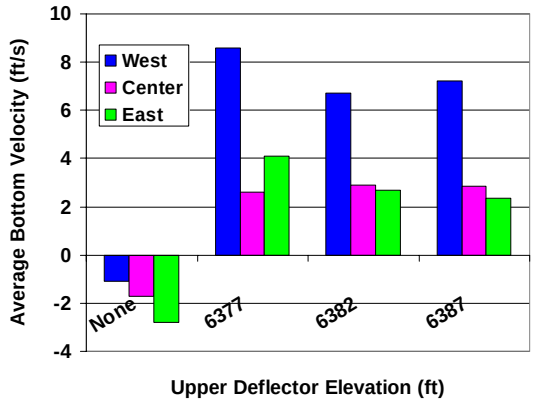

Figure 33. Option 1 - staggered deflectors — average bottom velocity measured at El. 6363 ft at the basin exit for Q = 4,301 ft^3/s.

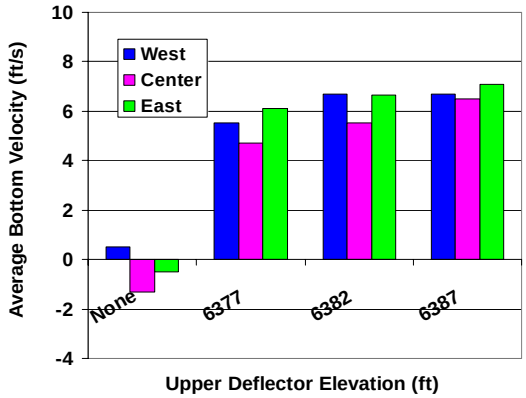

Figure 34. Option 1 - staggered deflectors— average bottom velocity measured at El. 6363 ft at the basin exit for Q = 6010 ft^3/s.

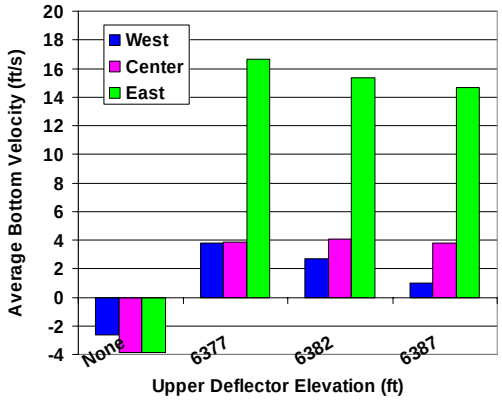

Figure 35. Option 1 - staggered deflectors - Average bottom velocity measured at El. 6363 ft at the basin exit for Q = 8601 ft^3/s.

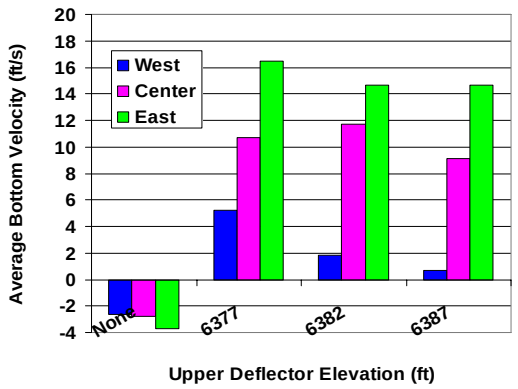

Figure 36 Option 1 - Staggered deflectors—average bottom velocity measured at El. 6363 ft at the basin exit for Q = 12,020 ft^3/s.

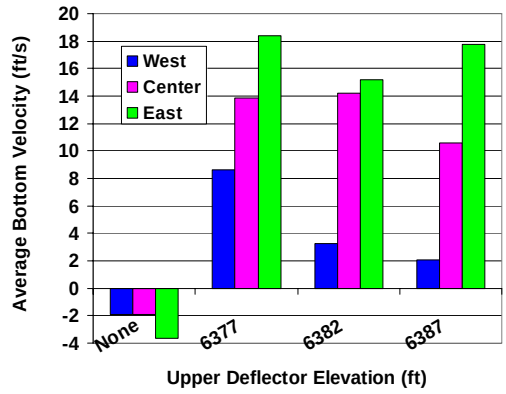

Figure 37. Option 1 - staggered deflectors — average bottom velocity measured at El. 6363 ft at the basin exit for Q = 14,200 ft^3/s.

Results from model investigations indicated that this arrangement of two flow deflectors staggered vertically and horizontally in position was a more effective arrangement than no deflector or a single deflector for all flows tested. After reviewing figures 30-37, it appeared that the dual deflector system could be fixed with the upper deflector positioned at elevation 6382 to produce the best overall performance. Velocity magnitudes for this arrangement are shown on figure 38. Deflector performance is better with this arrangement when compared with the performance of a single deflector.

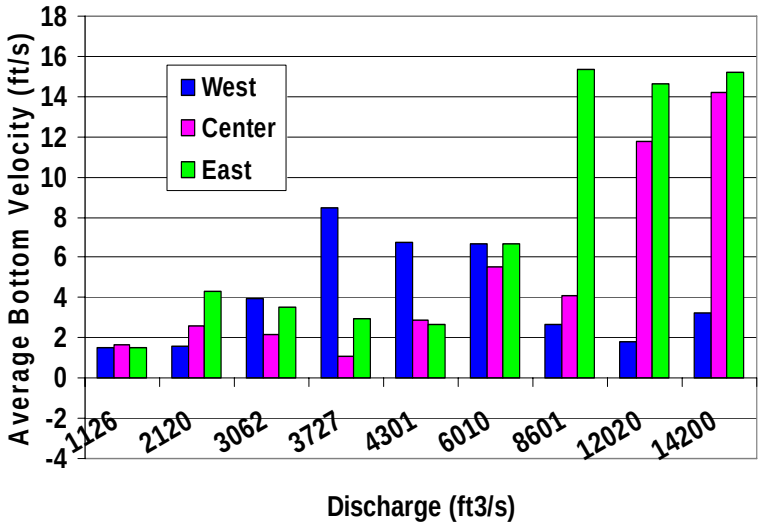

Figure 38. Option 1 - staggered deflector performance — deflector design with upper deflector vertical dimension 11.33 ft at elevation 6382 ft and lower deflector vertical dimension 8.5 ft at elevation 6368 ft.

However, after completion of these tests, concern arose regarding the magnitude of the velocities exiting the basin. Average velocities as high as 16 ft/s, with

maximum instantaneous velocities as high as 22 ft/s, were produced for discharges above 6,010 ft^3/s. The magnitude of velocities jumps significantly at a discharge of 8,601 ft^3/s because downstream flow has become strong and low in elevation at this discharge, so that the lower deflector is being utilized more fully to redirect flow downstream. Average velocities of this magnitude could cause movement of riprap as large as 3 ft in diameter. As a result, although this arrangement produced the best performance for preventing materials from being carried into the basin, it is not recommended because of the high potential for severe erosion. However, if geologic data for the area downstream from the basin presents evidence that severe erosion would not occur under velocities of this magnitude, this could be considered a viable option.

Option 2 - Optimal Staggered Deflector Arrangement

Additional testing was undertaken to investigate an alternative series of deflector sizes and locations that would lower the velocities exiting the basin to an acceptable level, while still providing effective performance. Best overall performance for each deflector configuration was evaluated and plotted. The graphs were then compared to determine which configuration produced the best overall performance throughout the operating range tested without producing downstream velocities of a magnitude that could cause severe erosion downstream from the basin.

Results from these tests showed that a two deflector staggered configuration, with the lower deflector reduced to a vertical dimension of 5.67 ft and raised in elevation to 6370.75 ft, or 13.75 ft above the basin invert provided best overall performance. Figures 39-45 show bottom velocities measured with the lower deflector stationary and the original upper panel unit moved in vertical increments.

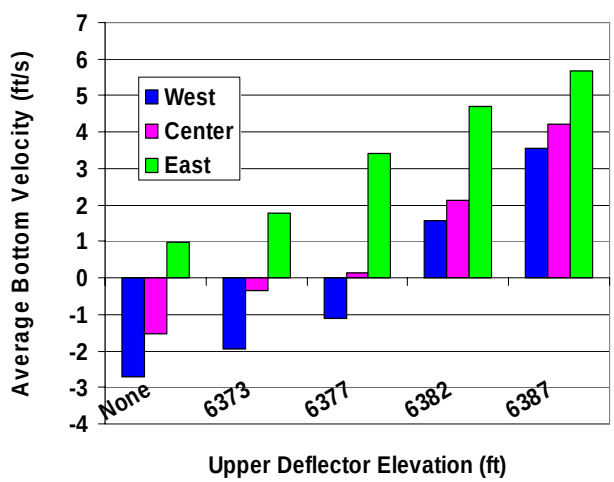

Figure 39. Option 2 - optimal staggered deflectors with lower deflector at elevation 6371 ft and reduced to 5.67 ft. Average bottom velocity measured at basin exit for Q = 2,120 ft^3/s.

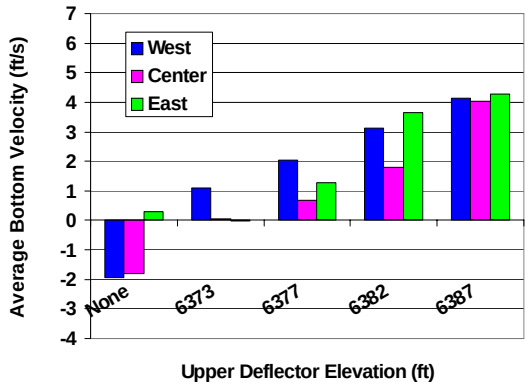

Figure 40. Option 2 - optimal staggered deflectors with lower deflector at elevation 6371 ft and reduced to 5.67 ft. Average bottom velocity measured at basin exit for Q = 3062 ft^3/s.

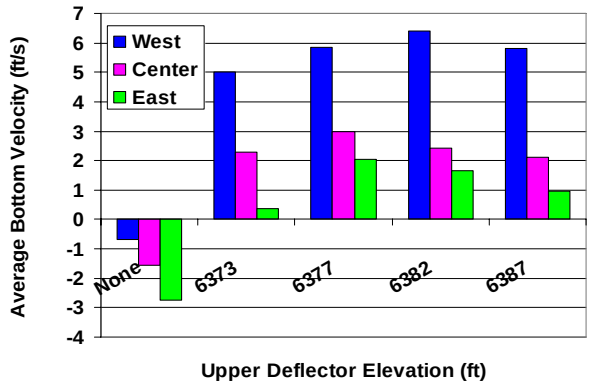

Figure 41. Option 2 - optimal staggered deflectors with lower deflector at elevation 6371 ft and reduced to 5.67 ft. Average bottom velocity measured at basin exit for Q = 4,301 ft^3/s.

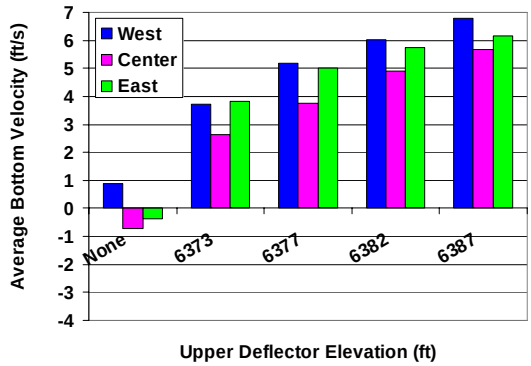

Figure 42. Option 2 - optimal staggered deflectors with lower deflector at elevation 6371 ft and reduced to 5.67 ft. Average bottom velocity measured at basin exit for Q = 6,010 ft^3/s.

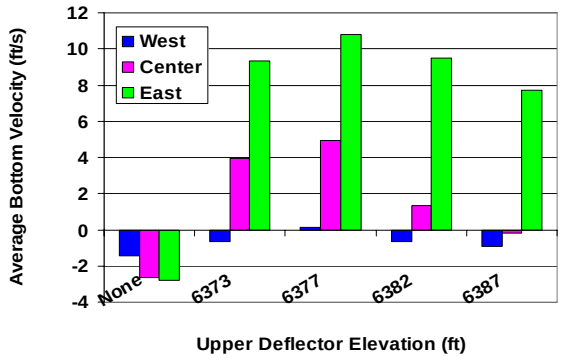

Figure 43. Option 2 - optimal staggered deflectors with lower deflector at elevation 6371 ft and reduced to 5.67 ft. Average bottom velocity measured at basin exit for Q = 8,601 ft^3/s.

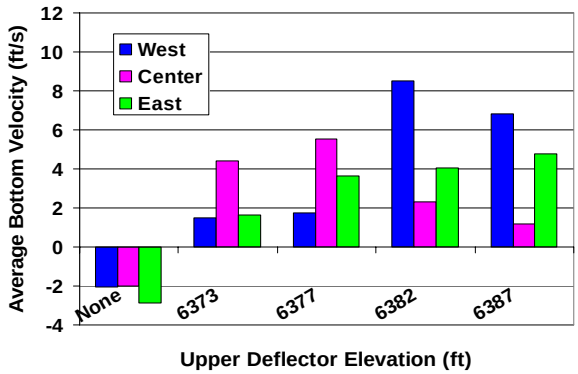

Figure 44. Option 2 - optimal staggered deflectors with lower deflector at elevation 6371 ft and reduced to 5.67 ft. Average bottom velocity measured at basin exit for Q = 12020 ft^3/s.

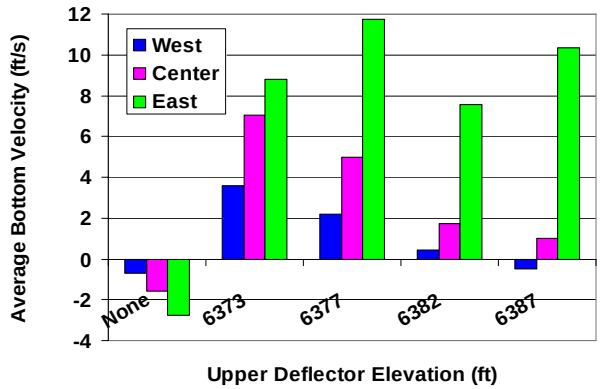

Figure 45 Option 2 – Optimal Staggered Deflectors with lower deflector at elevation 6371 ft and reduced to 5.67 ft. Average bottom velocity measured at basin exit for Q = 14200 ft^3/s

Best overall performance for this arrangement occurred with the upper deflector positioned at elevation 6382 ft. Figure 46 demonstrates that performance is good with this arrangement for discharges up to 6,010 ft^3/s. At discharges above 6,010 ft^3/s, average velocities indicate that some rocks less that 5 inches in diameter may be carried into the basin on the west side; however, high downstream velocities on the east side of the basin may help flush many of these materials out of the basin.

This could result in some abrasion damage; however, damage should be minimal compared to present damage experienced in the basin.

The highest average downstream velocity measured for this arrangement was 9.5 ft/s, with a maximum velocity from occasional flow surges as high as 15 ft/s to 18 ft/s. Velocities of this magnitude could cause some shifting of riprap; however, no significant erosion is expected for riprap greater than 18 inches in diameter. As a result of these findings, this is the final recommended deflector arrangement.

A brief description of two additional deflector configurations tested may be found in Appendices A and B.

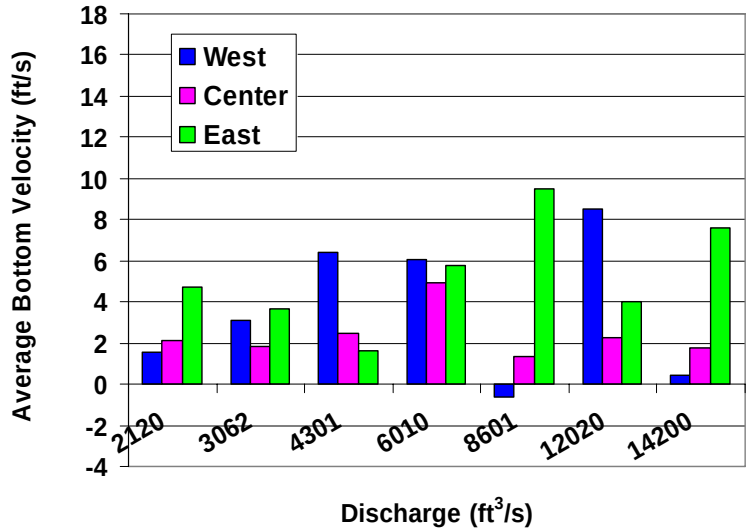

Figure 46. Option 2 - deflector performance for final recommended arrangement with upper stationary deflector vertical dimension 11.33 ft at elevation 6382 ft. Lower deflector with 5.67 ft vertical dimension at elevation 6370.7 ft.

Summary

Recommended Deflector Configuration

The following parameters summarize the final design for the recommended stationary, two deflector staggered configuration:

1) **Upper stationary deflector -** The upper deflector should be oriented vertically, with a vertical dimension of 11.33 ft, and positioned at elevation 6382 ft (referenced to the bottom of the deflector) and at a lateral location 13.33 ft upstream from the end of the stilling basin walls (referenced to the upstream face of the deflector).

2) **Lower stationary deflector** – The lower deflector should be positioned vertically, with a vertical dimension of 5.67 ft and positioned at elevation 6370.75 ft (referenced to the bottom of the deflector), and at a lateral location 6.65 ft upstream from the end of the stilling basin walls (referenced to the upstream face of the deflector).

Figure 47. View looking upstream at the end of the model basin operating at 6010 ft^3/s with the final staggered deflector configuration.

Figure 47 shows a view looking upstream at the model operating at 6,010 ft^3/s, with the final deflector configuration in place. Implementation of the recommended option should significantly reduce the amount of basin abrasion damage caused by upstream entrainment of material. However, it is also important that proper techniques in concrete repair be used [6]; otherwise, high-velocity flow could cause erosion and release of aggregate into the stilling basin. Although pieces of aggregate may be small, compared with riprap previously entrained in the basin, and although much of this material may be flushed from the basin over time, some abrasion damage may occur.

It is also important to note that past studies have shown that large variations in tailwater from the design values tested may affect the performance of the deflector. Past studies have shown that tailwater depths 5 to 10 percent higher

than the design value can significantly reduce the performance of the deflector, although performance was still improved compared to having no deflector in the stilling basin. Deflector performance was not significantly reduced for tailwater depths in the range of 5 to 10 percent below design value.

In addition, although no significant abrasion and downstream erosion are expected, it may be beneficial to have divers inspect the basin and downstream riprap after the basin has operated at discharges above 6,010 ft^3/s for an extended period of time or after high tailwater conditions have existed.

Finally, research in the development of flow deflectors for mitigating abrasion damage in wide-span basins greater than 25 ft in width is ongoing and may, in the future, result in the development of other solutions.

Deflector Loading

The final recommended deflector configuration consists of two stationary deflectors, spanning the width of the 62-ft wide basin. As a result, structural design of the deflector and its supports becomes important. The addition of one or two piers within the basin may also be required for the structural support of each deflector. Therefore, loading across the deflectors from upstream to downstream was investigated for structural design use.

Piezometer taps were installed on the upstream and downstream faces of the recommended upper and lower deflectors to measure differential static hydraulic loading. Taps 1 through 12 were installed equally spaced across the upper deflector from west (1) to east (12) and alternated facing upstream and downstream so that each adjacent pair would measure a pressure drop across the deflector (from upstream to downstream). Taps 13 through 24 were installed in a similar manner on the lower deflector with tap 13 closest to the west wall.

Pressure differentials at each location were measured for the same operating conditions tested previously and are shown in figures 48 and 49, respectively, for the upper and lower deflectors. The greatest pressure drop measured across the upper deflector was about 4.6 lb/in^2 and occurred at taps 11and 12, nearest the east wall, at a discharge of 14,200 ft^3/s. The greatest pressure drop measured across the lower deflector was about 8 lb/in^2 and occurred at taps 23 and 24, again nearest the east wall at a discharge of 14,200 ft^3/s.

In addition to the pressure drop across the deflectors (from upstream to downstream), figures 48 and 49 show that pressure drop along the span of each deflector, from west to east, is not uniform. As a result, there is also a differential load along the length of the deflector spanning between the basin walls. For discharges ranging from 1,126 ft^3/s to 4,301 ft^3/s, the pressure drop measured varies from high to low traveling from the basin west wall to the east wall. At

discharges greater than 6,010 ft^3/s, the differential load shifts so that the highest load is near the east wall. This pressure variation occurs because the hydraulic jump shifts in strength and from one side of the basin to the other as basin operations pass through the transition zone, as discussed in earlier sections.

Also note that for design purposes, that these differential pressure values do not include dynamic loading and do not include a factor of safety.

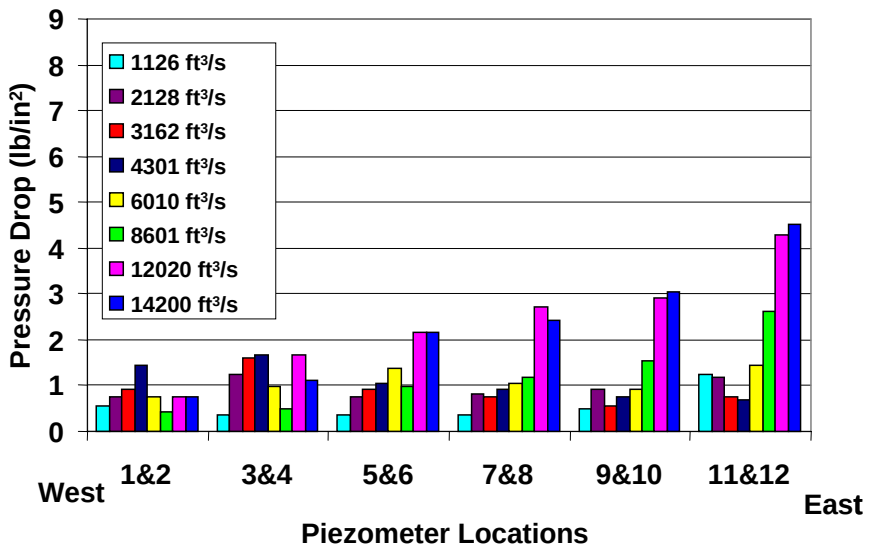

Figure 48. Pressure drop measured across upper deflector from west (taps 1 and 2) to east (taps 11 and 12).

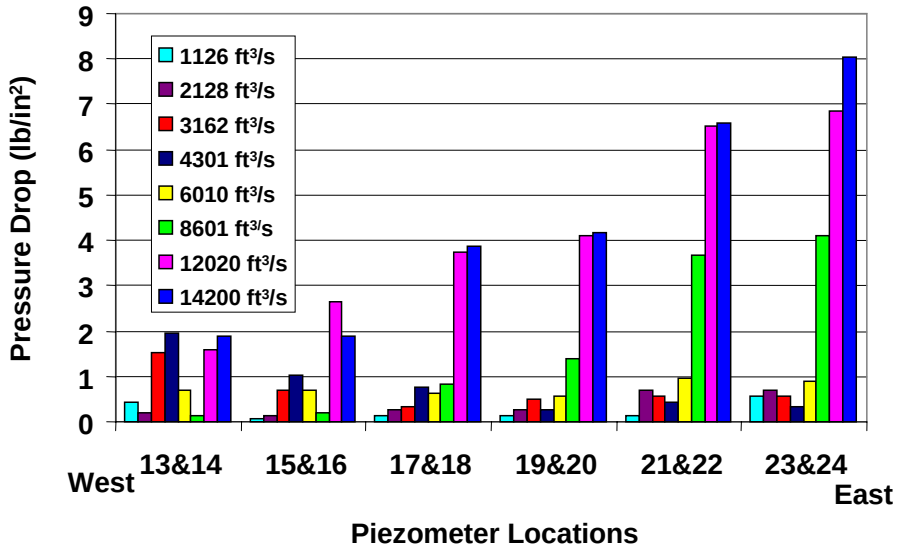

Figure 49. Pressure drop measured across lower deflector from west (taps 13 and 14) to east (taps 22 and 23).

References

1. Hanna, Leslie, “Mason Dam Flow Deflectors for Preventing Abrasion Damage,” Hydraulic Laboratory Report HL-2005-01, Bureau of Reclamation, Denver, Colorado, October 2005

2. “Inspection of Outlet Works Stilling Basin,” by Western Divers LTD, Casper Wyoming, November 2004.

3. Peterka, A.J., “Hydraulic Design of Stilling Basins and Energy Dissipaters,” Engineering Monograph No. 25, United States Department of Interior, Bureau of Reclamation, Denver, Colorado, May 1984.

4. Hanna, Leslie, “Generalized Design of Flow Deflectors for Mitigating Abrasion Damage in Stilling Basins,” Hydraulic Laboratory Report - Draft, Bureau of Reclamation, Denver, Colorado, 2007

5. Designer’s Operating Criteria, Fontenelle Dam and Power Plant, Seedskadee Project Wyoming, Office of Design and Construction, United States Department of Interior, Bureau of Reclamation, Denver Colorado, July 1972.

6. “Guide to Concrete Repair,” United States Department of Interior, Bureau of Reclamation, Denver, Colorado, August 1996.

Appendix A

Option 3 Velocity Data - Staggered Deflectors

A staggered arrangement was tested with a lower deflector vertical dimension of 5.67 ft and positioned at elevation 6368 ft, or 11 ft above the basin invert. The upper deflector panel unit had a vertical dimension of 11.33 ft. Bottom velocities were measured with the upper panel unit moved in vertical increments to determine best performance. Best overall performance for this configuration occurred with the upper deflector positioned at elevation 6382 ft (figure A-1). This arrangement produced effective performance for discharges up to 6,010 ft^3/s. However, performance at higher discharges was not as good as for the recommended arrangement with negative average velocities produced on the west side for all discharges above 6,010 ft^3/s. In addition, average downstream velocities on the east side were high enough to cause movement of riprap almost 3 ft in diameter. Bottom velocities measured as a function of deflector elevation for each discharge are provided in figures A-2 through A-8.

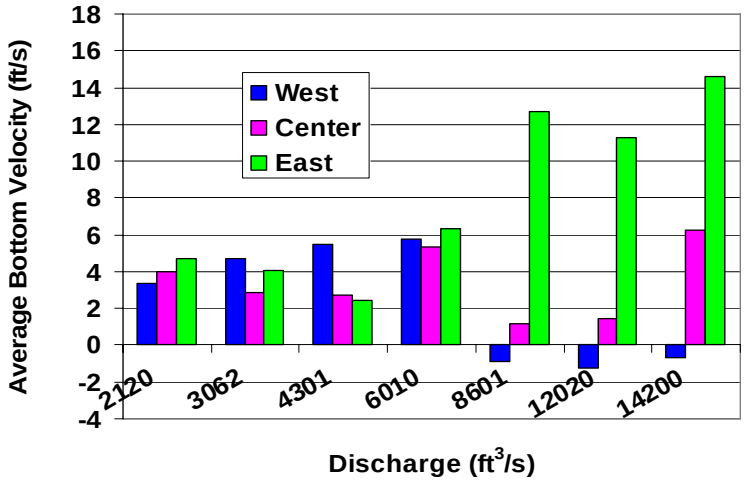

Figure A-1. Option 3 - deflector performance for staggered deflector arrangement, with the upper deflector vertical dimension of 11.33 ft and positioned at elevation 6387 ft and the lower deflector with vertical dimension of 5.67 ft and positioned at elevation 6368 ft.

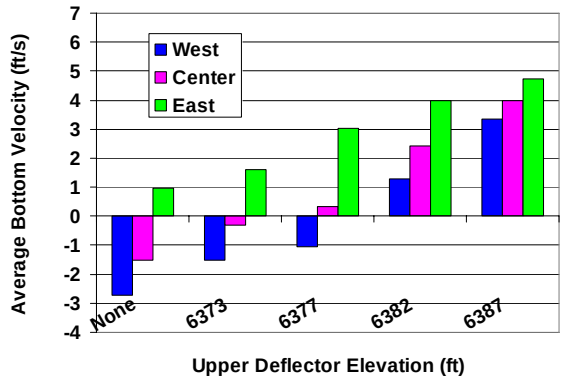

Figure A-2. Option 3 - staggered deflectors with lower deflector reduced to 5.67 ft. Average bottom velocity measured at basin exit for Q = 2,120 ft^3/s.

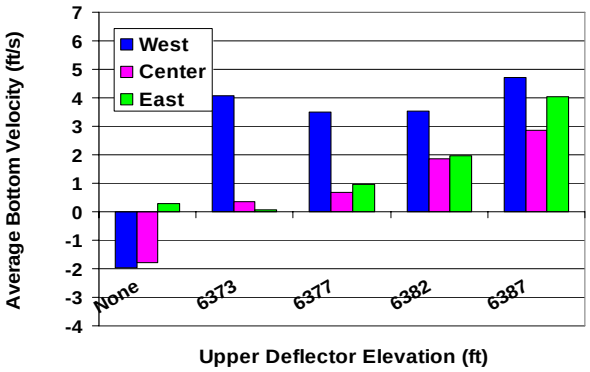

Figure A-3. Option 3 - staggered deflectors with lower deflector reduced to 5.67 ft. Average bottom velocity measured at basin exit for
Q =3,062 ft^3/s.

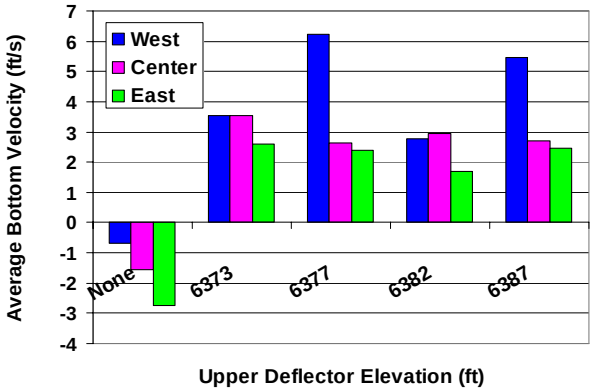

Figure A-4. Option 3 - staggered deflectors with lower deflector reduced to 5.67 ft. Average bottom velocity measured at basin exit for
Q = 4,301 ft^3/s.

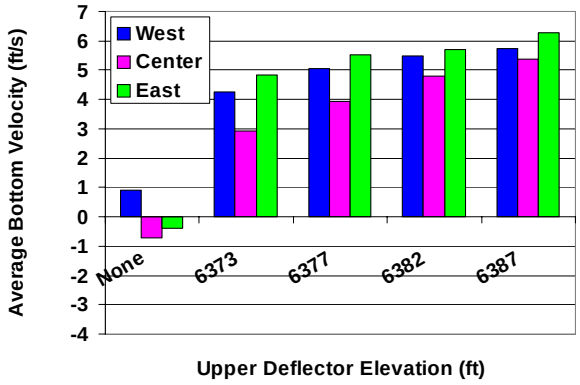

Figure A-5 Option 3 - staggered deflectors with lower deflector reduced to 5.67 ft. Average bottom velocity measured at basin exit for
Q = 6010 ft^3/s.

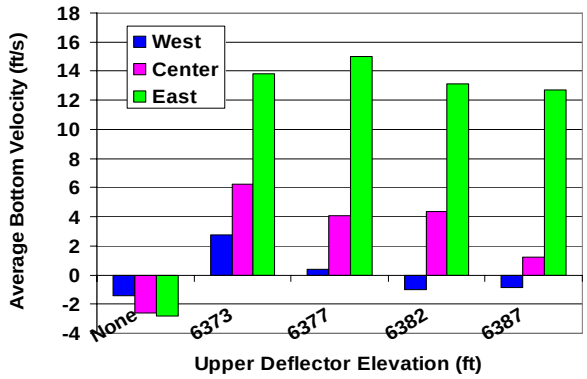

Figure A-6. Option 3 - staggered deflectors with lower deflector reduced to 5.67 ft. Average bottom velocity measured at basin exit for
Q = 8,601 ft^3/s.

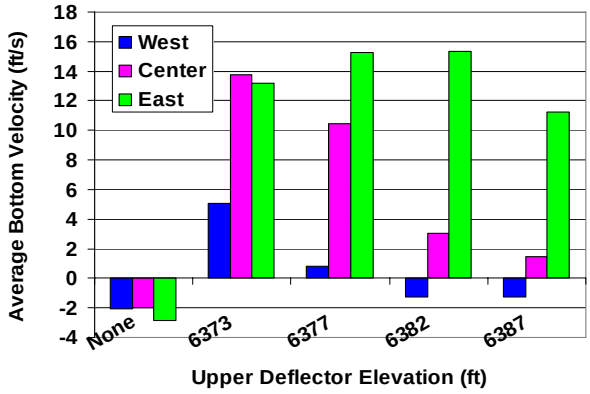

Figure A-7. Option 3 - staggered deflectors with lower deflector reduced to 5.67 ft. Average bottom velocity measured at basin exit for
Q = 12,020 ft^3/s.

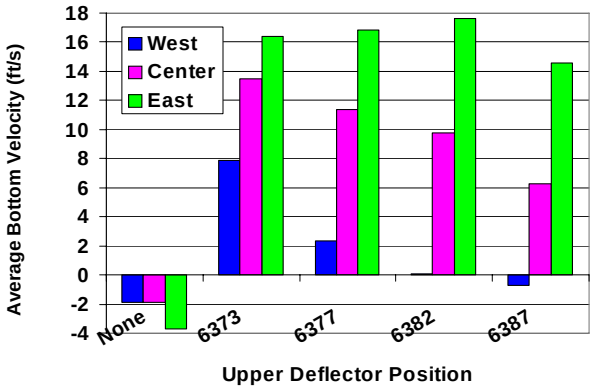

Figure A-8. Option 3 - staggered deflectors with lower deflector reduced to 5.67 ft. Average bottom velocity measured at basin exit for
Q = 14,200 ft^3/s.

Appendix B

Option 4 Velocity Data - Staggered Deflectors

A staggered arrangement was tested with the upper panel unit reduced to a vertical dimension of 8.5 ft. The lower deflector vertical dimension was 8.5 ft and was positioned at elevation 6368 ft, or 11 ft above the basin invert. Bottom velocities were measured with the new upper panel unit moved in vertical increments to determine best performance. Best overall performance for this configuration occurred with the upper deflector positioned at elevation 6387 ft (figure B-1). Performance for discharges up to 4,301 ft^3/s was reduced compared to the recommended arrangement and velocities measured at discharges above 6,010 ft^3/s were high enough to cause movement of riprap nearly 3 ft in diameter. Bottom velocities measured as a function of deflector elevation for each discharge are provided in figures B-2 through B-9.

These graphs represent the lower deflector with vertical dimension of 8.5 ft at elevation 6371 ft and the upper deflector with vertical dimension reduced to 8.5 ft.

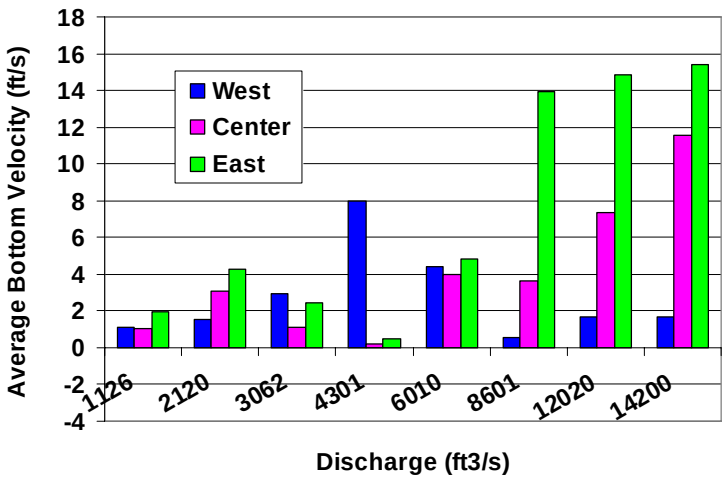

Figure B-1 Option 4 - Deflector performance for staggered deflector arrangement, with the upper deflector reduced in size to an 8.5 ft vertical dimension and positioned at elevation 6387 ft. Lower deflector with an 8.5 ft vertical dimension and positioned at elevation 6368 ft.

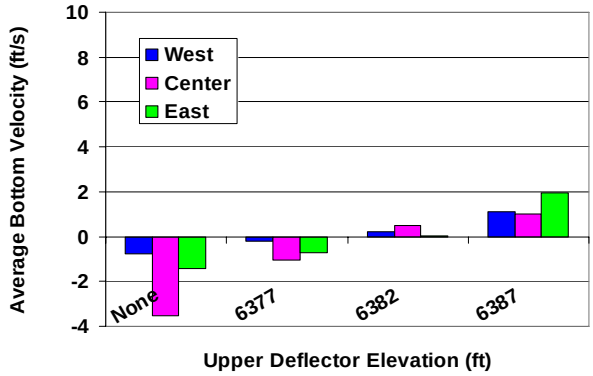

Figure B-2. Option 4 - staggered deflectors with upper deflector reduced to 8.5 ft. Average bottom velocity measured at basin exit for Q = 1,126 ft^3/s.

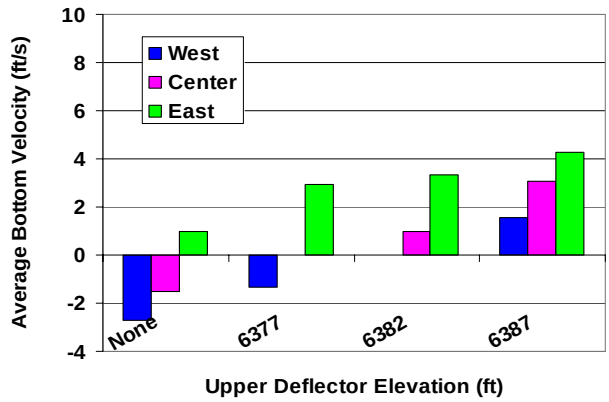

Figure B-3. Option 4 - staggered deflectors with upper deflector reduced to 8.5 ft. Average bottom velocity measured at basin exit for Q = 2,120 ft^3/s.

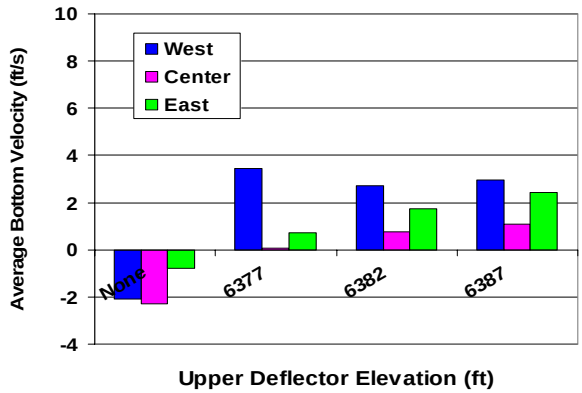

Figure B-4. Option 4 - staggered deflectors with upper deflector reduced to 8.5 ft. Average bottom velocity measured at basin exit for Q = 3,062 ft^3/s.

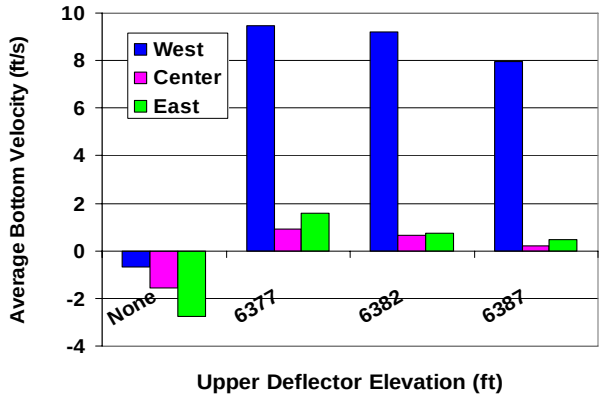

Figure B-5. Option 4 - staggered deflectors with upper deflector reduced to 8.5 ft. Average bottom velocity measured at basin exit for Q = 4,301 ft^3/s.

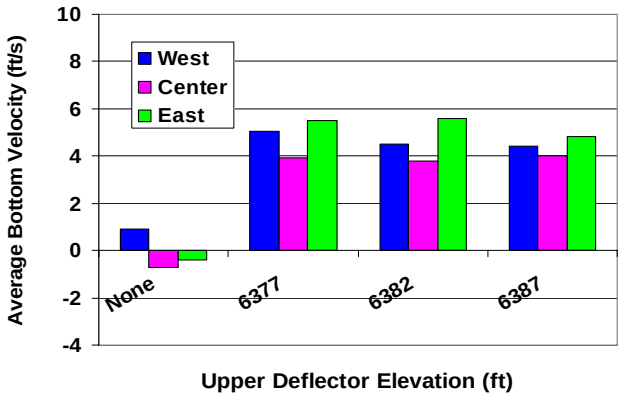

Figure B-6. Option 4 - staggered deflectors with upper deflector reduced to 8.5 ft. Average bottom velocity measured at basin exit for Q = 6,010 ft^3/s.

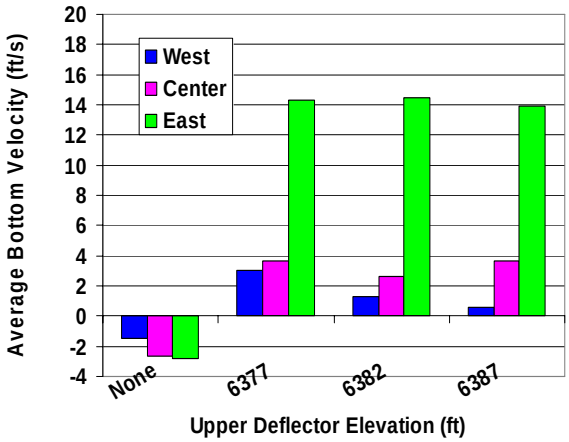

Figure B-7. Option 4 - staggered deflectors with upper deflector reduced to 8.5 ft. Average bottom velocity measured at basin exit for Q = 8,601 ft^3/s.

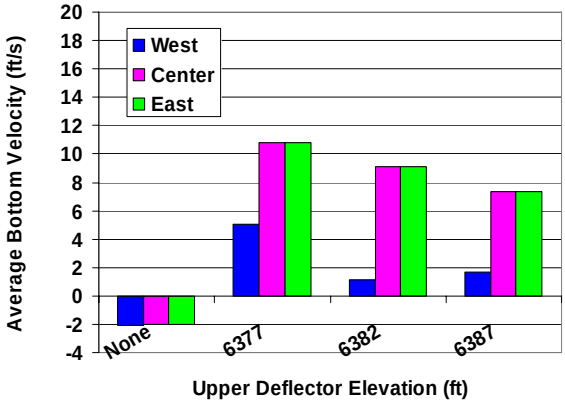

Figure B-8. Option 4 - Staggered deflectors with upper deflector reduced to 8.5 ft. Average bottom velocity measured at basin exit for Q = 12,020 ft^3/s.

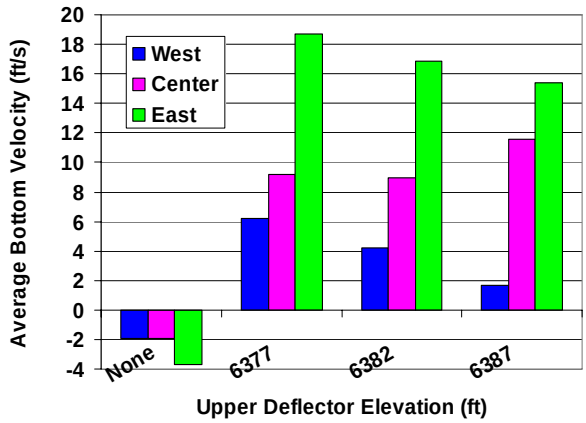

Figure B-9. Option 4 - staggered deflectors with upper deflector reduced to 8.5 ft. Average bottom velocity measured at basin exit for
Q = 14,200 ft^3/s.