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Hydraulic Model Study of the Grand Valley Irrigation Company Fish Screen Structure

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Background

A fish screen facility was installed in the Grand Valley Irrigation Company (GVIC) canal just downstream of the canal head gates on the Colorado River, figure 1. This canal is very old with water rights dating back to 1886. The fish screen facility was installed to reduce or prevent entrainment of threatened adult pikeminnow and razorback chub into the canal. The design approach velocity to the screen is 0.5 ft/s with a minimum sweeping velocity of 2 ft/s. The design flow rate into the canal is 680 ft^3/s with a minimum water surface elevation upstream from the head gates of El. 4681 or the top of the diversion dam in the river.

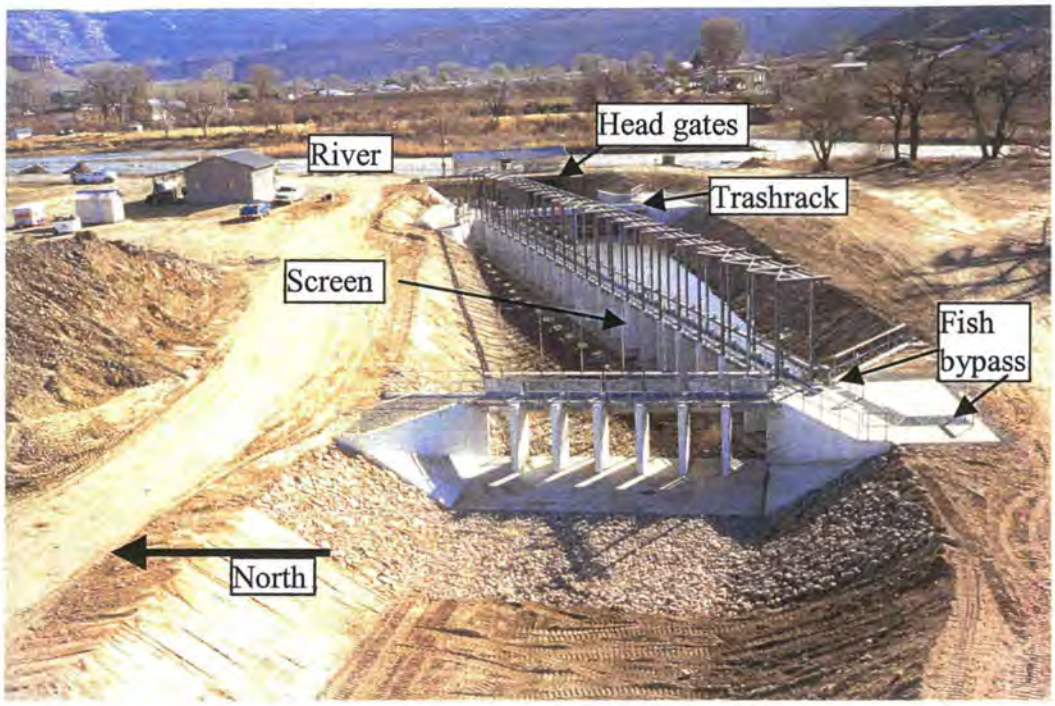


Figure 1. - GVIC screen structure shortly after construction in 2002. The canal head gates are on the Colorado River at the top of the picture with the screen structure situated diagonally across the canal.

The first season of screen operation revealed a problem with flow distribution heavily skewed to the left (looking downstream) through the trashrack in the canal section upstream from the screen. Observations of the flow conditions at the screen indicated that the sweeping velocity was non-existent in front of many of the upstream screen panels and that flow accumulated at the downstream end of the screen. Velocity measurements taken along the screen structure, with baffles adjusted, revealed that the approach velocity distribution is very non-uniform and sweeping velocities are very low causing approach velocity criteria to be exceeded and severe debris fouling issues. Therefore, Reclamation's Grand Junction Area Office requested a hydraulic model study be performed to determine the best solution for improving the screen flow conditions.

Objective

The objective of the hydraulic model study is to investigate the most efficient way to perform gate operational changes and/or structural modifications to the GVIC canal geometry to improve the flow conditions approaching the screen. Improvement of the general flow conditions would help bring the screen into conformance with approach velocity criteria and improve screen sweeping velocity and debris handling capability.

Recommendations

The location of the screen on the inside of a bend in the canal and the large flow area of the canal has produced most of the poor screen flow conditions. Flow entering the head gates from the river has to be redirected toward the right (looking downstream) or the north side of the canal upstream from the screen structure.

The location of the screen on the inside of the bend in the canal will never produce ideal sweeping velocities. However, the following recommendations from the hydraulic model studies will greatly improve screen performance:

- Install a wall on a straight line at about an 8-degree angle from the downstream end of the south pier wall to the left side of the bypass opening. The wall should be vertical in the upstream portion of the canal from the pier wall, past the trashrack bridge to the beginning of the existing slope adjacent to the screen section. From this point to the bypass entrance the wall can be vertical or sloped on the same 2:1 slope as the existing slope and matched to the bypass entrance as needed. The wall should be at least the height of the maximum water surface or about 10 ft. This modification both reduces the canal flow area and redirects the flow from the left head gates.
- Install a vertical deflector wall 21-ft-long from the center head gate pier at a 15.5-degree angle to the north. Again, this wall should be at least the height of the maximum water level. The deflector wall redirects the flow from the right gates to the north side of the canal.
- Install a vertical wall on the north side of the canal in front of the sloped area upstream from the bridge deck to eliminate the eddy zone and reduce head loss.
- Screen baffles should be used only to mitigate minor "hot spots" or higher approach velocity zones on the screen. With improved general flow conditions use of the baffles should be minimized.

In addition, the initial design of the screen with an approach velocity of 0.5 ft/s and locating the screen with the offset away from the canal wall cannot be overcome, by these geometry changes alone. A smaller design approach velocity would have produced a larger screen area and less debris issues, but would have been more expensive. The 1.38 ft offset of the screen away from the canal wall will reduce the effectiveness of the first screen bay.

Design Parameters

The original screen was designed with an approach velocity of 0.5 ft/s and an assumed sweeping velocity of 2 ft/s. This represents a 4:1 sweeping to approach velocity ratio. Higher sweeping to approach velocity ratios produce better screen debris handling capability. The length of the screen is 254 ft. The screen was designed with cleaning brushes and air burst mechanisms to remove debris from the screen. GVIC measures the flow at a rated section downstream from the screen check structure that was installed with the screen. The flow requirement for deliveries is measured at the rated section. The original design discharge for the screen was 640 ft^3/s with about 40 ft^3/s going through the fish bypass at the end of the screen. Later discussions indicated that the water removed from the river at the head gates might be as much as 680 ft^3/s with about 60 ft^3/s being sent back to the river through the fish bypass. The minimum elevation upstream from the head gates is the elevation of the river diversion dam at El. 4681 ft. As water flows over the diversion dam, the head increases up to a normal maximum of El. 4686 ft.

Available head is an issue at the structure, therefore, whatever changes are constructed in the canal need to maintain the same head loss or, if possible, reduce head loss.

Operation and Maintenance Features

A well designed and place fish screen when operated correctly should produce little, if any, head loss in the system. The GVIC screen was designed with a 0.5 ft/s approach velocity. Typically, screens are designed with an approach velocity of about 0.2 to 0.33 ft/s, depending upon the species and life stage of the fish targeted by the screening structure. The higher approach velocity was felt adequate for adult pikeminnow and razorback chubs. There is nothing that can be physically done about the higher than typical design approach velocities except to enlarge the screen area. This is not an option at this point. Operationally, the higher, less conservative, approach velocity means that debris in the system will more likely be a problem.

The design sweeping to approach velocity ratio is 4:1, providing a sweeping velocity of 2 ft/s. This sweeping to approach velocity ratio should be adequate, although recent studies have indicated that even higher ratios improve screen performance. The poor hydraulics approaching the GVIC screen section, the location of the screen on the inside of the canal bend, and the large canal area prevents the flow conditions necessary for the screen to see the design sweeping velocity, thus not allowing adequate handling of debris.

The location of the screen, on the inside of the bend in the canal, is never going to produce ideal sweeping velocities; however, the recommended modifications will greatly improve the overall flow conditions to the screen. Improving the uniformity of flow to the screen will produce the intended sweeping of the flow and debris past the screen. Once sweeping is occurring then the small areas of non-uniform approach flow or "hot spots" on the screen can be handled with the baffling that has been provided.

Even with improvements to the flow conditions, the screen must be clean to perform properly. The cleaning brushes must be operated with the air burst system as necessary to keep the screen free of debris. The debris in the system varies with low and high water years. During low water

years, as has occurred recently, the major problem is algae. Algae are one of the most difficult debris problems for screens. Buildup of debris that plugs a portion of the screen increases approach velocities on the remaining portion of the screen, compounding the problem. Therefore, the cleaning system might need to be operated continuously to keep up with removal of algae. Continuous operation of screen cleaning systems is not unusual and will enhance screen performance.

In the previous irrigation season, the screen clogged with debris and panels were removed. This produced a high velocity flow through a small opening in the screen and erosion of the bank behind the screen structure. This will not occur once the flow conditions are improved, increasing and making sweeping velocities more uniform and appropriate cleaning of the screen is accomplished. A clean screen with higher sweeping velocities will minimize buildup of head on the upstream side of the screen and head loss across the screen.

The offset of the screen back from the wall to keep the cleaning brushes out of the flow when parked will always cause some separation and a dead zone in front of the first screen bay.

The Model

The model was constructed to a 1:15 Froude scale. The model included a short portion of the river upstream and the improvements constructed just upstream from the canal head gates, the canal head gate structure, the canal geometry with the screen structure including perforated plate with the same porosity as the prototype wedge wire, and the downstream check structure. In addition, the model included the concrete jetty on the right bank of the river and the debris boom at the upstream end of the concrete entrance section. The model was constructed to the as-built drawings, photographs, and other information provided by the Grand Junction Area Office.

It was agreed to not include the bypass pipe or the baffling behind the screen structure. Both these items added additional complexity to the model that was not needed to investigate the primary problem related to the flow conditions approaching the screen. The bypass will increase sweeping velocities, so if sweeping velocities seem appropriate without the bypass then operational efficiency should improve with the bypass operating in the prototype. Baffling is only used to correct small areas of discontinuity or "hot spots" on the screen and will not solve the major issue of poor flow conditions upstream from the screen. Therefore, the model will not have the flexibility of the prototype, but will address the major flow issues.

It was also agreed to not model the river sluice structure so that all the river flow is taken into the canal. Different river stage or head values can be used as necessary to simulate the effect of larger river flows on the head gates and canal discharges and velocities.

Test Plan

The model was operated to bring the total canal and bypass flow into the canal even though the bypass was not modeled. This meant conservative results for the screen behavior because more flow was going through the screen than if the bypass were operating. The initial model results were obtained by operating the model to provide a flow of 640 ft^3/s in the canal downstream

from the bypass pipe with a minimum water surface in the river upstream from the canal head gates of El. 4681. The water surface in the canal downstream from the screen will be set at El. 4679.6. Operations with increased head that would represent increased river flow will be investigated as necessary. Primary emphasis will be placed on optimum operation with canal flow of 640 ft^3/s under low river flow conditions.

The following tests will be performed:

- Calibrate the model to the field data provided and observations made by the Grand Junction Area Office personnel.
- Evaluate flow conditions and gather velocity data with the as-built geometry and current operating conditions.
- Perform gate opening sequencing at the canal head gates and evaluate flow conditions in the canal.
- Investigate possible placement of flow vanes at various locations in the channel upstream of the screen that would produce more uniform flow distribution to the screen.
- Investigate the influence of canal geometry modifications on the flow distribution and screen approach and sweeping velocity ratios.
- Investigate means of increasing the screen sweeping velocity, including increasing the head upstream from the head gates.

Data Gathered - Initial impressions of the flow conditions and many of the modifications were made by only flow visualization techniques including dye, surface floats and spot velocity measurements. Minimal documentation was performed for many of the quick attempts to improve flow conditions, as the option was clearly ineffective. More data were gathered once modifications looked promising and refinements were being accomplished.

Velocity data were initially obtained with a propeller meter along the upstream face of the trashrack bridge deck. Velocities were taken at the bridge location to obtain information about improvements made to the canal geometry prior to reaching the screen area. Velocity data were taken at 0.6 tenths depth only. In addition, spot checks of sweeping velocity in front of the first few screen bays were made with some upstream modifications to determine if the modification was effective.

Sweeping velocities were also measured along the screen with the propeller meter for the initial condition and as the modifications became more finalized. The velocities measured in front of the screen are only an indication of what can be expected since the bypass and baffling was not included in the model. Measured sweeping velocities should be higher in the prototype with bypass flow and more uniformity of approach velocity should be attained with baffle adjustments.

For the final few modifications, the velocities were measured with an acoustic velocity meter that measures in two dimensions. These velocity measurements would be more accurate because the meter does not need to be perfectly aligned with the flow. In addition, both the sweeping component and the approach component can be measured simultaneously.

The final geometry was also tested for 680 ft^3/s coming into the canal with river water surface El. 4686 and water surface elevation downstream from the canal of El. 4679.6. This would increase the flow velocities in the canal. The final deflector geometry with the sloping and vertical left wall geometries were verified with this canal discharge.

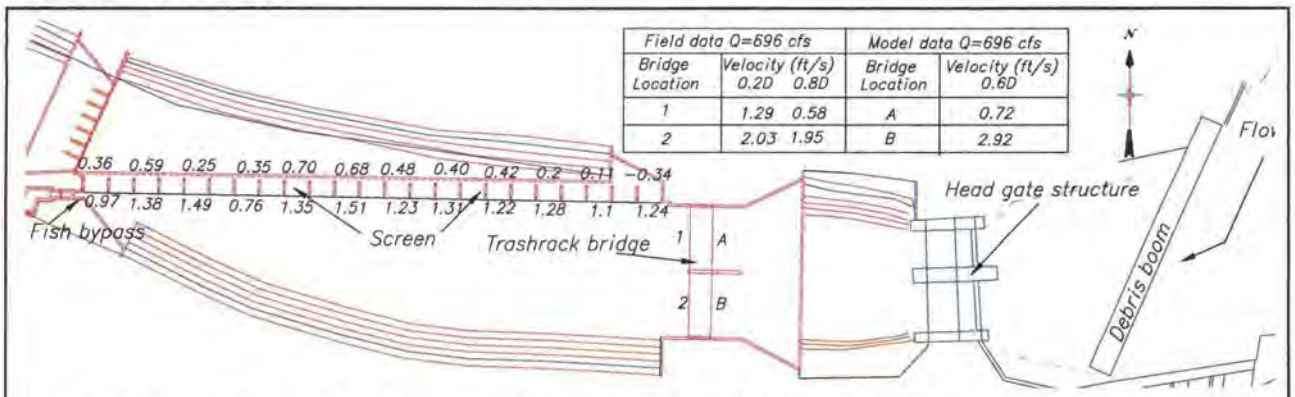
Head loss data were not gathered in the model, due to the very small differentials that existed. Any modification that streamlined the flow or reduced eddy formation would decrease the head loss through the system.

Investigations

The investigations began by ensuring that the geometry and flow conditions seen in the field were similar in the model. Figure 2 shows a schematic plan view of the existing canal geometry. Convention holds that left and right (or south and north for the GVIC screen) are directions looking downstream or in the direction of flow in the river and canal.

Model Verification Tests - Field velocity data were measured with a flow of 696 ft^3/s in the canal, a water surface elevation of 4681 upstream of the head gates and an elevation of 4680.2 in the channel upstream from the screen. Field personnel measured sweeping and approach velocities between the screen and the baffles with the baffles adjusted to improve usage of the screen area and from the bridge deck at the trashrack structure in the canal upstream from the screen. Reported field velocities are averaged from velocities measured at 0.2D and 0.8D with D equal to depth. Model velocities are reported for 0.6 percent of the flow depth in the canal at the bridge location.

The average field sweeping and approach velocities are shown in front of and behind the screen respectively, in figure 2. Unfortunately, the screen velocities were taken with the baffles adjusted by various amounts, from fully closed at the downstream end to fully open at the upstream end. Use of the screen baffles helped even out the sweeping velocities and actually produced usage of most of the upstream screen bays. The sweeping velocities were; however, very low, never attaining the 2 ft/s assumed for the design. The approach velocities were very high at points on the screen and actually showed reverse flow through the screen in the most upstream or first bay.



Field data Q=696 cfs			Model data Q=696 cfs	
Bridge Location	Velocity (ft/s) 0.2D	0.8D	Bridge Location	Velocity (ft/s) 0.6D
1	1.29	0.58	A	0.72
2	2.03	1.95	B	2.92

Figure 2. - Verification tests for the model operation with Q = 696 ft^3/s. Field screen sweeping (in front of screen) and approach (behind screen) velocities are reported with the screen baffles adjusted. All field velocities are averaged with depth. (verify.wmf)

It appeared in the field that the flow was piling up at the downstream end of the screen. To investigate the consequences of this, the baffles behind screen bay 21, second from the downstream end of the screen, were fully opened and a velocity measurement taken. An approach velocity measurement of 1.5 ft/s and a sweeping velocity of 0.9 ft/s were obtained. All these measurements indicated a very serious problem with flow distribution and the sweeping versus approach velocity ratio on the existing screen.

Use of the baffles when measuring the field velocities could not be duplicated in the model because the baffles were not included in the model, nor were the baffle positions known for each screen bay. Dye tracings in the model showed that the flow conditions at the screen were similar to those in the field. No dye entered the screen at the upstream end with the dye passing through the screen more as it progressively traveled downstream. Sweeping velocities were measured in the model after one of the first modifications to the model that narrowed the width of the channel adjacent to the screen without performing any upstream modifications. The implication of the measured screen sweeping velocities is discussed under Test 3.

Figure 2 shows field velocity measurements on the downstream side of the trashrack bridge. Velocities were assumed to have been taken in the middle of the bridge bays. The velocity measurement of about 2 ft/s was consistent throughout the depth on the left side, but had a great disparity between the bottom and surface readings on the right side. The bottom reading is probably more representative of the actual flow conditions and the 0.58 ft/s velocity was used for comparison to the model results. This would show about a 3.5 to 1 velocity and flow ratio from the left to the right side of the canal and is a good indicator of the flow problems on the screen.

Velocities were measured in the model at the upstream side of the trashrack bridge as shown at locations A and B on figure 2. Velocity measurements from the model were very similar to those recorded in the field showing approximately 4 times greater velocity on the left side of the bridge pier than on the right. Both the bridge velocity measurements and the observation of the dye near the screen produced confidence that the model would provide representative results for the prototype.

Test Sequence - With verification of initial flow conditions, the test program began to solve the flow distribution problems. Initial tests were performed to try to attain uniform velocity across the canal width while minimizing the cost of the modification. The following criteria were used to investigate possible modifications:

- Effective modification at minimal cost.
- Perform modifications that would maintain or reduce the present head loss.
- Try to keep modifications in the canal and out of the river.
- Keep trashrack velocities at 2.5 to 3 ft/s to prevent buildup of trash on the manually cleaned rack.
- Increase sweeping to approach velocity ratio as much as possible.
- Implement modifications that would not create a sediment problem within the canal.

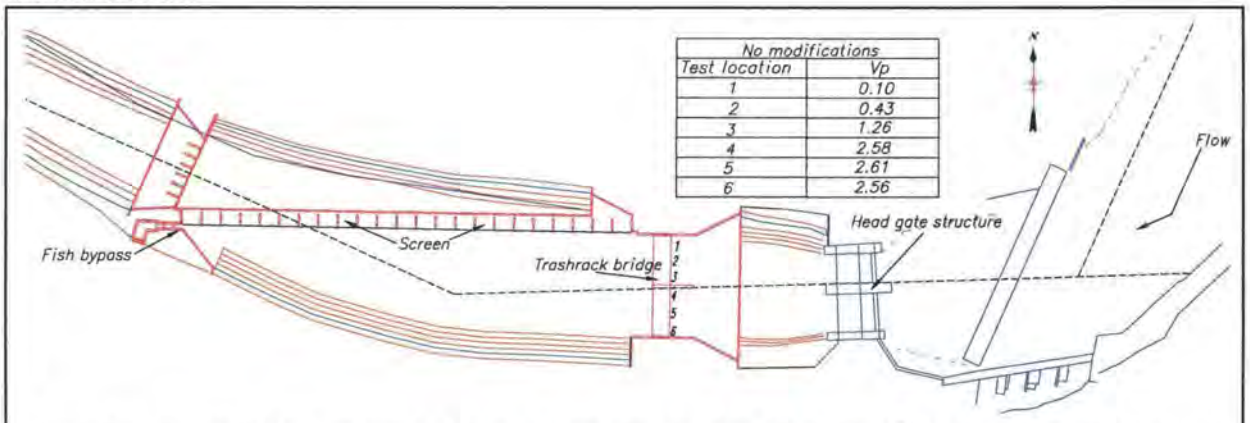
A summary of the tests conducted is given in Table 1. Often a quick test was conducted without documentation other than observation of the flow condition. The test numbers below are used to reference the remainder of the test conditions documented.

Table 1. Summary of test configurations used to improve velocity distribution in the canal and in front of the screen.

Test	Geometry	Observations
1	Original	Velocities heavily skewed from right to left. Reverse flow on right. Velocity on left exceeding trashrack criteria. Flow from gates shoots across the canal to the left side. Recirculation in sloping sides of canal.
2	Gate adjustments	Nothing works. All flow still on left.
3	Downstream screen channel narrowed	Helped but still looks like too much area. Redirection needed upstream.
4	Flow deflectors added in upstream channel	Successfully diverted portions of flow. Hot spots and dead zones still created.
5	Bendway weirs added in downstream screen channel	Velocities not high enough to deflect flow over towards screen face.
6	Center wall inserted between head gate pier and trashrack pier	Flow is divided equally between left & right sets of gates. Velocities not uniformly spread across section.
7	Deflector added to center wall in right bay	Appeared that cross-sectional area of canal still too large to adequately deflect flow.
8	Left wall added to narrow left side and center wall angled to right	Narrowed canal section on both sides of pier to increase velocities. Still not uniform distribution.
9	Center wall removed left wall angled more toward the right making opening at bridge 36.6' wide	Velocities still skewed to left, but overall conditions better and geometry simplified. Dye tracings showed that only upstream portion of center wall was needed.
10	Deflector angled right at 10 degrees added to center head gate pier	Deflects right gate flow to right side wall where the screen is located.
11	Vertical wall added to right side of canal downstream from gates	Prevents eddy formation and head loss reduced. Velocity minimally affected.
12	Straight vertical wall installed from the left head gate pier to the bypass entrance w/center pier deflector and right canal wall - Center deflector geometry and placement optimized	
12a	Center deflector angled at 10 degrees with 21 ft-long deflector and continuous left vertical wall and vertical right wall	Similar to test 11 with the temporary left test wall replaced by a continuous straight wall from the left head gate pier to the fish bypass opening. This alignment produces something easy to construct but a little wider section at the bridge. Velocities somewhat lower, particularly near right wall.

Test	Geometry	Observations
12b	Center deflector angled at 17 degrees with 21 ft-long deflector and continuous left vertical wall and vertical right wall	Steeper angle for the deflector produced slightly higher velocities near the right wall.
12c	Center deflector angled at 13 degrees with 16.25-ft-long deflector and continuous left vertical wall and vertical right wall	Shorter deflector wall at intermediate angle produced lower velocities near the right wall.
12d	Center deflector angled at 22.6 degrees with 16.25-ft-long deflector and continuous left vertical wall and vertical right wall	Shorter deflector wall with a very steep angle seemed to produce some rebound off of the right wall upstream from the bridge and lower velocities near the right wall at the bridge.
13	Center deflector angled at 15.5 degrees with 21-ft-long deflector and continuous left vertical wall and vertical right wall installed	Longer deflector needed to redirect flow from the right gates. Intermediate deflector angle provided best velocity distribution by the right wall. Continuously increasing sweeping velocities along the screen.
14	Sloped wall installed downstream from the bridge deck to transition at the bypass entrance w/center pier and right canal wall and vertical right wall installed	Longer deflector needed to redirect flow from the right gates. Intermediate deflector angle provided best velocity distribution by the right wall. The sloping wall downstream from the bridge through the screen section provides adequate sweeping velocities, but the larger flow area caused sweeping velocity to attain and maintain a maximum value.

Original Geometry (1) - No upstream modifications with the canal flow at 640 ft^3/s produced similar velocities at the trashrack bridge location to the verification tests, figure 3. The average of these velocities is 0.6 ft/s on the right and 2.58 ft/s on the left, still indicating at least a 4 to 1 ratio of skewness. In addition, the velocity was almost zero near the right wall and increased across the channel to the left. Dye tracings showed the same trends as indicated by the velocity measurements.



No modifications	
Test location	Vp
1	0.10
2	0.43
3	1.26
4	2.58
5	2.61
6	2.56

Figure 3. - Current GVIC screen and channel geometry with a flow of 640 ft^3/s in the canal. The fish bypass was not modeled. Velocities were measured at the bridge where the trashrack would be located in the prototype. (nomodsgvi.wmf)

The most telling observation is the alignment of the head gates and canal compared to the screen location. If an imaginary straight line were drawn from the centerline and normal to the head gates, it would actually be turning away from the screen toward the left side of the canal, figure 3. At the head gate entrance to the canal, the flow is directed to the left side with a vortex forming in front of the right set of gates caused by the separation around the center pier and the flow direction. There is about a 90-degree turn into the head gates from the river. The screen is located on the inside of a bend in the canal with flow directed to the outside of the bend away from the screen.

Gate Operations (2) - The flow was heavily skewed to the left side of the canal. Therefore, closure of the left gates and corresponding opening of the right gates was tested to help balance out the flow conditions. The right three gates were fully opened and the left three gates were closed appropriately to maintain the same upstream head as was set with uniform gate operation.

Spot velocities were measured only in the center of the bridge bay openings and produced a velocity on the right of 0.45 ft/s and on the left of 2.28 ft/s. This produced a velocity on the left side of the bridge pier five times that on the left side. This exacerbated the problem putting even more flow on the left than before. This was not intuitively what was expected, but dye tracings showed that flow from the right gates shot across the canal downstream from the center head gate pier to the left side of the canal. Solving the problem of unequal flow distribution through gate operations was not considered further.

Initial Reduction in Downstream Channel Width (3) – The sweeping bend on the left side of the canal opposite the screen was an obvious problem. An initial modification narrowed this section to reduce flow area and hopefully increase sweeping velocity. The modification was to use the same 2:1 slope as the initial wall and follow a straight line from the vertical wall downstream from the trashrack bridge to the fish bypass opening as shown in figure 4.

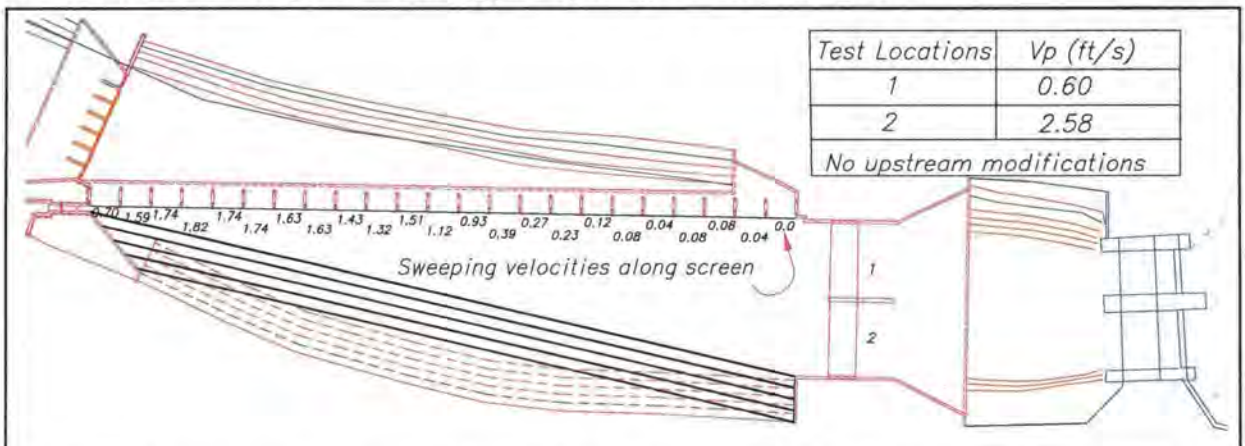


Figure 4. - Downstream channel area adjacent to the screen narrowed as a needed first modification. The canal flow is 640 ft^3/s with upstream El. 4681. Screen sweeping velocities were taken at this time for comparison to the field data and future modifications. (orig screen vels.wmf)

The sweeping velocities recorded in figure 4 were measured with no upstream modifications and the narrowed area opposite the screen. The very small sweeping velocities in front of the first 10

screen bays or about one-half the screen area indicated that the flow was not flowing along this portion of the screen, leaving a major section of the screen area under utilized. These initial measured screen sweeping velocities indicated the same problem that the field measurements had implied. They were also used as a baseline for comparison of the effectiveness of upstream modifications to follow. The poor use of the upstream portion of the screen and the skewed canal velocities indicated that the flow needed to be redirected in the canal upstream from the screen.

Upstream Flow Deflectors (4) - Several different short deflectors that were about one half the height of the flow depth were tried at several locations upstream from the trashrack bridge. Primarily they were placed at various locations downstream from the left or south gates. The initial thought was that these would be inexpensive to place in the field and that they would move enough of the flow to the right side of the canal to be effective. A typical example of deflector placement is shown in figure 5. Figure 5 also shows that the point canal velocities were more uniform than with the initial geometry. The overall velocity through this section was still too low and dye tracings revealed "dead zones" or recirculation zones near both side walls of the canal. The deflectors would likely soon be buried in sediment quickly and could produce a maintenance problem. More flow had to be redirected than could be managed by a small deflector.

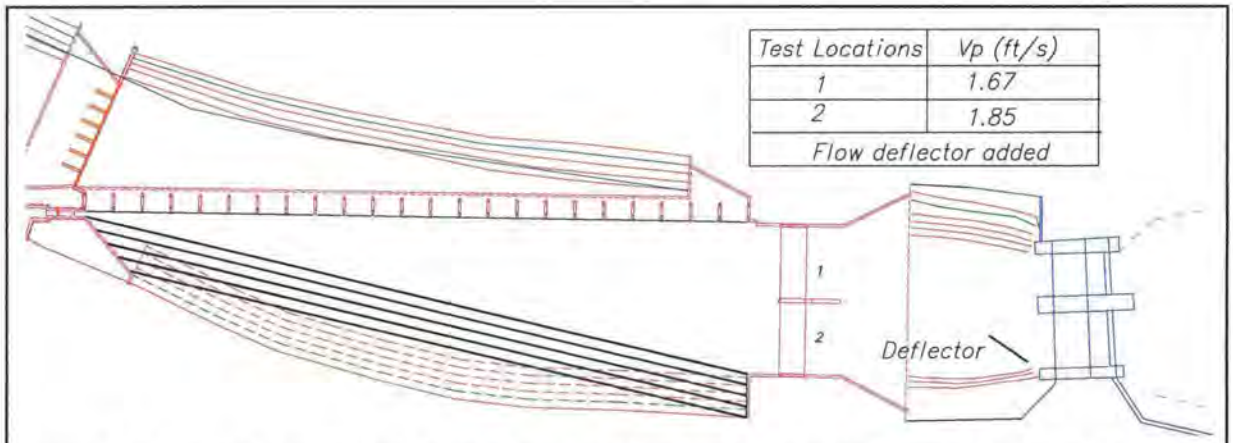


Figure 5. - Example of an attempt made to use a relatively small deflector placed in the canal to redirect flow from the south to the north side of the canal by the screen. (lftgatdef.wmf)

Rock Weirs in the Downstream Channel (5) – Another option investigated was to place rock weirs in the channel opposite the screen. Weirs of this type have been used in bends of natural channels to prevent incision of the banks by redirecting the flow and causing deposition of sediment on the backside of each weir. Both of these features seemed attractive, as the flow needs to be redirected towards the screen and the area reduced. Several rock weirs, also referred to as bendway weirs, were placed in the model on the south wall opposite the screen.

The result of the rock weir investigation, however, did not meet expectations. The velocity was too low to significantly redirect flow towards the screen. Deposition of fine sediment would occur, but in the interim, there would be no direct benefit from the weirs. Therefore, this alternative was not pursued further.

Narrowed downstream channel and center wall divider (6) - Previous tests indicated that reduced area in the screen channel was necessary, but was not enough to significantly improve screen performance. The 2:1 sloping wall on a straight line from the downstream end of the trashrack bridge to the fish bypass entrance, test 3, remained in the model while various upstream canal modifications were investigated.

Test 6 involved temporarily installing a center divider wall from the center pier of the head gate structure to the center pier of the trashrack bridge, figure 6, with the narrowed downstream channel. The purpose of the divider wall was to investigate if the gates were passing a similar amount of flow or if significant modifications were potentially needed in the river upstream from the gates to redistribute the flow into the gates.

Velocities were measured at the trashrack bridge, figure 6, and along the screen with a propeller meter set at 0.6 depth. Figure 7 shows that the velocities measured in the center of the bridge bays were each about 1.8 ft/s. Injecting dye in each section still revealed eddy zones and recirculation near both canal walls. Dye was used upstream of the gates to look at the flow patterns upstream of the head gates in the river. Dye placed on the right side upstream from the head gates appeared to mostly be drawn through the right gates. Based upon both the velocity and dye results, it was concluded that the flow quantities through each set of gates was similar and that flow was concentrating on the left or south side after entering the canal. Therefore, no attempts were made to make modifications in the river upstream from the gates.

Figure 6. - Center pier wall installed between the existing head gate pier and the trashrack pier. This modification split the flow from each set of gates allowing independent determination of flow amounts through each set of gates.

With the center divider keeping about one-half the discharge flowing through each side of the canal, the screen sweeping velocities should have improved. Figures 4 and 7 show the screen sweeping velocities with the divided flow have improved compared to the initial canal geometry. Figure 8 shows the sweeping velocities graphically for comparison. Prior to separating the flow with the center divider, sweeping velocities showed poor use of the screen until at least bay 11. With the center divider, sweeping velocities were increased, but still showed that the first 6 bays are not well utilized. Dye tracings confirmed that velocities near the north or right canal wall were still very low upstream from the screen.

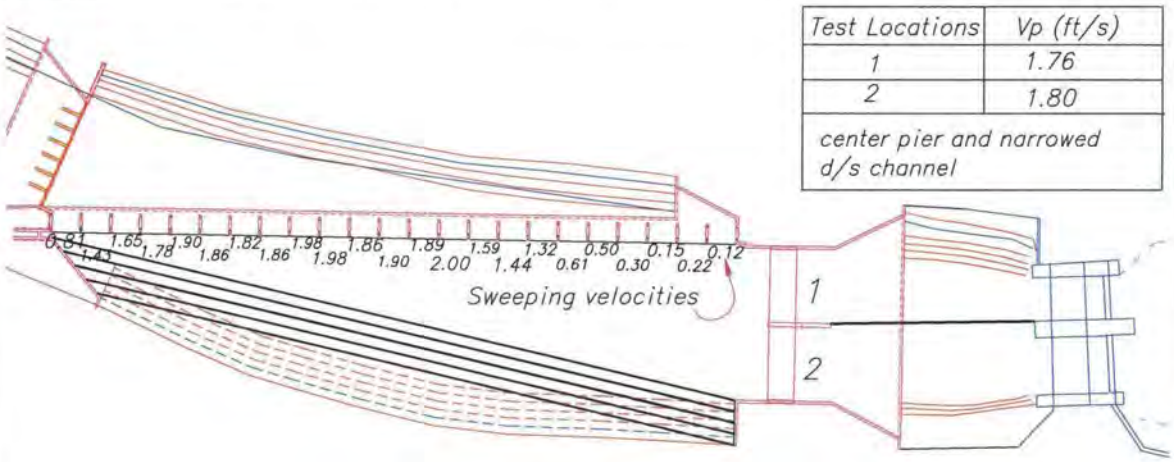


Figure 7. - Schematic of center pier divider wall with bridge and sweeping velocities in initially narrowed downstream channel. (narrow&center.wmf)

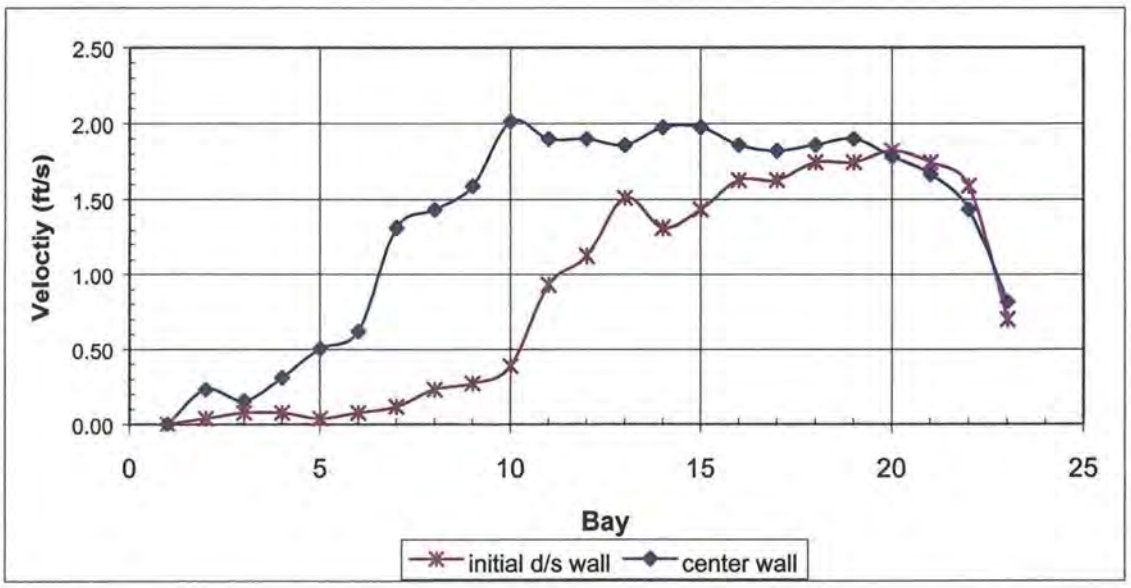


Figure 8. - Screen sweeping velocities measured with a propeller meter in the center of the screen bays. Note that the velocities are very low in front of the first several bays for both conditions. With no redirection of the flow upstream, the sweeping velocity increases about bay 11. With the gate flow divided the sweeping velocity increases about bay 7. Note: Bays are numbered beginning with 1 at the upstream end of the screen.

Center Wall Divider with Deflector Wall Attached (7) – The previous test indicated that the velocity near the right wall upstream from the screen was still very low or there was an eddy zone near the wall. In this modification, a short deflector angled toward the right wall was added to the center wall, figure 9. The result of this test was that the deflector helped but the width or area under the bridge is too large to eliminate eddies and increase the velocity enough to produce adequate sweeping velocities over the upstream end of the screen.

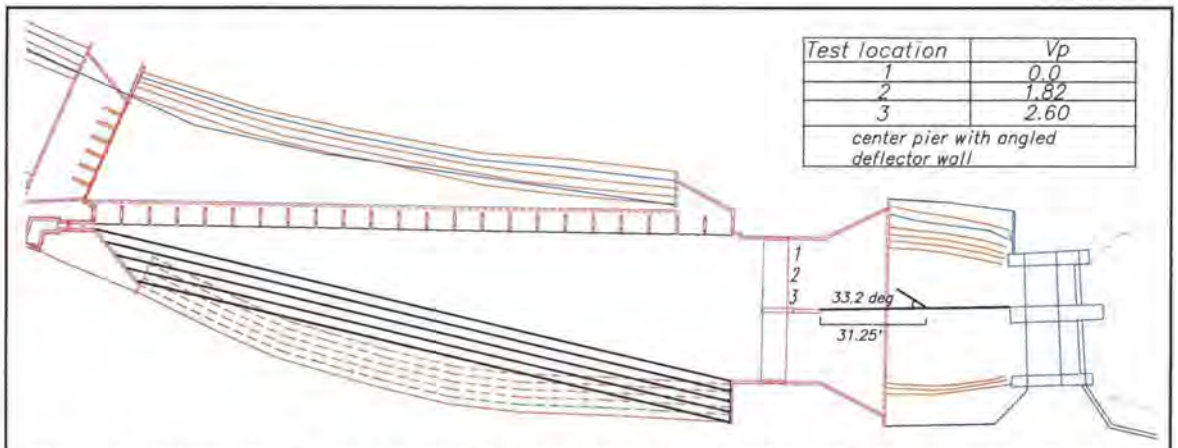


Figure 9 - Schematic of center pier modification with a short angled deflector located on the center wall to direct more flow to the right. (centerwdef.wmf)

Two Angled Walls (8) – The next approach was to reduce the area or width under the trashrack bridge to increase the average velocity and reduce eddies. The width needed at the bridge to maintain 2.5 ft/s velocity through the trashrack was determined using the continuity equation:

$$Q = VA$$

or

$$Q = Vwd \xrightarrow{rearranging} w = Q/Vd$$

Where Q = canal discharge (ft^3/s)
V = flow velocity or 2.5 ft/s
A = area (ft^2) = width, w (ft), times depth, d (ft)

A width of 36.6 ft is needed at the bridge section for a discharge of 640 ft^3/s and a depth of 7 ft in the canal with a design trashrack velocity of 2.5 ft/s. The objective became to produce a uniform velocity distribution as close to 2.5 ft/s as possible by decreasing the area under the bridge.

The initial attempt at producing this width was to install a wall on the left or south side of the canal from the head gate pier to the bridge while also angling the center wall, figures 10 and 11. This made approximately equal widths at the bridge for both bridge bays. Figure 11 shows that overall the velocities are higher, but there is still a skewness of the flow to the left within both bays. Sweeping velocities measured at a few locations on the screen showed a great improvement over the initial conditions and previous modification attempts.

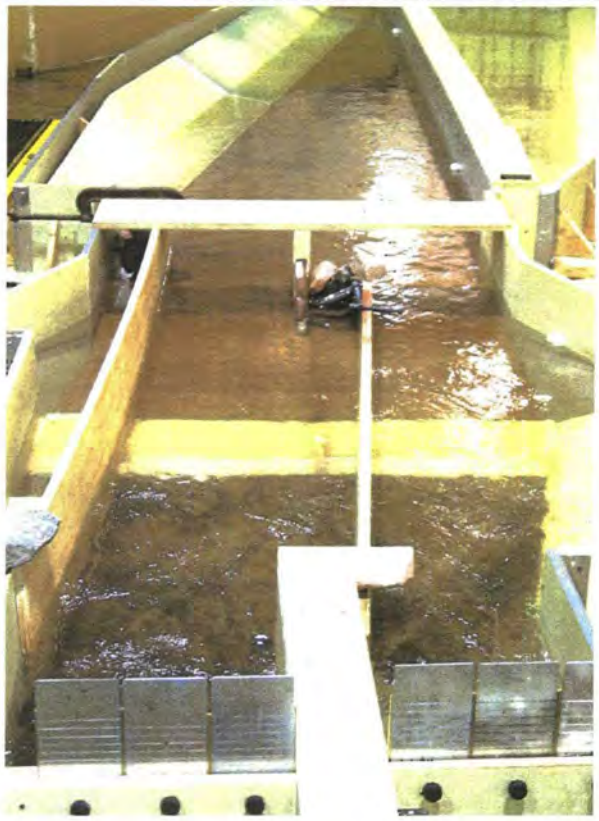
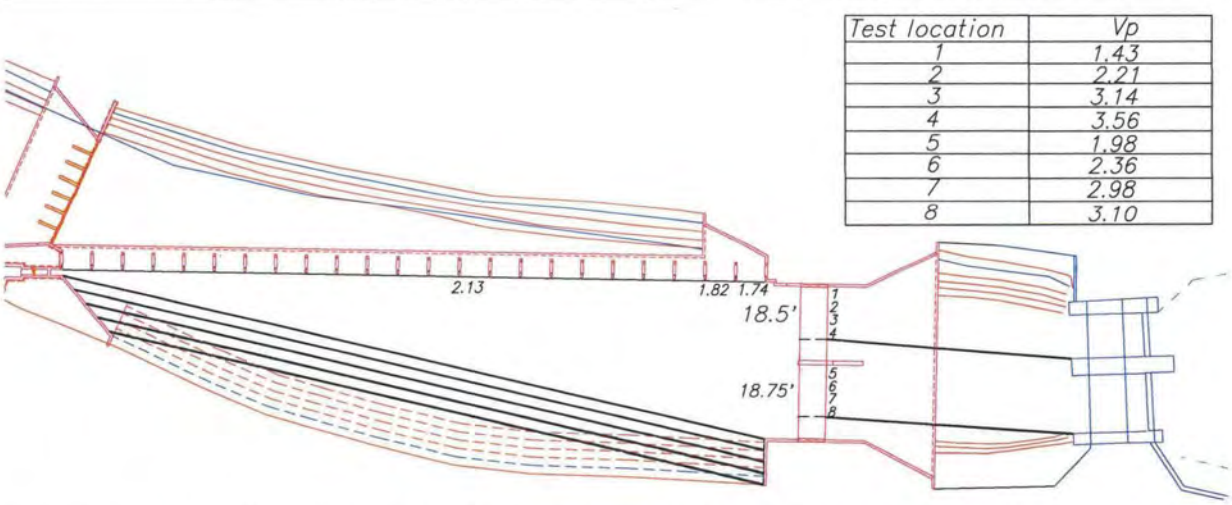


Figure 10. - Installation of two angled walls from the head gate structure to the trashrack bridge reducing the flow area at the bridge.



Test location	Vp
1	1.43
2	2.21
3	3.14
4	3.56
5	1.98
6	2.36
7	2.98
8	3.10

Figure 11. - Two angled walls from head gate structure to bridge reducing flow area. (2slopedwalls.wmf)

Dye tracings show that the flow is staying near the right wall and in front of the screen when injected in the right bridge bay, figure 12. Dye injected in the left bay heads downstream and spreads some over the sloped wall area, but is more confined than previously observed, figure 13.

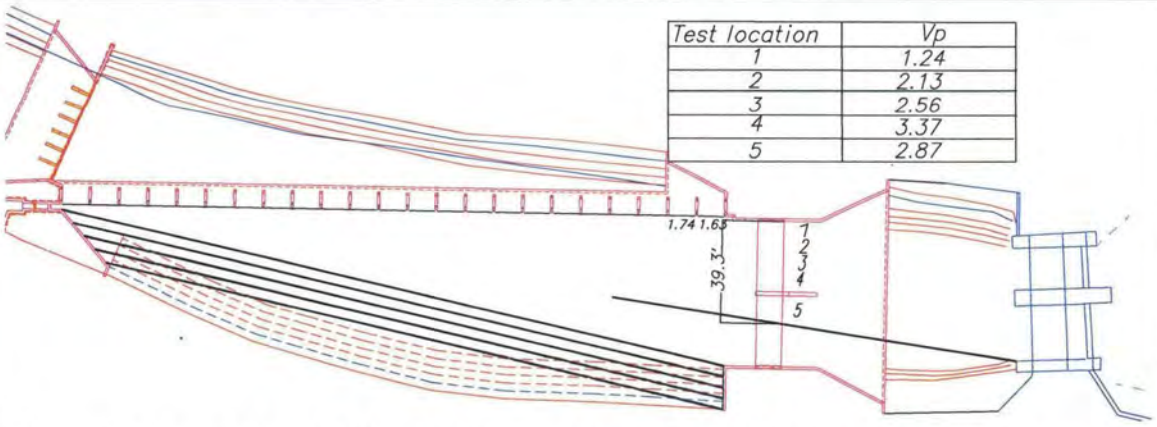
Figure 12. - Flow conditions with the two angled walls shown with dye injected in the right bridge bay. The flow is now heading downstream near the right wall and in front of the screen.

Figure 13. - Flow conditions with the two angled walls with dye injected in the left bridge bay. Flow heads downstream but spreads out over the sloped wall.

Left Angled Wall with Center Wall Removed (9) – Figures 14 and 15 show this modification that included removing the center wall and angling the left wall more toward the right narrowing the total width at the bridge to about 37 ft. The objective was to determine the influence of the center wall versus the narrower total canal width and redirection provided by the left wall alone. The width of the right bay under the bridge was the original width with the left wall narrowing the left bridge bay. Velocities showed that more flow was getting over to the right side of the canal as the velocities were generally increased across the section. However, the majority of the flow and higher velocities were still on the left side of the canal, figure 15.

Narrowing of the area under the bridge produced much more uniform flow conditions, but still more flow needed to be directed to the right side. Dye was injected downstream from the center head gate pier to investigate the extent of the area of the right gate flow that travels across the canal to the left wall.

Figure 14. - Left wall angled from the head gate structure through the bridge and into the downstream channel. The center divider wall was removed. Total canal width at the bridge is about 37 ft.



Test location	Vp
1	1.24
2	2.13
3	2.56
4	3.37
5	2.87

Figure 15. - Single wall on the left side of the canal to reduce the total width of the canal section at the trashrack bridge. (single us wall.wmf)

The left wall was extended downstream from the bridge section into the area opposite the screen to help reduce this area also. This seemed like a beneficial modification, as this area remained too large.

Left Wall with Head Gate Center Pier Deflector (10) – The dye tracings from the previous test showed that the center wall only needs to be long enough to deflect the flow from the right set of gates back to the right side. The initial test was with a 21-ft-long deflector placed at about a 10-degree angle to the right. The right, sloped area of the canal section upstream from the bridge was not modified. Figures 16 and 17 show the geometry of this temporary modification. The

Figure 16. - Angled left wall with 21-ft-long center pier deflector angled 10 degrees to the right. Q= 640 ft^3/s.

photograph shows that the deflector wall angle is pointing in the direction of right wall at the bridge. It also shows the large flow area over the sloped wall between the gates and the bridge that allows formation of eddies and reverse currents. Figure 17 shows the velocities measured at the bridge location are quite uniform and the sweeping velocity in front of the first two screen bays are much improved over test 9 without the center pier deflector.

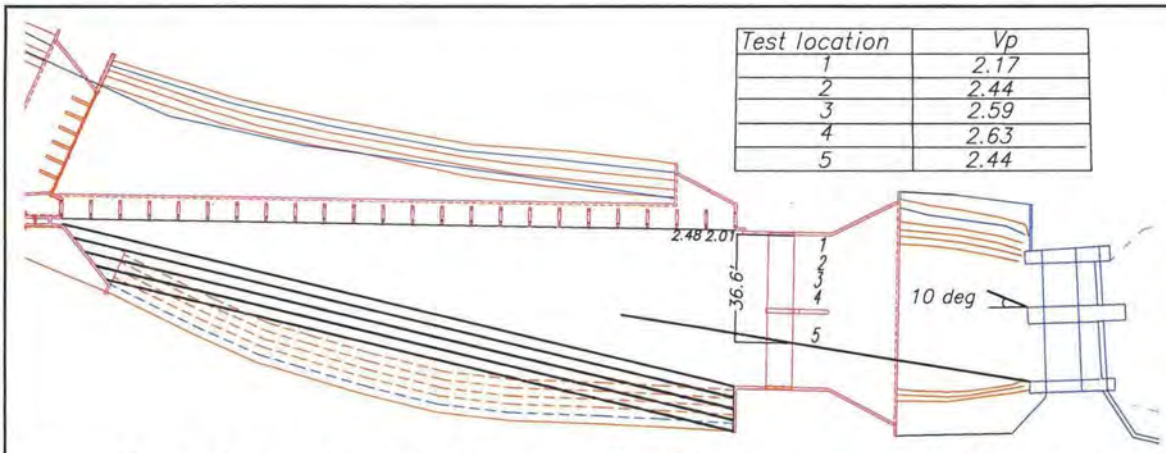
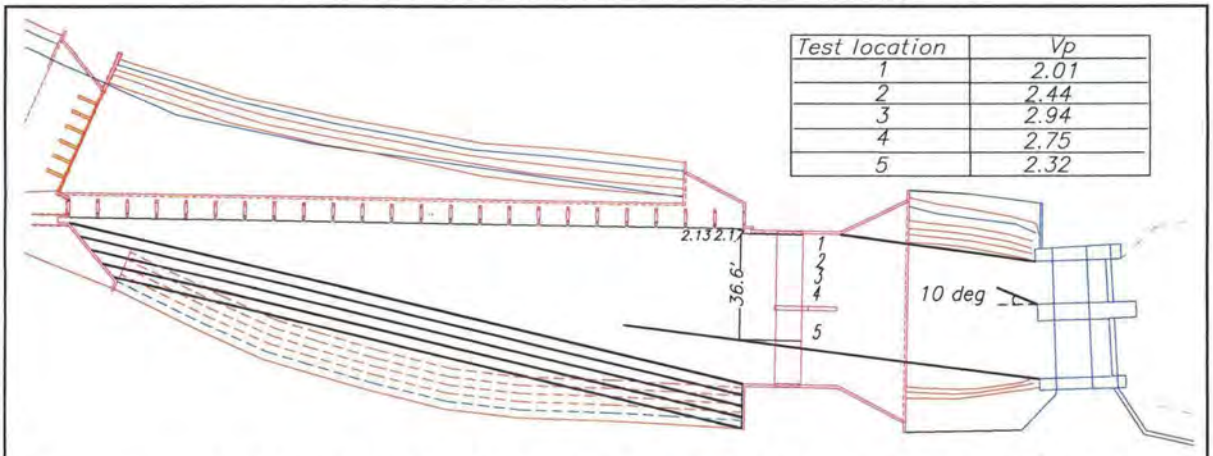


Figure 17. - Angled left wall and 21-ft-long center pier deflector to narrow canal width and redirect flow to the right side. Q=640 ft^3/s. (short left wall&def.wmf)

Left Angled Wall with Center Pier Deflector and Right Wall (11) – This is the same configuration as test number 10 with a vertical wall added on the right side of the canal upstream from the bridge. Figures 18 and 19 show the 21-ft-long defector installed at a 10-degree angle to the right with the vertical wall on the right to prevent eddies in the area upstream from the trashrack bridge. Comparing flow conditions in figures 16 and 18 shows that the flow is less turbulent with the right wall installed. This will improve flow conditions into the screen and reduce eddy zones preventing sediment deposition and reducing head loss. The velocities at the bridge section are not substantially different, but might be slightly shifted to the left. The screen sweeping velocities are slightly different than in test 10 but still greatly improved.

Figure 18. - Angled left wall, center pier deflector and right vertical wall covering sloped canal wall. Q=640 ft^3/s.



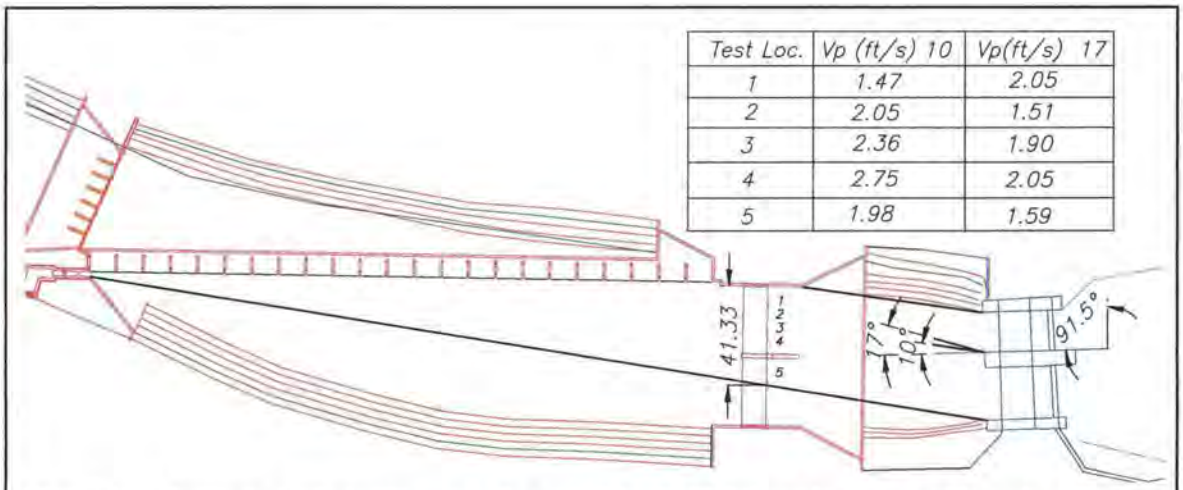
Test location	Vp
1	2.01
2	2.44
3	2.94
4	2.75
5	2.32

Figure 19. - Angled left wall with 21-ft-long center pier deflector and vertical right wall added. Q= 640 ft^3/s. (short left wall&def&rt.wmf)

These tests confirmed that this approach to the problem was correct. The left wall was nearly on a straight line between the left head gate pier and the fish bypass entrance so it was replaced with a vertical wall connecting those structures. More optimization was performed on the center deflector placement angle and length in the following tests.

Optimization of Center Pier Deflector Length and Angle (12) - The previous tests showed that the canal area reduction and deflector on the center pier were necessary to improve the flow conditions near the screen. All the following center pier deflector tests were performed with a vertical wall installed on a straight line from the end of the left head gate pier to the fish bypass entrance. The wall alignment produced a slightly wider width at the bridge of 41.25 ft. The slightly wider section will somewhat reduce the average velocity in the section, but the amount of flow near the right side of the canal could be increased using the center pier deflector.

This section discusses several of the different deflector angles and lengths used to optimize the flow conditions for a canal discharge of 640 ft^3/s and upstream river water surface elevation of 4681. All remaining tests were performed with the right wall installed in front of the sloped wall upstream from the bridge. Figure 20 shows options 12a and 12b with a 21-ft-long deflector wall at 10 and 17 degree angles with the newly installed straight vertical wall from the left pier to the fish bypass opening.

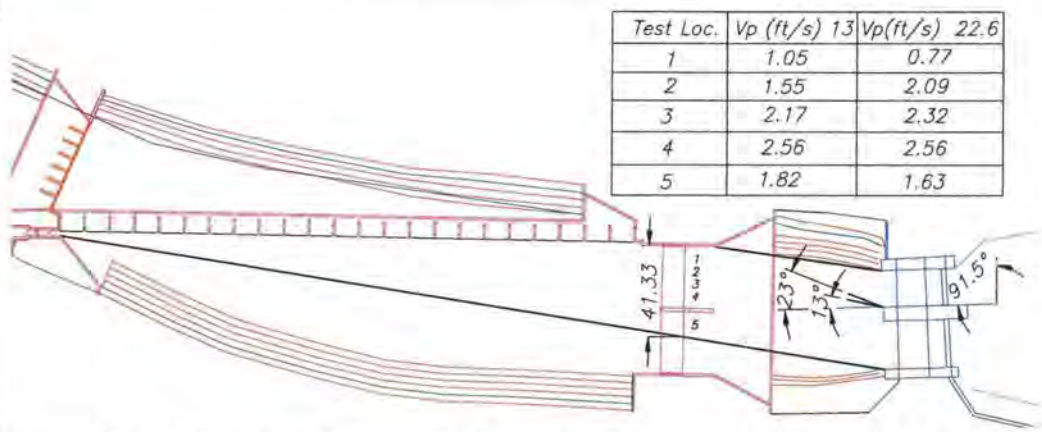


Test Loc.	Vp (ft/s) 10	Vp(ft/s) 17
1	1.47	2.05
2	2.05	1.51
3	2.36	1.90
4	2.75	2.05
5	1.98	1.59

Figure 20. - Deflector optimization tests 12a and 12b with 21-ft-long deflectors with angles of 10 and 17 degrees with the straight vertical wall in the channel. Q = 640 ft^3/s. (vert wall tests A&B.wmf)

The velocities measured at the bridge location with the 10-degree deflector show a slight overall decrease when compared to test 11 and the narrower section shown on figure 14. This is only a function of continuity and the slightly larger flow area with the newly installed straight wall. The average velocity for the 10-degree deflector was 2.12 ft/s. The critical velocity near the right wall was 1.47 ft/s with the 10-degree deflector wall angle. The 17-degree wall angle was then investigated to see if a greater deflection angle would redirect more flow to the right. Velocities were improved directly next to the wall and compared well to the velocity of the temporary wall in test 11. This option seemed slightly better than the 10-degree deflector angle.

The next deflector options, tests 12c and 12d, were to shorten the deflector wall to 16.25 ft and test various angles as shown in figure 21. Angles of 13 and 22.6 degrees were tested with the shorter length deflector. Figure 21 shows the velocities near the right wall were significantly less compared to the previous deflector length and angles. The longer 21 ft deflector was then reinstalled and the angle optimized in the remaining tests.



Test Loc.	Vp (ft/s) 13	Vp(ft/s) 22.6
1	1.05	0.77
2	1.55	2.09
3	2.17	2.32
4	2.56	2.56
5	1.82	1.63

Figure 21. - Optimization of center pier deflector with shorter 16.25-ft-long deflectors on steeper angles with the straight vertical wall in the channel. Q = 640 ft^3/s. (vert wall tests C&D.wmf)

Straight Vertical Wall Through Canal with Final Center Pier Deflector Geometry and Right Wall (13) – A few more deflector angles with the 21-ft-long deflector were tested until the final angle of 15.5 degrees was determined to be optimal. Figures 22, 23, 24 and 25 show the optimal deflector angle and the flow conditions and velocities near the right wall upstream from the bridge and along the screen. The deflector directed the flow as much as possible toward the right, increasing velocities and producing better use of the first few screen bays. Figure 23 shows dye tracings injected in the left bay downstream from the gates and upstream from the bridge on the right side. Notice that the dye injected downstream from the left set of gates is mixed by the turbulent flow then travels downstream, even spreading somewhat over to the right of the trashrack bridge pier. Figure 24 shows dye injected near the right wall upstream from the bridge stays near the wall and travels downstream in front of the screen.

Velocities in figure 25 were measured with the *Flow Tracker*, an acoustic velocity meter that measures in two dimensions. The acoustic meter recorded the total velocity magnitude at the bridge section and both the sweeping and approach velocities along the screen. The velocity of 2.3 ft/s near the screen is about the highest recorded velocity near the wall for any of the tests. The vertical wall in the channel opposite the screen clearly reduces the flow area and increases the sweeping velocities compared to the original canal area at the screen location. Standing waves formed in channel as it narrowed toward the bypass. These were accentuated without the bypass modeled to allow flow to leave at the end of the screen. The minimum screen sweeping velocity was 2.2 ft/s and increased downstream as the adjacent channel width decreased. Higher sweeping velocities mean better debris handling by the screen.

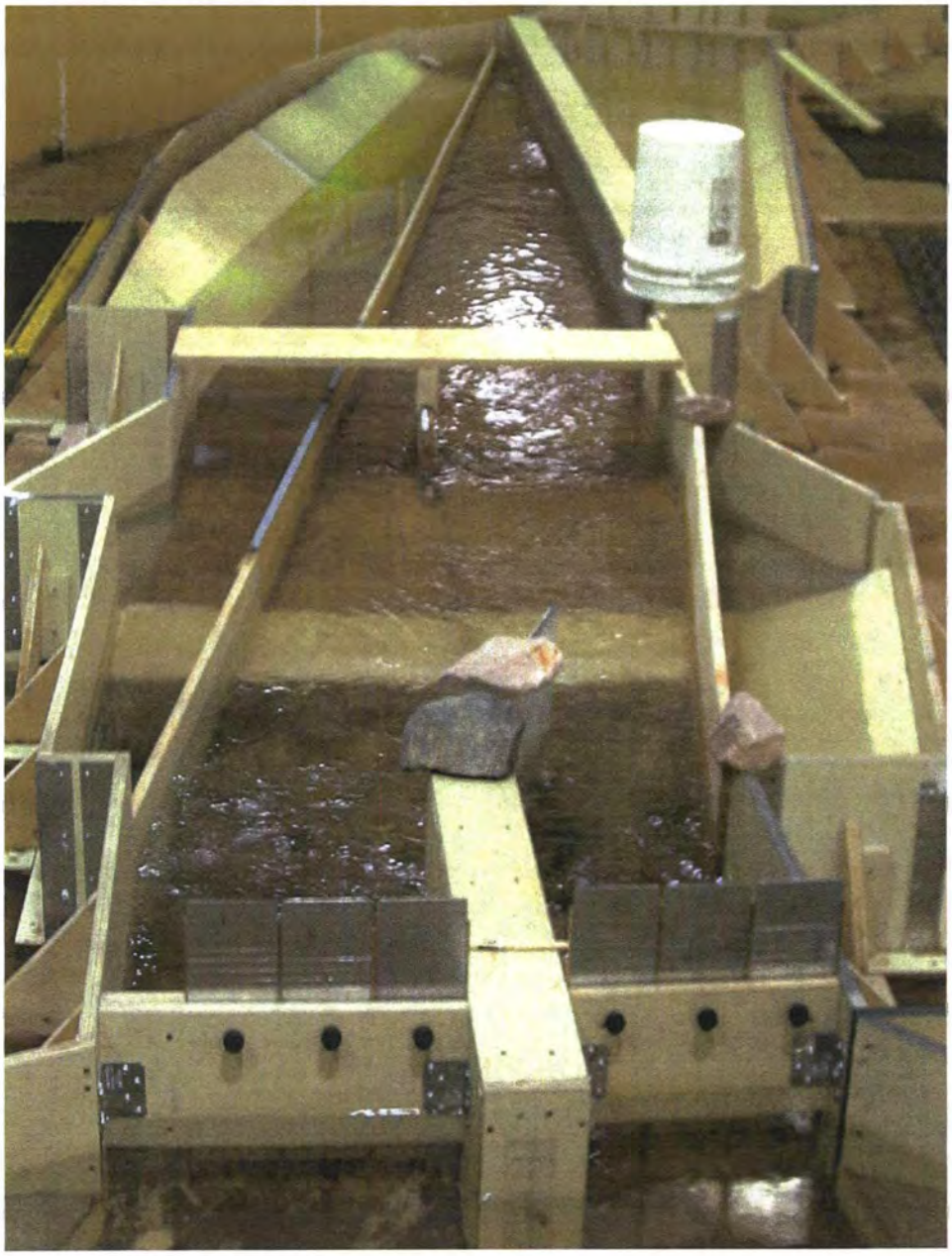


Figure 22. - Test 13, for the GVIC screen modifications with a straight vertical left wall, 15.5-degree, 21-ft-long deflector and right wall for a discharge of 640 ft^3/s.

Figure 23. - Test 13, close up view of dye tracing from downstream of the left (south) gates for a canal discharge of 640 ft^3/s for the 15.5-degree, 21-ft-long deflector with the straight angled left wall with the right wall installed. Note that a portion of the flow now spreads towards the right side of the channel.

Figure 24. - Test 13, close up view of the right wall of the screen with the vertical straight wall, the 15.5-degree, 21-ft-long angled center pier deflector and right wall upstream. Q = 640 ft^3/s.

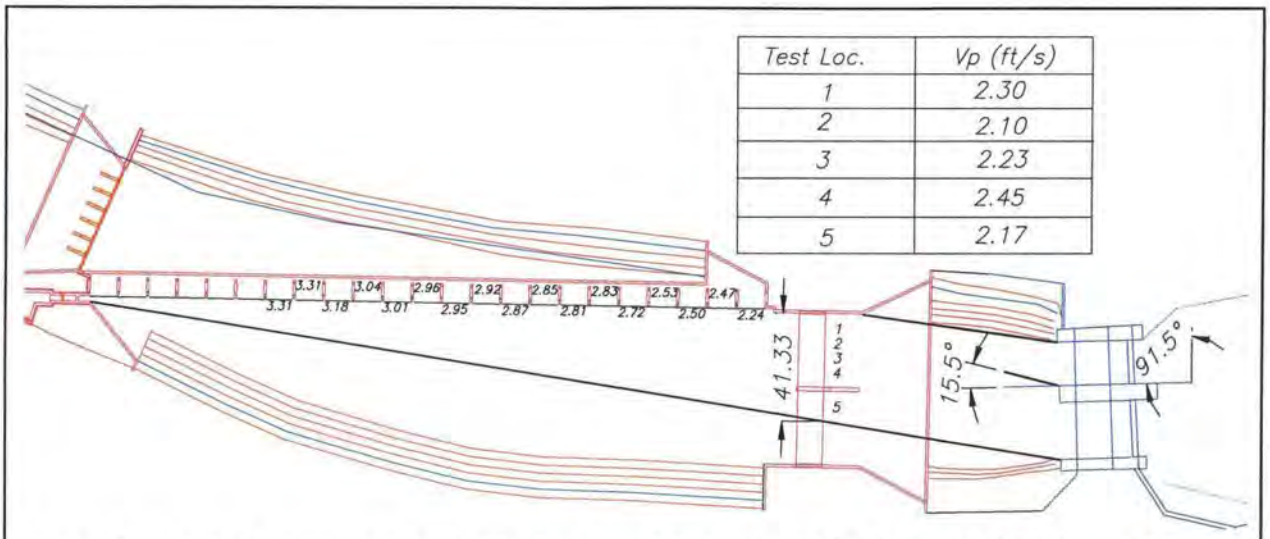


Figure 25. - Test 13, geometry and velocities for a canal discharge of 640 ft^3/s for the 15.5-degree, 21-ft-long deflector with the straight vertical left wall with the right wall installed. Note that canal and screen sweeping velociteis are increased. (vert wall tracker.wmf)

Additional tests were conducted to ensure acceptable performance of the deflector geometry with an increase in canal flow rate and upstream water level. Tests were conducted with a canal discharge of 680 ft^3/s with the river water surface at El. 4686 representing 5 ft over the diversion dam and the canal water surface downstream at El. 4679.6. Figure 26 shows the velocities that were measured at the bridge and screen locations for comparison to the original test discharge of 640 ft^3/s and the minimum river water surface elevation 4681, figure 25. Velocity measurements along the screen could not be continued downstream because of the limited area to place the instrument. Screen sweeping velocities were similar to those at 640 ft^3/s except in front of the first bay where the offset away from the screen is probably having a greater adverse influence with the higher driving head. The screen sweeping velocities still increase downstream

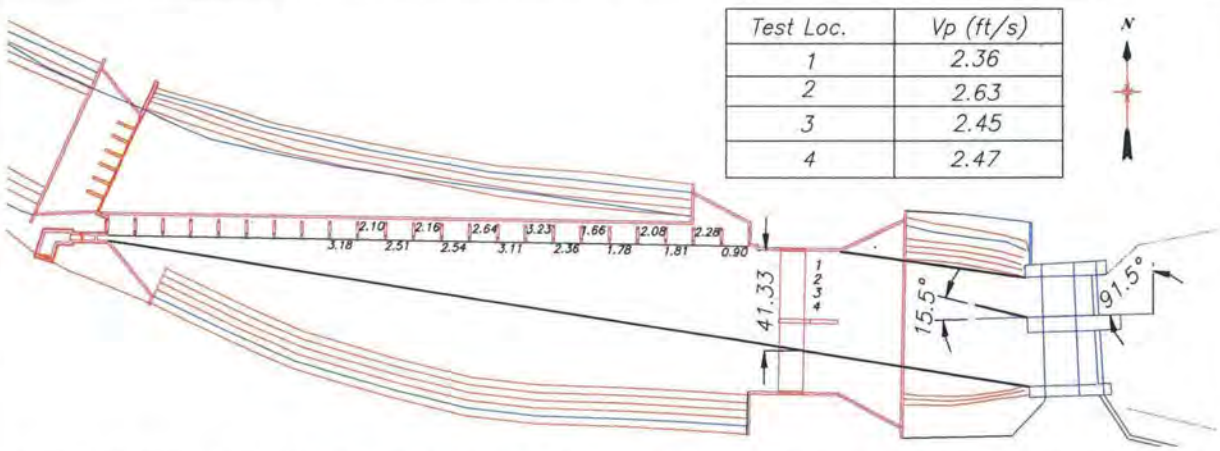


Figure 26. - Velocities measured with the straight vertical wall for a discharge of 680 ft^3/s, upstream river water surface El. 4686, and downstream canal El. 4679. Note the velocities at the bridge location are slightly higher than the canal velocities with the river water surface at el. 4681 and a discharge of 640 ft^3/s. (vrtQ78.wmf)

toward the bypass. The vertical wall clearly confines the flow with some waves developing. Flow conditions were very similar to the previous flow rate and would be considered greatly improved over the existing condition.

The velocities measured at the bridge location were, in general, slightly higher with a higher head differential and discharge, as would be expected. Figure 27 shows slightly more turbulence downstream from the gates, but this turbulence did not seem to extend down to the bridge location. Dye injected stays along the right wall, in figure 28, and is drawn into the first screen bay.

Figure 27. - Overall view of the canal operating under a discharge of 680 ft3/s and river El. 4686 with the final deflector geometry and the straight vertical wall on the left.

Figure 28. - View of the canal operating under a discharge of 680 ft3/s and river El. 4686 with the final deflector geometry and the straight vertical wall on the left.

Straight Sloped Wall Through Canal with Final Center Pier Deflector Geometry and Right Wall (14) – This test was performed to look at the influence of installing a 2:1 sloped wall instead of a vertical wall from the trashrack bridge to the fish bypass entrance opposite the screen structure. It was felt by the client that this geometry would be less expensive to construct. The upstream geometry and velocities remain unchanged from the previous test, but the area of the canal will be larger opposite the screen. Figures 29, 30, and 31 show the geometry, flow conditions and velocities associated with the sloping wall placed at the location of the toe of the vertical wall in the previous tests.

Flow conditions upstream of the bridge remain identical. Figure 29 shows the larger flow area and slightly less wavy flow surface along the screen compared to the vertical wall for the 640 ft^3/s canal discharge. Figure 30 shows that the dye travels almost directly downstream but does spread laterally somewhat.

Figure 29. - Overall view of the canal operating under a discharge of 640 ft^3/s and river El. 4681 with the final deflector and the toe of the 2:1 sloping wall opposite the screen located on the same line as the vertical wall.

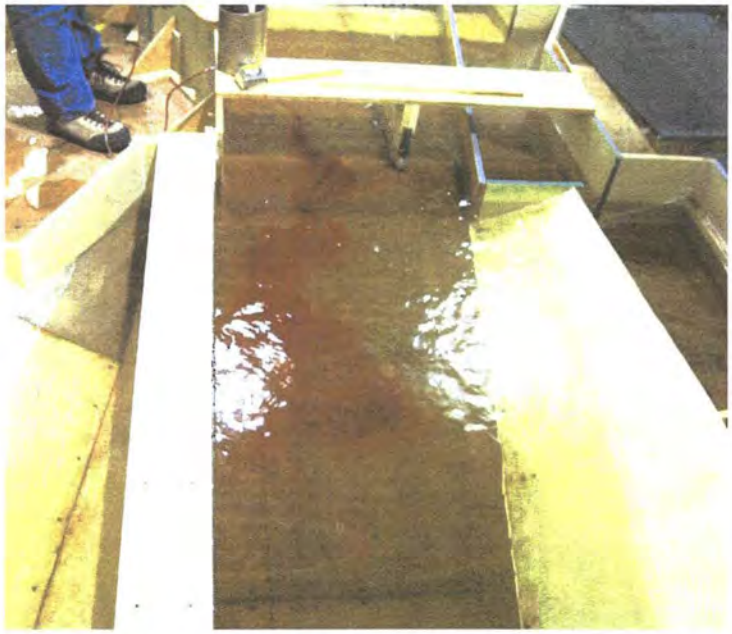
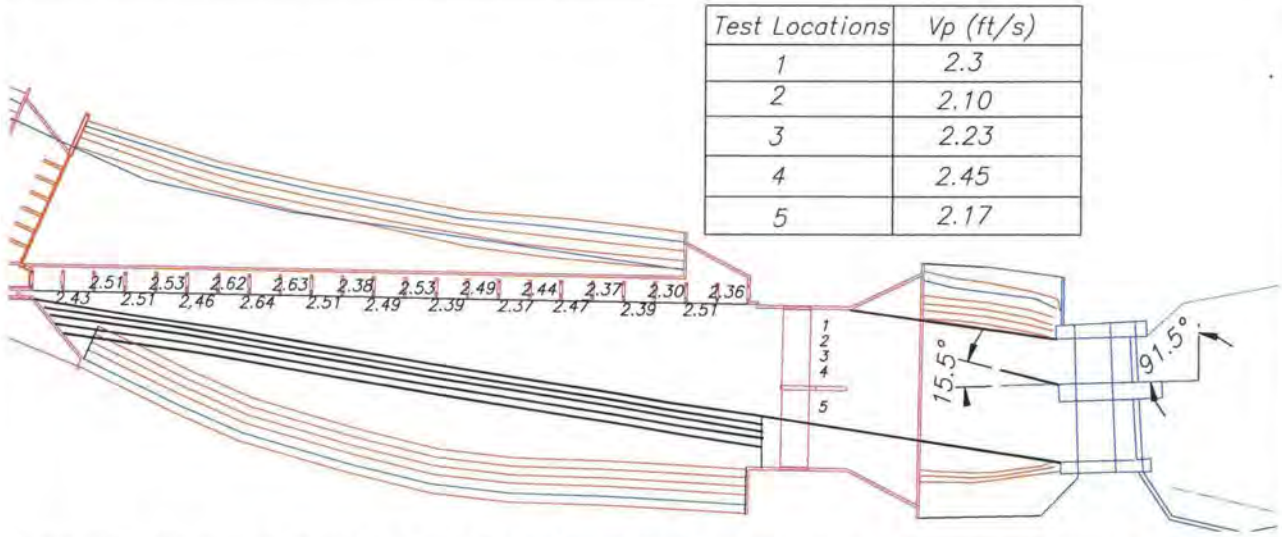


Figure 30. - Close up of dye tracing near the screen with the 2:1 sloping wall in the downstream screen section for a discharge of 640 ft^3/s and river El. 4681.

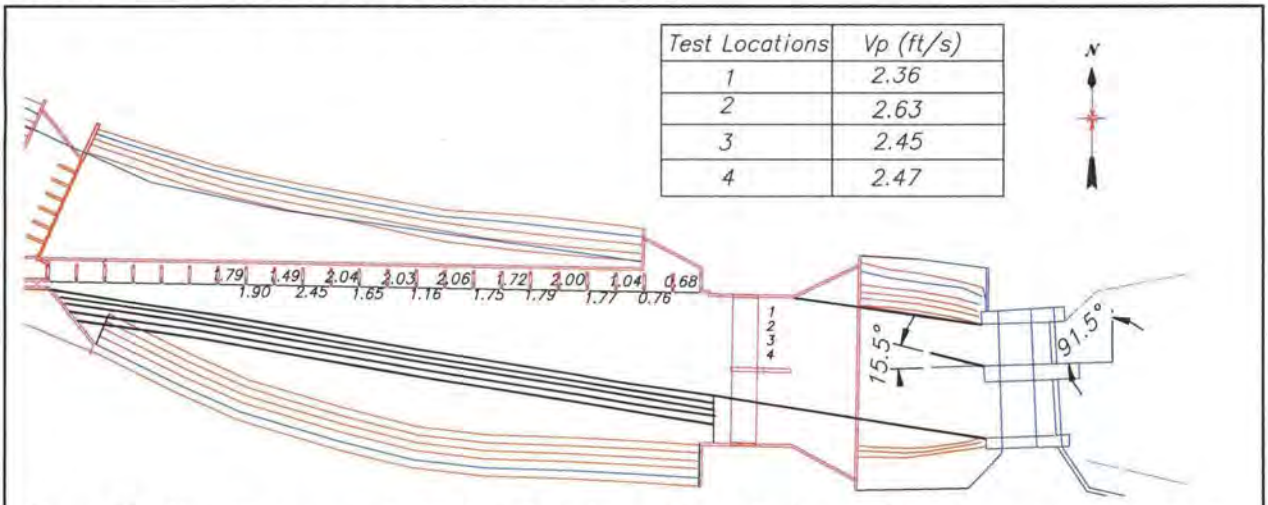
Figure 31 shows that the slightly larger flow area of the 2:1 sloping wall reduces the velocity along the screen. Sweeping velocities, measured with the propeller meter, generally exceed about 2 ft/s, but do not continue to increase significantly with distance downstream along the screen. These velocities should be adequate to improve debris handling.



Test Locations	Vp (ft/s)
1	2.3
2	2.10
3	2.23
4	2.45
5	2.17

Figure 31. - Sloping wall installed along the preferred line of area reduction adjacent to the screen with a discharge of 640 ft^3/s. (sloped wall prop.wmf)

Additional tests were conducted to ensure acceptable performance of the deflector geometry with an increase in canal flow rate and the sloping wall opposite the screen. Tests were conducted with a canal discharge of 680 ft^3/s with the river water surface at El. 4686 representing 5 ft over the diversion dam and the canal water surface downstream at El. 4679.6. Figure 32 shows the velocities that were measured at the bridge and screen locations for comparison to the previous test discharge. Upstream velocities are identical to those shown in figure 26 for the same discharge. The screen sweeping velocities are slightly lower than measured with the smaller discharge, as was the case with the previous test, but are about 2 ft/s along the screen. These velocities should be adequate for improving cleaning.



Test Locations	Vp (ft/s)
1	2.36
2	2.63
3	2.45
4	2.47

Figure 32. - Velocities measured with the 2:1 sloping wall for a discharge of 680 ft^3/s, upstream river water surface El. 4686, and downstream canal El. 4679. Note the velocities at the bridge location are slightly higher than the canal velocities with the river water surface at el. 4681 and a discharge of 640 ft^3/s. (sloped wall head 5.wmf)

Figures 33 and 34 show the deflector wall was still effective in redirecting the flow even with considerably higher upstream head than previously tested. Dye injected near the right side of the canal stays near the screen.

Figure 33 - Overall view of the final deflector geometry and the sloping wall downstream opposite the screen with a discharge of 680 ft^3/s and river El. 4686.

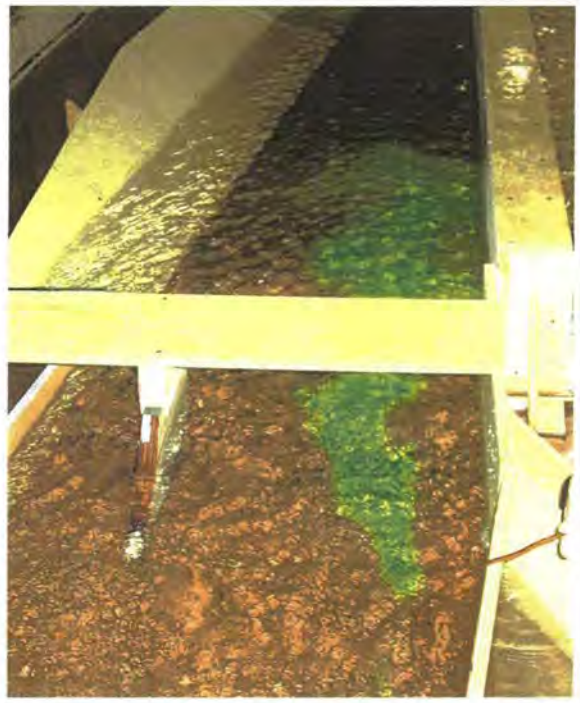


Figure 34 - Dye tracing showing the bridge and upstream screen section with a canal discharge of 680 ft^3/s and river El. 4686.

Summary of Screen Sweeping and Approach Velocities

Sweeping and approach velocities were measured in the field and in the model for the various geometries investigated. The field sweeping and approach velocities were measured for a discharge of 696 ft^3/s with the screen baffling utilized. The model screen velocities were taken with either a propeller meter that could only measure the sweeping component or an acoustic meter that could measure both sweeping and approach velocity components. Figure 35 shows field and model velocities measured for a discharge of 696 ft^3/s before modifications, and the test discharge of 640 ft^3/s for the final modifications. Figure 36 shows the screen velocity measurements for the test discharge of 680 ft^3/s for the final modifications. The two different canal discharges and head differentials were tested to ensure that the proposed modifications would be effective over the possible range of operation.

Current opinion is to increase sweeping to approach velocity ratios as much as possible to optimize debris handling. Debris is a big issue for the GVIC screen, so high sweeping and low approach velocities should be the goal. The design value for the sweeping velocity was 2 ft/s with an approach velocity design value of 0.5 ft/s. These two components would produce a

sweeping to approach velocity ratio of 4. This has been shown to be unattainable with the existing screen structure and canal geometry.

Figure 35 shows the velocities measured in the field and model before modifications for a discharge of 696 ft^3/s. The field sweeping and approach velocities were measured with the downstream end of the screen fully baffled to force flow through the upstream screen bays. The model had no baffles installed so there is no way to make direct comparisons, but trends can be evaluated. The field sweeping velocities were generally between 1 to 1.5 ft/s with approach velocities showing reverse flow through the most upstream bay to exceeding the criteria further down the screen. At best the sweeping to approach velocity ratio would be about 1.5/0.5 = 3:1 for the field measurements. This ratio is misleading though because the general flow conditions upstream from the screen are so poor that the sweeping velocities are low and the approach velocities are often above the criteria.

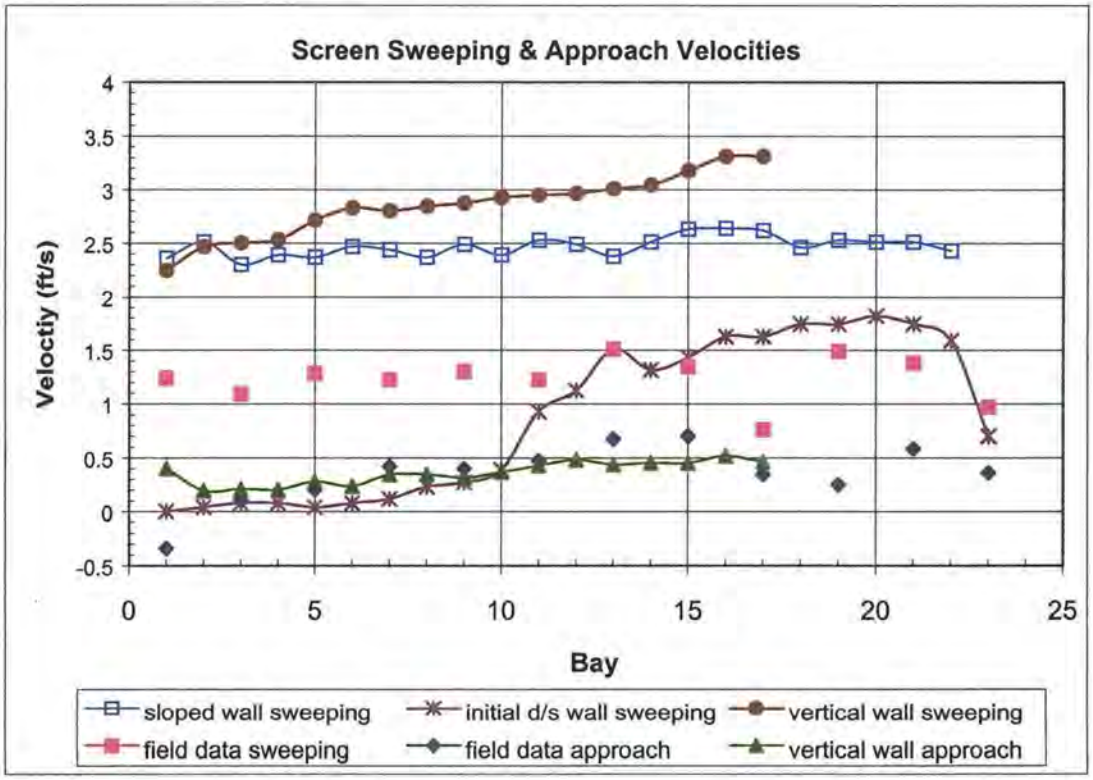


Figure 35. - Screen sweeping and approach velocity comparisons for the field screen, the initial model conditions and the vertical and 2:1 sloping wall geometries for a discharge of 640 ft^3/s and river water surface EL. 4681.

Figure 35 shows the sweeping velocities measured in the model initially without modifications. They show that the sweeping velocity is very low over about the entire upstream half of the screen and rise only to about 1.5 ft/s near the end of the screen. The field personnel observed this trend when the screen operated last summer. It was aided by the field baffling, but is obviously not acceptable performance.

The initial canal geometry was then modified to the final geometry of the vertical or sloped left wall from the head gate pier to the bypass entrance. These modifications, tests 13 and 14,

improved the general flow conditions and increased sweeping velocity at the beginning of the screen. Sweeping and approach velocities with the modified canal geometry show remarkable improvement with either the vertical wall or the sloped wall opposite the screen, figure 35. The sweeping velocities for the vertical wall are higher than those for the sloping wall and continue to increase downstream.

A spreadsheet was developed to check the screen area and approach velocities necessary to pass 640 ft^3/s through the screen with the vertical wall geometry. An average approach velocity of 0.35 ft/s is required to pass the desired discharge. The measured approach velocities are at or below the theoretical value of 0.35 and below 0.5 until nearing the end of the screen. At the end of the screen the sweeping and approach velocities increase with the vertical wall geometry because of the narrower area and lack of a bypass in the model. Some waves formed in the channel with the vertical wall because of the restricted area and no bypass flow. A good estimate of sweeping to approach velocity ratios for the vertical wall opposite the screen would be on average 8:1 and for the sloping wall 13:1, because the approach velocities were so low. All "hot spot" or discontinuities on the screen surface should be able to be handled by the baffles now that the flow conditions have been improved by use of either the vertical or sloping wall through the channel opposite the screen. With the improved hydraulic performance the screen should function properly.

The velocities, shown in figure 36, measured in the model for the higher discharge of 680 ft^3/s show excellent approach velocities for both configurations, except near the downstream end of the vertical wall geometry. The flow area is reduced there and the approach velocity is increased. The approach velocity would be less with the bypass and baffling could be performed

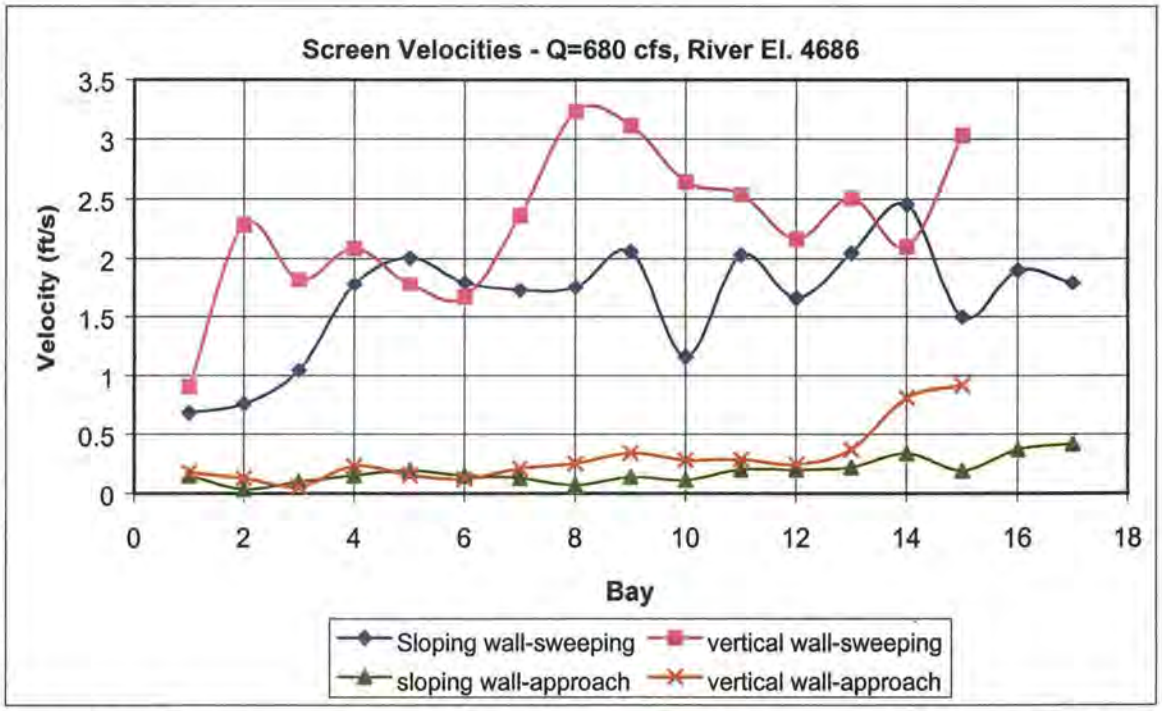


Figure 36. - Sweeping and approach velocities measured for a canal discharge of 680 ft/s and river water surface El. 4686. Note slightly higher values of both the sweeping and approach velocities for the vertical wall.

to decrease any localized problem. Sweeping velocities are rather inconsistent. Sweeping velocities are lower over the upstream portion when compared to the previous discharge but overall are adequate.

There was some concern that the sweeping velocity would be higher than the bypass entrance velocity. The bypass area of 2 * 5 ft = 10 ft^2 at the first gate would produce a velocity of 4 to 6 ft/s at the bypass, depending upon the bypass discharge. The screen sweeping velocities will not exceed the bypass velocities.

The velocities in both figures 35 and 36 indicate that the flow patterns have been greatly improved. The vertical or sloping wall with the pier deflector and right canal wall upstream from the screen may be constructed in the field with expected improved screen performance.