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DURANGO PUMPING PLANT
1:12 SCALE PHYSICAL MODEL STUDY

December 2002
(Revised January 2003)

U.S. DEPARTMENT OF THE INTERIOR
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As part of the design process for the Durango Pumping Plant of the Animas - La Plata Project, a 1:12 scale physical model study of the intake structure was conducted at the Bureau of Reclamation, Water Resources Research Laboratory at the Technical Service center in Denver, Colorado. Objectives of the study include: determination of intake capacity at various stream flow rates; identifying approach and sweeping velocity distributions along the fish screen; characterization of flow conditions in the pump intake forebay; and observation of pumping impacts on current patterns in the Animas River near the intake mouth. Data collection included: water surface elevations at locations along the Animas river as well as in the intake structure; measurement of discharges upstream of the division; delivered to the pump intake conduit, and flowing through the fish bypass; two-dimensional velocity measurements along the fish screen and in front of the debris screen at the intake mouth; and video documentation of dy injected into flows.

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by

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Denver Technical Service Center
Water Resources Services
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Denver, Colorado

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UNITED STATES DEPARTMENT OF THE INTERIOR ★ BUREAU OF RECLAMATION

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This hydraulic model study was conducted by the authors under the supervision of Al Bernstein, Durango Pumping Plant Design Team Leader; of Cliff Pugh, Manager of the Water Resources Research Laboratory Group; and in accordance with directions from Rick Ehat, Farmington Construction Office, Upper Colorado Region. The modeling effort was accomplished in cooperation with the Civil Engineering Services Branch, Structural and Architectural, and Water Conveyance Groups; and Infrastructure Services Branch, Hydraulic Equipment Group.

The contributions of Morris “Rudy” Campbell, for efforts in instrumentation, calibration, surveying and data acquisition; of the laboratory shop craftsmen for model construction; and of Cheryl Rieniets, for getting this report printed and distributed are greatly appreciated.

Tony Wahl was instrumental in providing conceptual framework for the modeling program and model construction layout, and in performing a Peer Review of this report.

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INTRODUCTION

A 1:12 scale physical model study was conducted from December of 2001 through August of 2002 for the Durango Pumping Plant (DPP) inlet structure of the Animas – La Plata (ALP) project at the U.S. Bureau of Reclamation's Water Resources Research Laboratory at the Denver Technical Service Center. The open channel portion of the intake structure, along with approximately 450 ft of the Animas River was modeled, beginning 98 ft upstream of the proposed inlet site. Primary objectives – as outlined at the Concept C briefing prior to initial testing – were to investigate the following aspects:

- Intake structure capacity
- Fish screen velocities
- Pump intake forebay
- Recreational aspects

The design process of this twice-revived project was on an accelerated, or "fast track" schedule in an effort to minimize the design time prior to initiation of construction. This decision was apparently based on the fact that much work – including physical modeling – had been done on previous, larger versions of the project. The fast track schedule resulted in a less than ideal chronology. Model construction was initiated despite the fact that a Value Engineering (VE) study hadn't yet been conducted.

The initial model construction was completed simultaneously with the VE team recommending significant changes. Thus the first configuration was obsolete upon completion. While changes were pending, preliminary testing was done with the initial model configuration in March and early April. By early May, design modifications that addressed issues raised in the VE study had been made to the model and testing resumed. Further design modifications were made in an effort to improve observed hydraulic performance. By mid June, tests of design-critical performance were essentially complete. Modifications to provide structural support for an access deck, to address sedimentation concerns, and to improve maintenance access were made late in the design process. These changes were tested to verify that they would not negatively impact hydraulic performance.

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1:12 SCALE PHYSICAL HYDRAULIC MODEL

Figure 1 is a plan sketch of the modeled section of the Animas River and the Durango Pumping Plant inlet structure in its final model configuration. Shown are contours on two foot intervals and the location of five streambed taps that were used to identify the water surface profile along the Animas River for modeled flow conditions. Flow delivered to the model was measured using venturi meters in the lab pipe system. Fish bypass discharge was measured using a flat plate orifice that was calibrated in the model. Flow to the DPP pumps was measured using a broad crested weir.

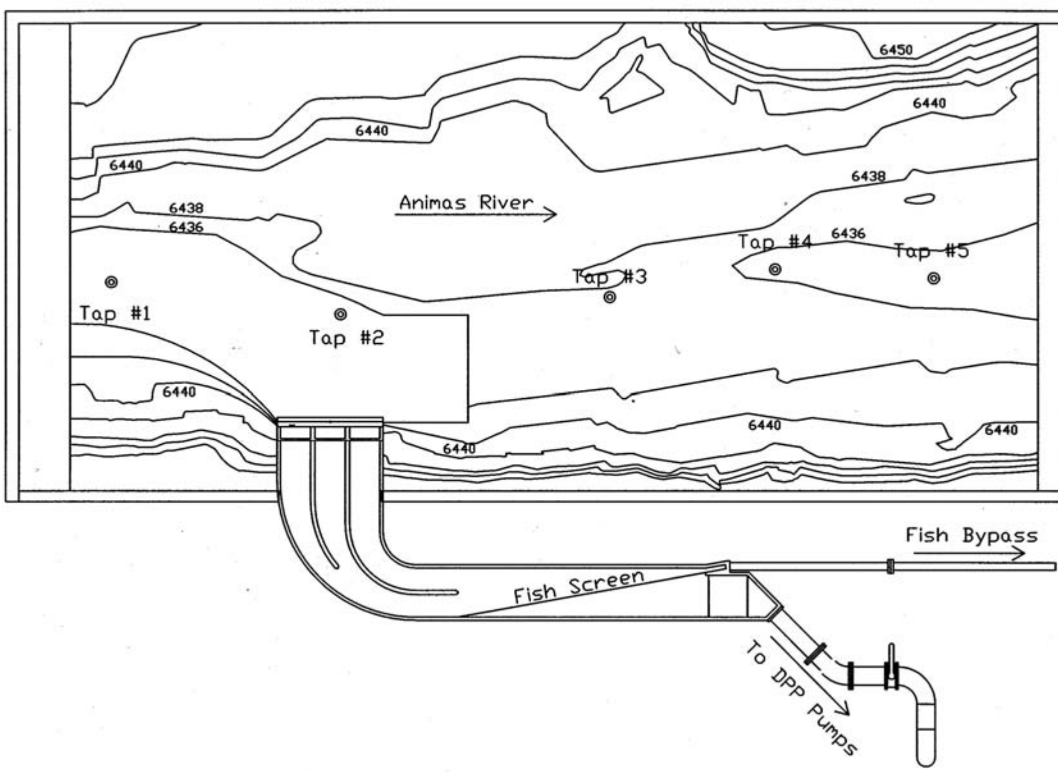

Figure 1. Plan sketch of the 1:12 scale model of the DPP inlet structure

The inlet mouth is 46’ 9” wide. This width is configured as three bays, each 14’ 3” wide, separated by two piers each 2’ 0” wide. Vanes extend from these piers to improve flow distribution along the fish screen. A secondary function of the vanes is support for an access deck that covers this section. Vanes are configured such that there is a minimum of 8’ 0” clearance between vanes for maintenance access in the intake.

Figure 2 is an overhead rendition showing the intake structure and pump building drawn by J. F. Pattie. The access deck above the vanes in the curved section is seen in this view. The pump inlet conduit, (underground), follows a straight path from the upper right portion of the inlet structure to what is the upper side of the pump building in this picture.

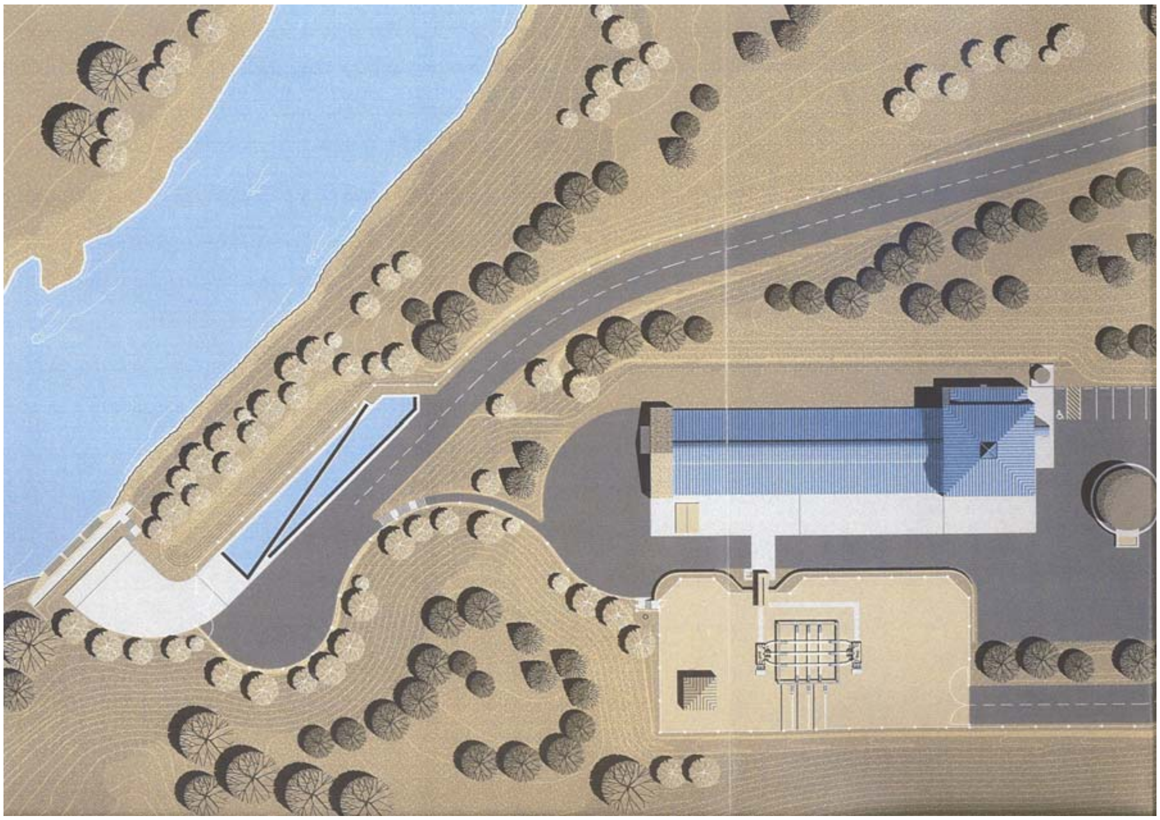

Figure 2. Plan DPP site rendition, (drawn by J. F Pattie, D-8120)

Downstream of the curved section, the inlet structure width is 23' 0". Twenty feet downstream from the end of the curve, a diagonal wall that supports the fish screen begins, oriented at an angle of 10^o with the right side of the channel. This diagonal wall crosses the channel from right to left as viewed in the direction of flow. The actual screen is 100' 0" long by 8' 0" high with an open area of approximately 40%.

At the point beyond the fish screen section where the diagonal wall is 2' 0" from the left wall of the intake structure, the left wall angles at 10^o. It then parallels the fish screen wall, forming a 2' 0" wide channel approximately 8' 10" long. At the downstream end of this channel is the entrance to the fish bypass conduit. On the downstream side of the fish screen, (right side of the channel), the channel width beyond the end of the screen is 19' 6". The channel walls are parallel at this width for approximately 19' 8" in the direction of flow. The invert elevation transitions down from 6429.75 to 6418.67 in this reach. The lower invert section is the forebay to the 84" diameter pump intake conduit.

NOTE: The point in the Animas River where the fish bypass conduit returns is beyond the downstream extent of the model. Hence, it was not possible to examine the hydraulics of the fish bypass conduit in this study. In the prototype, discharge through the fish bypass conduit will be a function of the head differential between the water surface elevation in the intake at the conduit entrance and the water surface elevation of the Animas River at the conduit outlet. In the model, flow through the fish bypass was controlled by adjusting a butterfly valve. For this study, fish bypass discharge was held constant at 30 ft^3/s for all tests modeling pump diversions.

Figure 3 is a sketch of the intake profile, showing the centerline of the structure as if laid out on a flat plane. The fish screen is displayed as it projects on a vertical plane running parallel to the inlet channel sides. Invert elevation at the mouth of the inlet is 6436.84. Invert elevation in the fish screen section is 6429.75, and sump elevation at the suction intake for the 84" conduit leading to the DPP pumps is 6418.67. The transition from 6436.84 to 6429.75 covers a 16' 0" horizontal distance immediately downstream of the trash racks. The sediment excluding crest gates are mounted at the upper end of this transition.

Figure 3 shows the third profile configuration tested. An initial modification was made to improve hydraulic performance of the model. The second modification was made to reduce potential for sediment accumulation both upstream and immediately behind the crest gates. Profile sketches of all tested profile configurations are included in the Appendix.

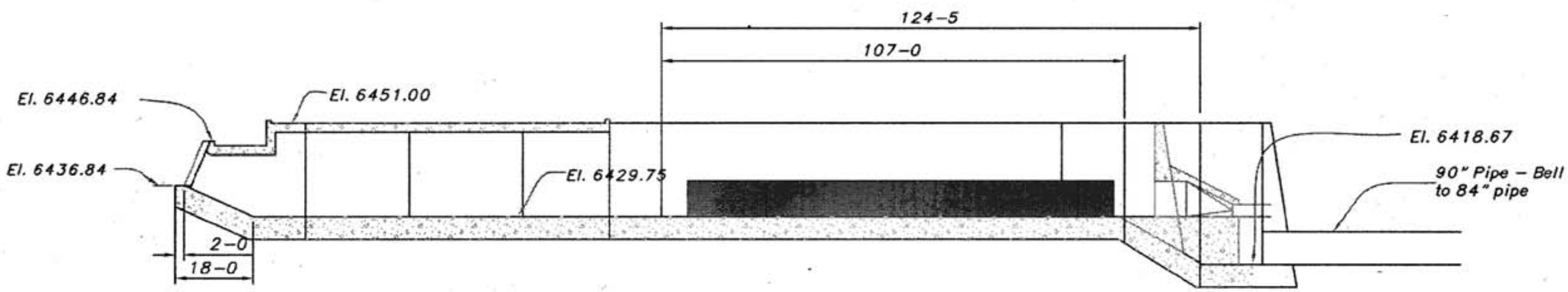

Figure 3. Centerline profile of DPP Inlet

NOTE: Between the intake mouth and fish screen, the structure makes a 90° turn. In this sketch, the true scaled length of the centerline through the curve is displayed. Moving either direction from the curve, the centerline sections shown are planar, hence true scaled lengths are shown. The fish screen is oriented at an angle of 10° with channel walls. The displayed length of the fish screen is the scaled length of the projection of the fish screen onto a plane parallel with the channel walls.

Figure 4 is a photograph of the right bank of the modeled stretch of the Animas River including the intake structure. Stream flow is from upper right to lower left in the picture. Rocks shown in the river channel were placed to simulate size and position information from a channel boulder survey that was performed prior to the first model study for the Durango Pumping Plant, conducted in 1993.

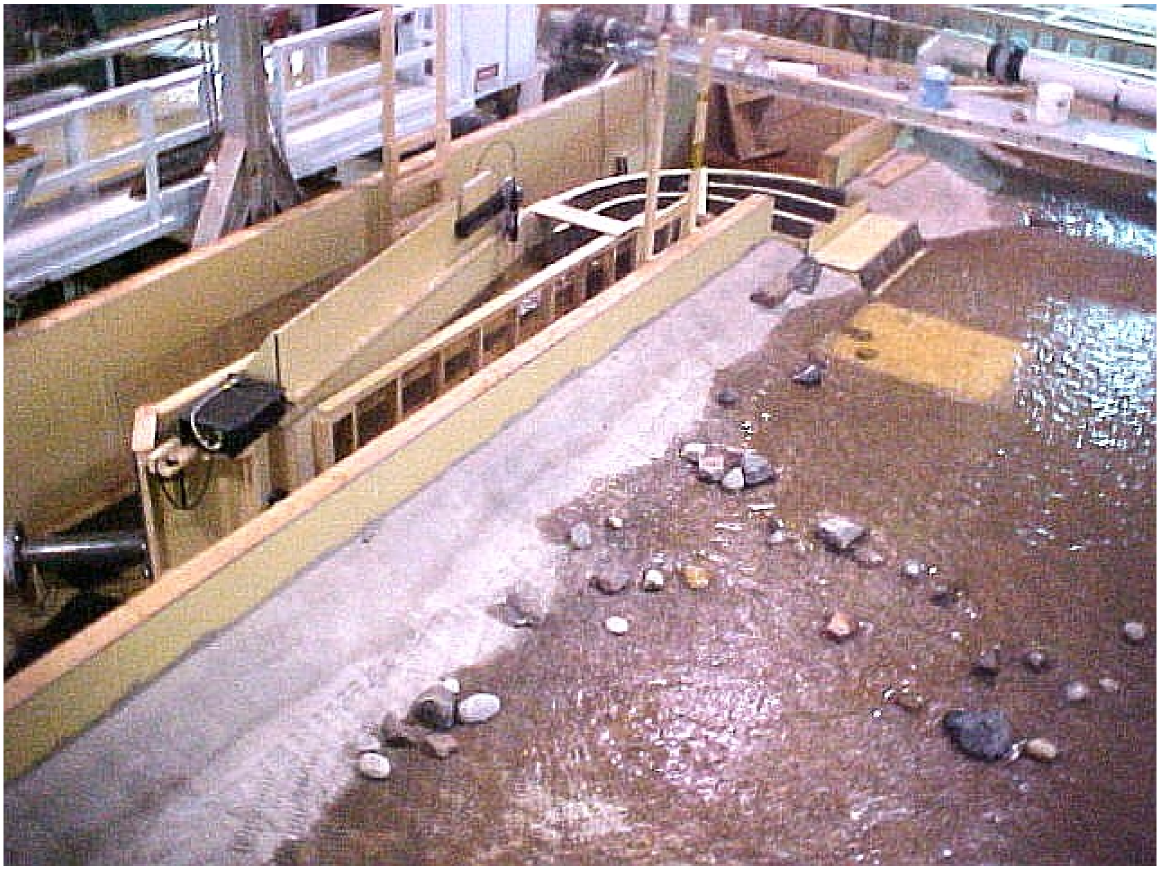

Figure 4. 1:12 scale model of the Durango Pumping Plant intake structure

The fish bypass conduit is not visible in Figure 4. Pump flow exits the intake sump through the black pipe shown just below the mid-left side of the photo. As the model was being designed, there was uncertainty whether channel modifications near the intake mouth to enhance hydraulic performance might be investigated. The yellow rectangle near the intake mouth does not have the concrete covering seen in the rest of the streambed so that possible channel modifications could be easily made in that area.

Water Surface Elevation – Stream Discharge relationship: Initial testing with the model was to identify water surface profiles for selected stream discharges with no diversions. Water surface elevations were obtained using a manometer board connected to the five streambed taps shown in Figure 1 in the preceding section of this document. Of key interest was the water surface elevation – stream discharge relationship at the mouth of the DPP intake. Flow delivered to the pumps and through the fish bypass will be dependent on the river stage at this point. Figure 5 is a plot of water surface elevations near the inlet mouth, (tap #2) observed in model testing, along with a curve fitted to the data. Plots of water surface elevations for all taps are in the Appendix.

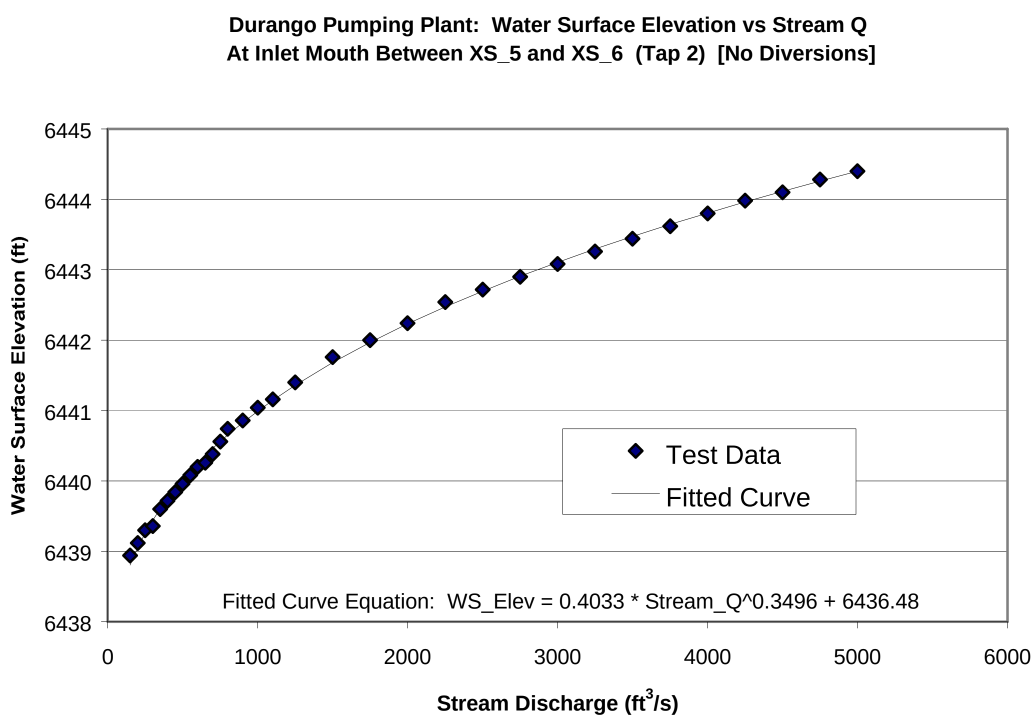

Figure 5. Observed water surface elevation – stream discharge relationship in 2002 model of Animas River at DPP inlet

In order to determine the validity of the model's ability to simulate actual conditions, this information was compared with data assembled in previous work including field data, results of previous model studies and numerical model output. A similar task had been performed by Tony Wahl, (D-8560 [formerly D-3752]), in an effort to establish validity of water surface – stream discharge observations from the 1992 model study. His finding at that time was that agreement was not particularly good between his physical model and the existing field data. Numerical modeling, using PSEUDO software was done concurrently with the 1992 model tests by Joseph Lyons, (D-8520 [formerly D-5753]).

In a January, 1993 report, (a copy is in the Appendix of this document), examining discrepancies between field data and physical model observations, it is noted that using the PSEUDO numeric model, an unusually high Manning's roughness coefficient of 0.080 is required to obtain surface elevations similar to the field data, while using a more reasonable coefficient value of 0.040 results in surface elevations similar to values from the 1992 physical model. To evaluate performance of the current physical model, the fitted curve from Figure 5 is plotted with values from field data, from the 1992 physical model study, and from 1992 output of the PSEUDO numeric model using a Manning's coefficient of 0.040. This information is displayed in Figure 6.

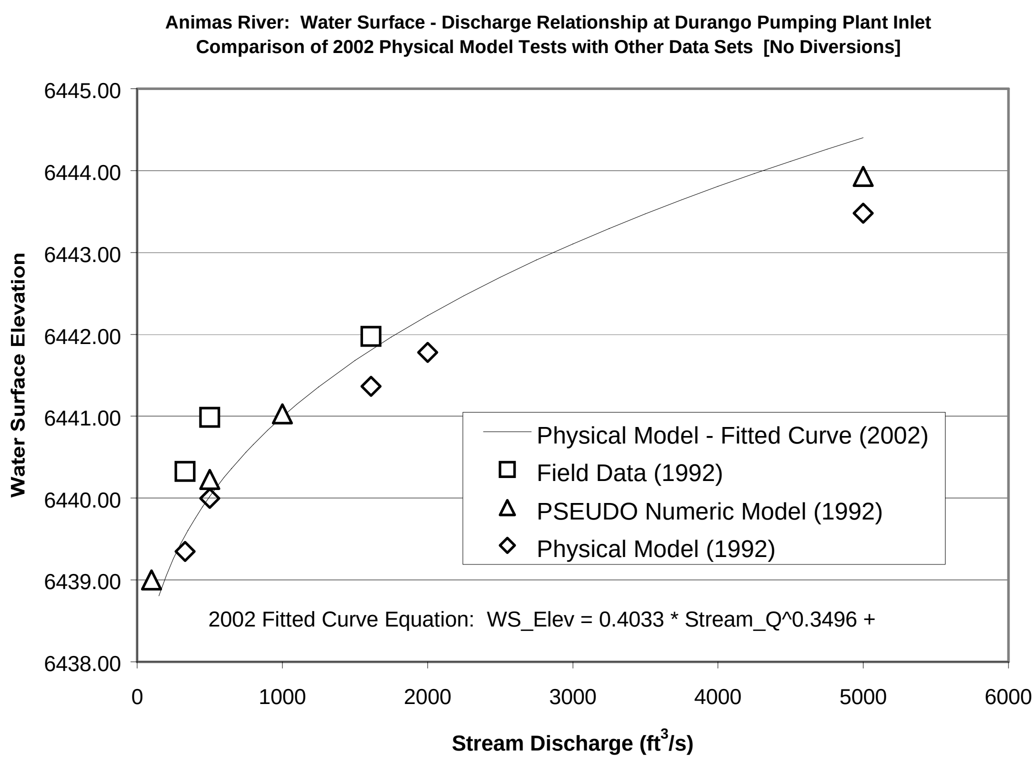

Figure 6. Comparison of 2002 physical model water surface – stream discharge observations with other data sets

At lower stream flows, diversion discharge will be hydraulically limited by the water surface elevation at the inlet mouth. As seen in Figure 6, there is reasonable agreement in water surface elevation at stream discharges below 1000 ft^3/s among both the current and 1992 physical model studies as well as with the numeric model output, (using a 0.040 roughness coefficient). These three data sets predict water surface elevations significantly below that indicated by the limited body of field data available.

The degree of agreement of the non-field data information was accepted as a suitable level of validation for the model's ability to simulate the prototype conditions. Since the water surface elevations observed with the current physical model for stream discharge less than 1000 ft^3/s are lower that those indicated by the field data, using the current physical model data would be conservative from a design perspective.

Intake Structure Capacity: Tests were performed to determine the relationship between discharge that could be diverted for pumping and stream discharge rates. The tests were operated under two constraints. Water level in the intake structure was maintained at or above the top of the fish screen at all times in order to maintain the maximum cross-sectional flow area through the screens. Levels were monitored using taps placed in the left wall of the inlet located across from either end of the screen.

The second constraint was maintaining a fish bypass flow of 30 ft^3/s. As noted above, this value is a simplifying approximation. [The 30 ft^3/s value is identified as the approximate fish bypass discharge on page 31 of the ***ALP-DPP Design Data Update, January, 2002*** booklet.] Actual fish bypass discharge will vary as a function of the head differential between water level in the intake at the fish bypass conduit entrance and elevation of the water surface in the Animas River at the conduit exit.

In the first configuration tested, (the design reflecting modifications suggested by the Value Analysis team), the drop in invert elevation from 6435.78 to 6429.75 occurred at the downstream end of the curved section. In that configuration the fish screen was mounted atop a 6" curb at elevation 6930.25. A stream discharge of approximately 800 ft^3/s was the minimum at which the full 280 ft^3/s pump discharge could be achieved.

In an effort to improve hydraulic performance, design modifications were implemented. The location of the drop in invert elevation from 6435.78 to 6429.75 was moved from the downstream to the upstream end of the curved section. The 6" curb was also removed leaving the bottom of the fish screen at elevation of 6429.75. Additionally two vanes that began at the two piers dividing the intake mouth and extending downstream through the curved section were installed. With this configuration, the full 280 ft^3/s pump discharge was achieved with a stream discharge of approximately 580 ft^3/s.

A modification made late in the design process to reduce sedimentation potential near the crest gates was tested to ensure that hydraulic performance was not diminished. A comparison of the pump limit – stream discharge relationships of all model configurations is presented in Figure 7. As can be seen in this plot, the latest configuration shows little change when compared with data from tests with the second configuration. Test data indicates that the full 280 ft^3/s pump discharge may still be diverted from a river discharge of approximately 580 ft^3/s.

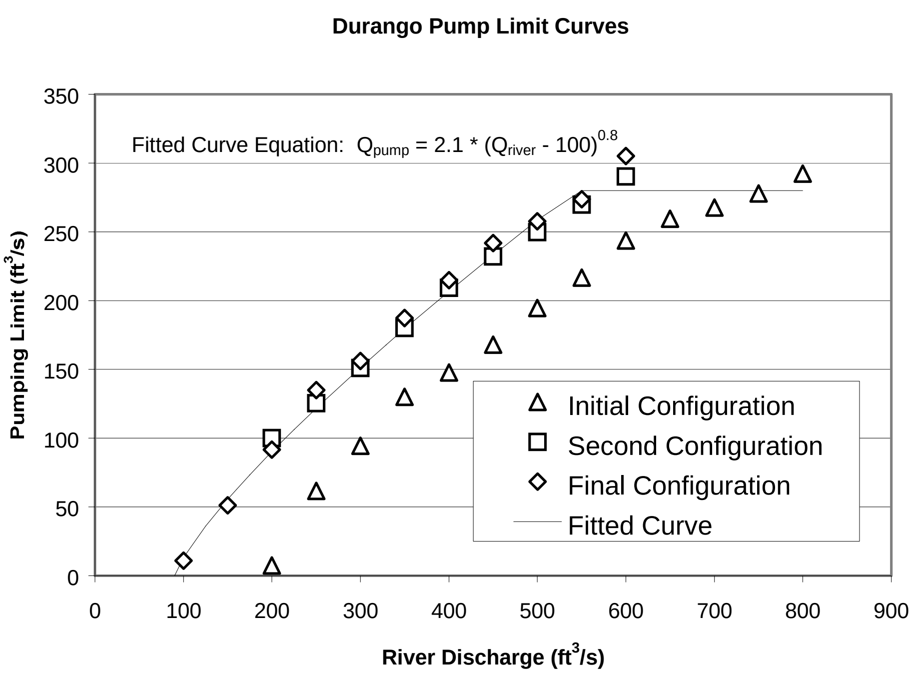

Figure 7. Hydraulic pumping limit – stream discharge relationships

In addition to hydraulic limits, ability to divert will be constrained by seasonal aquatic bypass flow requirements. These limits are: 225 ft^3/s, (April – September); 160 ft^3/s, (October – November); 125 ft^3/s, (December – March). In Figure 8, these constraints are shown with the fitted curve from Figure 6 up to the full pump discharge of 280 ft^3/s.

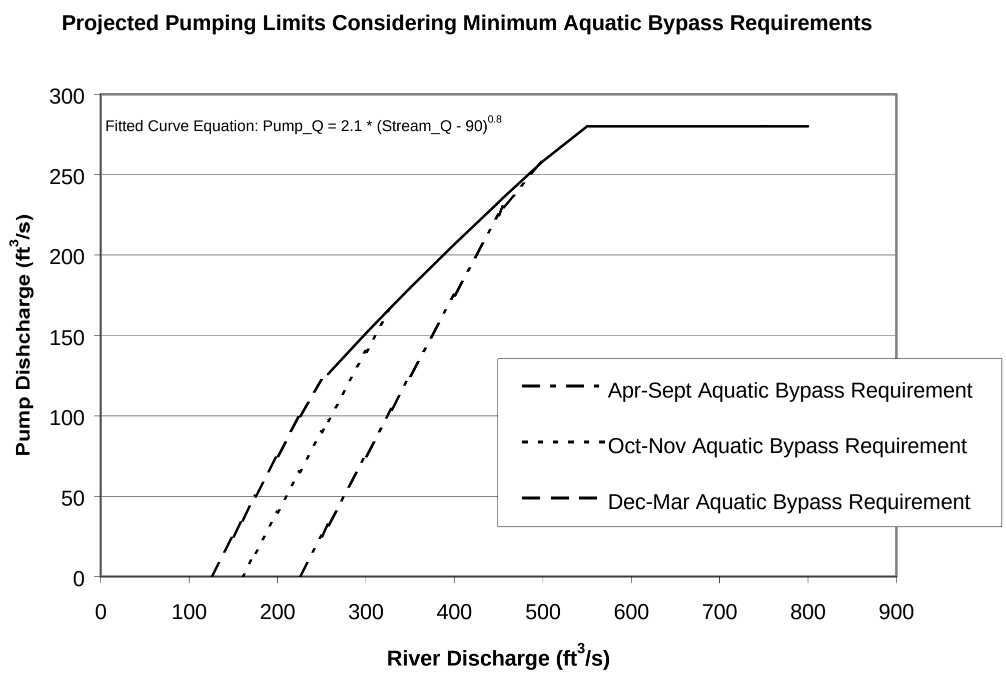

Figure 8. Hydraulic pump limit and seasonal aquatic bypass constraints

The overshot crest gates will be installed in the intake immediately behind the trash rack to reduce diversion of sediments. At high flows, the gates will be raised to limit the amount of bed-load sediment that enters the intake. When fully raised, the elevation of the overshot gates will be 6439.84 – three feet above the intake crest elevation of 6436.84. Tests were performed over a range of stream discharges to identify the hydraulic pump limits with the overshot gates in fully raised position.

In Figure 9 the limit with gates raised is plotted with the limit with gates lowered. Note that the stream discharge at which the full 280 ft^3/s pump discharge is achieved is approximately 2160 ft^3/s, compared with full pump discharge at stream discharge of 580 ft^3/s with lowered gates.

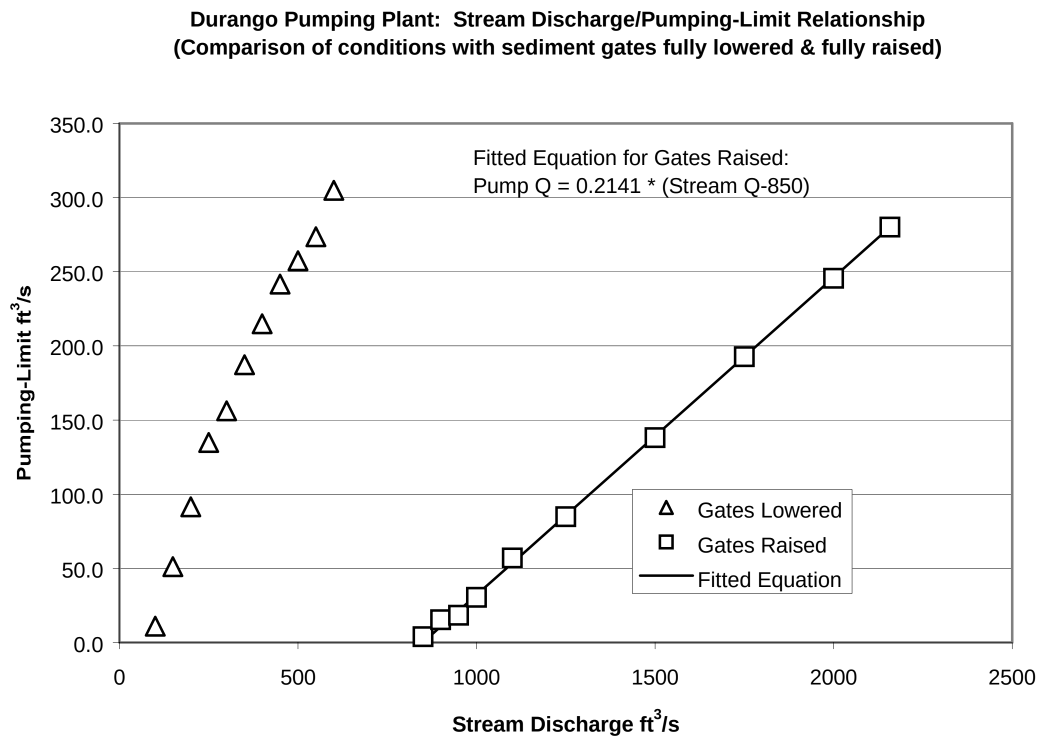

Figure 9. Comparison of hydraulic pump limit curves with crest gates fully lowered and fully raised

Fish Screen Velocities: Two-dimensional velocities in the horizontal plane were obtained for flows at the fish screen. Approach, (normal to the screen), and sweeping, (parallel to the screen) velocities were measured approximately three inches (prototype) in front of the screen using an Acoustic Doppler Velocimeter, (ADV), from Sontek. Readings were taken at 2, 4 and 6 ft above the screen invert.

The ADV probe was mounted on a traversing table that covered a span of 21 ft down and back along the screen per cycle. The sensor was programmed to make 25 readings per second and the cycle time of the traversing table was 70 seconds. The data obtained was separated in 0.6 second segments and averaged to obtain a mean velocity for 3 ft increments. The effect of the traverse travel speed was negated by averaging the values obtained as the probe traveled in each direction past each 3 ft increment along the screen. Confined space prevented making velocity measurements over the furthest downstream 8.5 ft of the fish screen.

In order to obtain an acceptable approach velocity distribution, the mechanical design group, [Rick Christensen, D-8410], indicated that louvers would be installed on the downstream side of the fish screens. The design criteria used for this screen is an approach velocity of 0.5 ft/s or less.

The critical condition is at the minimum stream discharge for which the full pump discharge is possible. Under this condition, flow through the screen is at the maximum, and the depth of flow in the intake is equal to the height of the fish screen – the minimum operational flow depth. During testing, the most uneven approach velocity was observed for this flow condition. Additionally, a minimum amount of head would be available for using the louvers to adjust approach velocity distribution.

To verify accuracy of values obtained with the ADV, a crude check of continuity was performed by integrating the individual approach velocity values over the area segments where each value was observed. Averaged values for the furthest downstream 10 ft for which readings could be obtained were extrapolated across the 8.5 ft at the end of the screen where readings could not be taken. For test conditions discussed below, calculated pump discharges fell within a range of 86% to 115%, indicating a reasonable degree of accuracy in observed approach velocity values.

Figure 10 is a plot of the fish screen approach velocity distribution for the model in the initial configuration. As shown in Figure 6 above, minimum stream discharge for full pump discharge with this configuration was 800 ft^3/s. This model geometry featured an invert elevation of 6435.78 through the curve with no vanes. During tests it was observed that flow impacted the right wall near the end of the curve, then reflected back toward the left wall along the upstream reach of the fish screen. This observation is corroborated by the plotted approach velocity distribution. Note the low approach velocities along the upstream third of the fish screen and the high velocities further downstream.

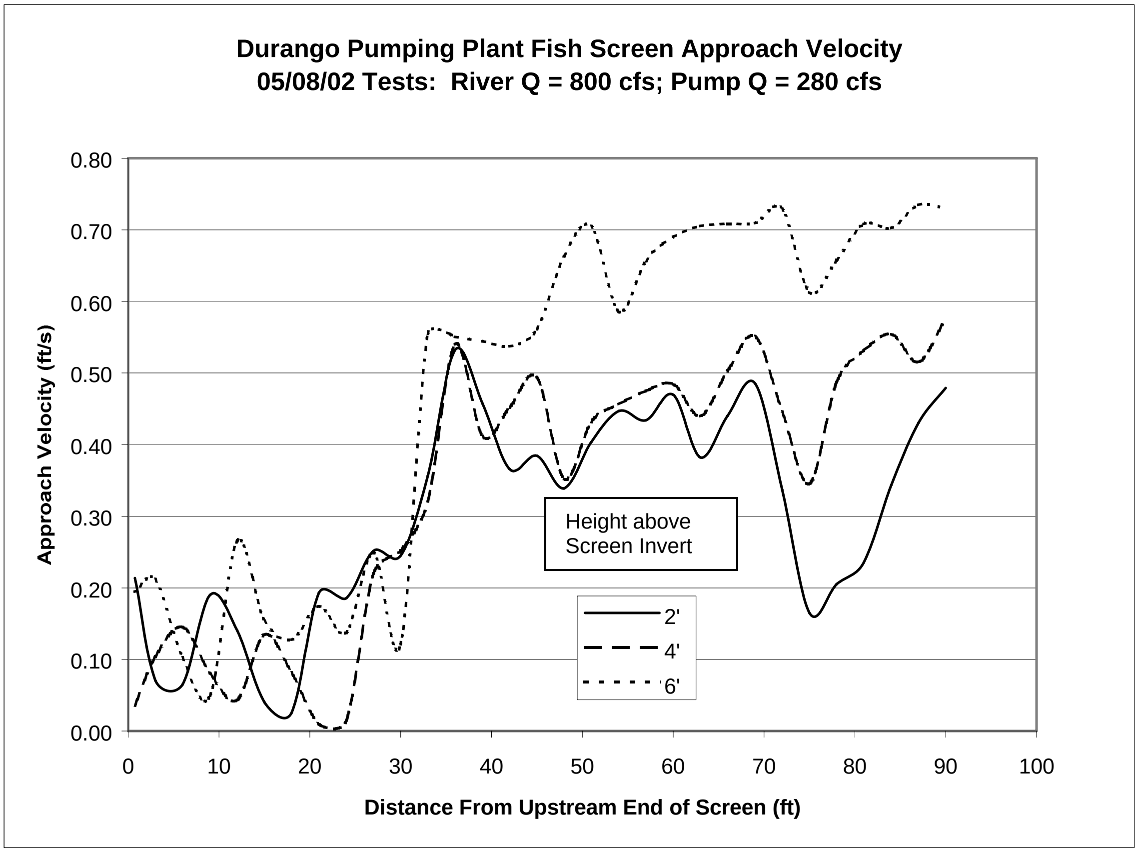

Figure 10. Fish screen approach velocity distribution, initial model configuration; Stream Q = 800, Pump Q = 280 ft³/s

With the initial model configuration, as Figure 10 shows, there was poor distribution of the approach velocity along the fish screen and velocities well in excess of 0.5 ft/s were observed. As noted in the *Intake Structure Capacity* section, the original configuration of the model was modified by moving the location of the drop in invert from 6435.78 to 6429.75 from the downstream end of the curve to a position upstream of the curve. A 6" curb at the base of the fish screen was removed. At the same time, vanes were installed that extended through the curved section to improve distribution of flow in the inlet. Improvements were seen both in reduced energy loss and improved distribution of flow through the fish screen. As previously noted, full pump discharge was achieved with the modified design with a stream discharge of 580 ft^3/s. Figure 11 shows the improvement in fish screen approach velocity distribution.

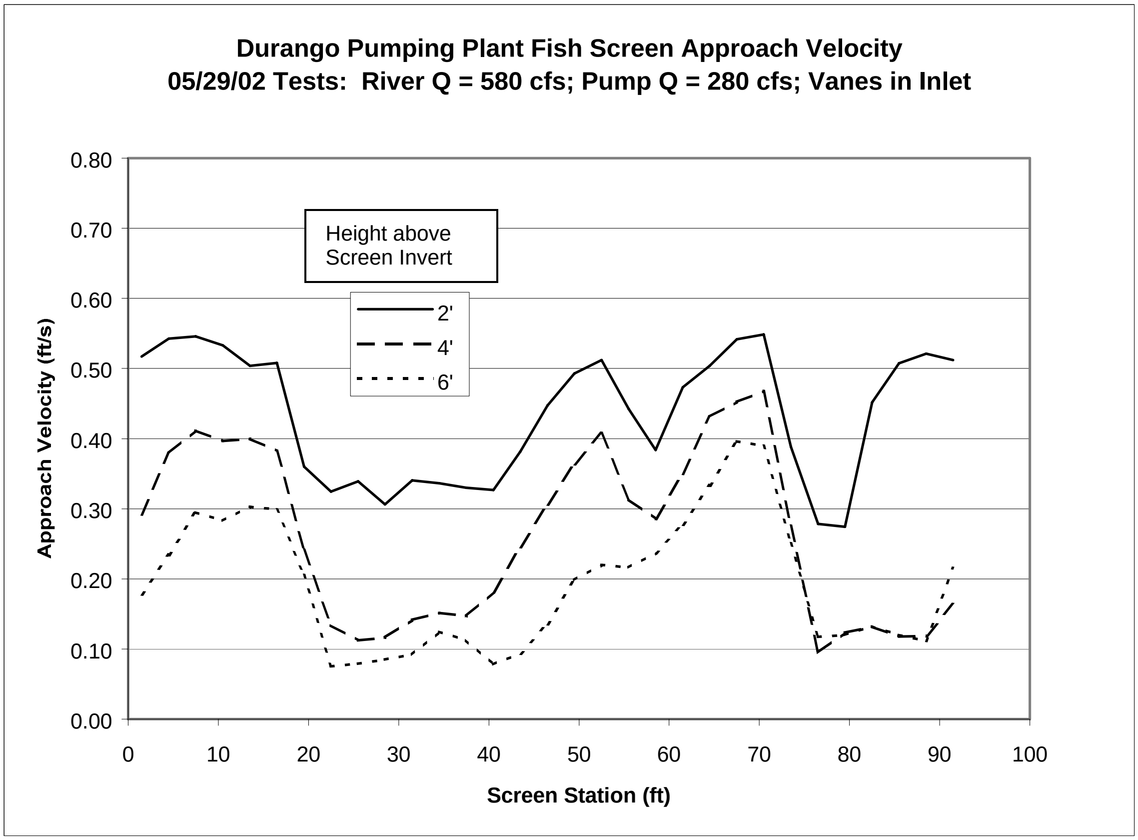

Figure 11. Fish screen approach velocity distribution, Modified model configuration w/vanes to end of curve; Stream Q = 580, Pump Q = 280 ft³/s

Following these tests, (i.e. results shown in Figure 11), the guide vanes were extended an additional 20 ft for to meet structural objectives. This modification was modeled to determine what effect the change would have on hydraulic performance of the inlet. For a stream discharge of 580 ft^3/s, the full pump discharge of 280 ft^3/s could be diverted. Hence the vane extensions do not negatively impact intake capacity. Figure 12 shows the approach velocity distribution for this test. The vane extensions appear to have improved the approach velocity distribution.

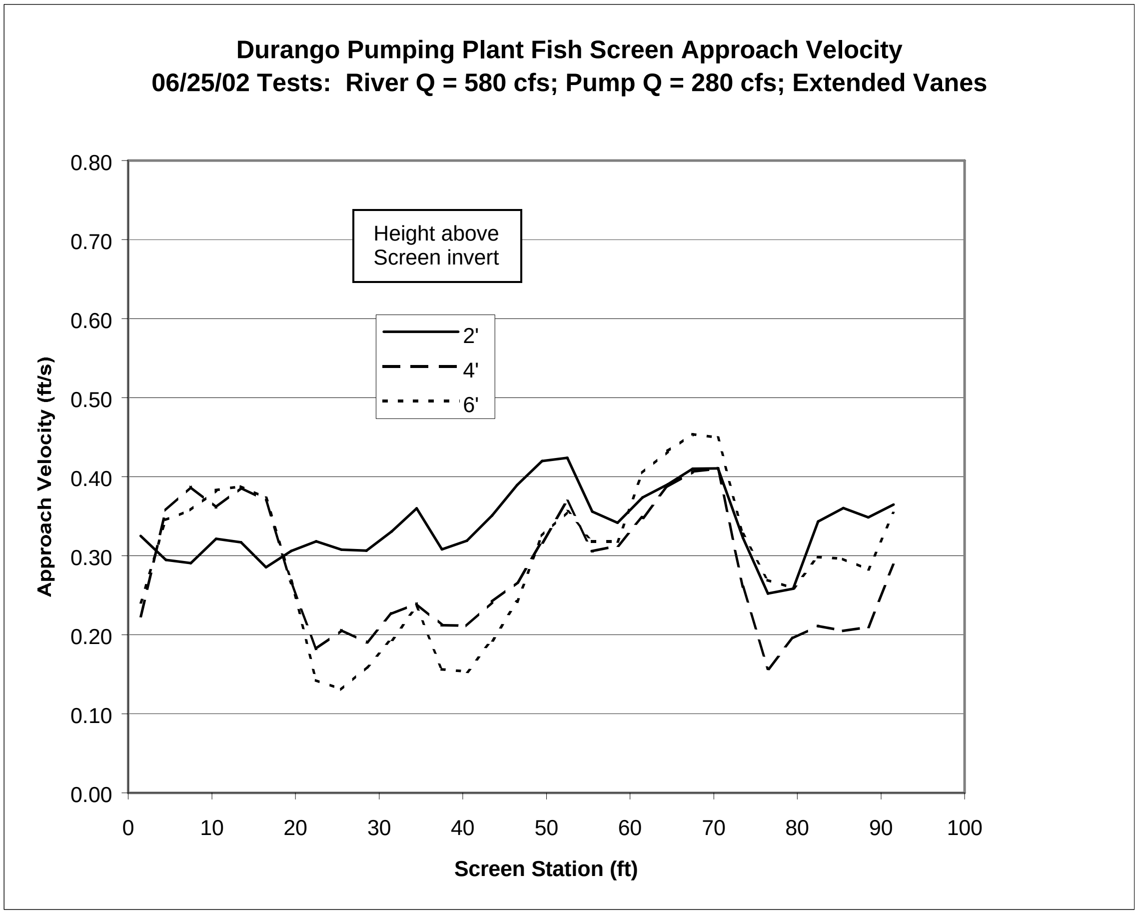

Figure 12. Fish screen approach velocity distribution after installation of 20' vane extensions Stream Q = 580 ft³/s Pump Q = 280 ft³/s

A late modification was made to the vane configuration. To address maintenance concerns, the vanes were reconfigured to maintain a minimum of 8' 0" clearance between vanes/walls at all locations upstream from the fish screen wall. In this orientation, the left vane begins at the left pier in the inlet mouth and ends in the center of the channel at a point normal to the upstream end of the fish screen wall. The right vane begins at the right pier in the inlet mouth and extends downstream through the curve bisecting the channel width between the left vane and the right wall. The right vane ends where clearances between the right wall, the right vane and the left vane are 8'0". Results of tests with this guide vane configuration, shown in Figure 13, indicate a further improvement in approach velocity distribution along the fish screen.

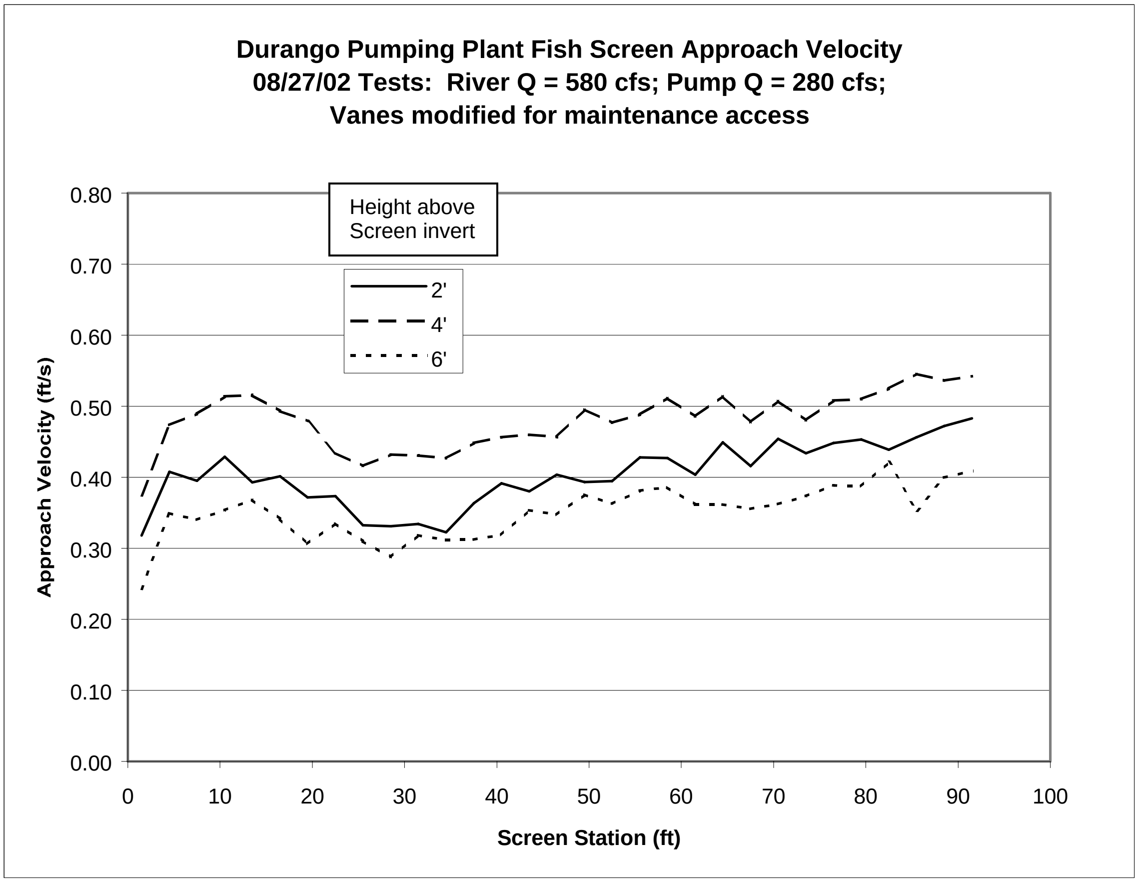

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Figure 13. Fish screen approach velocity distribution with vanes reconfigured to maintain minimum 8' 0" access clearance Stream Q = 580 ft³/s Pump Q = 280 ft³/s

Pump Intake Forebay: Data collection regarding flow conditions at the pump intake forebay included recording water surface elevations and making visual observation of flow conditions, (looking for vortex formation and/or other undesirable flow conditions). In general satisfactory flow conditions were observed for the test conditions. Critical flow conditions, (580 ft^3/s stream flow, 280 ft^3/s pumping rate), do appear to be near the threshold of vortex formation in the model sump. For this test condition, circular currents were observed near the water surface in the proximity of the pump intake conduit entrance as were temporary vortex dimples on the water surface. These conditions were documented on video tape with dye injected in the flow.

Observations made while adjusting to critical flow conditions provide additional indications that critical flow conditions are near the vortex formation threshold. Small temporary vortices was observed for near capacity pump discharge with water surface level in the intake below the top of the fish screens, and for pump discharge greater than 280 ft^3/s with intake water surface level at the top of the fish screens.

Corresponding performance between a prototype and model is a function of the degree of similitude present. Complete similitude is a condition rarely possible to achieve due to unavailability of modeling fluids with suitable properties, (including density, surface tension, viscosity and elastic compressibility), to meet the scaling ratios. For free-surface turbulent flow, accurate modeling is normally possible for geometrically similar models if the Froude number ratio of model and prototype are equal and if the model has a sufficiently high Reynolds number. [The Froude number is a dimensionless ratio comparing gravity forces with inertial forces while the Reynolds number is a dimensionless ratio comparing viscous forces with inertial forces.]

Near a pump intake entrance, the approach Reynolds number is of interest for modeling possible formation of vortices. Values for approach Reynolds number that have been identified as lower limits for good modeling are: 2×10^5 for modeling surface vortices and 3×10^4 for modeling submerged vortices. The approach Reynolds number in the pump intake forebay of the DPP model for critical flow conditions, (maximum pump discharge and minimum operating water level in the intake), is 3.06×10^4. This value is near the lower limit for modeling submerged vortices and below the limit for modeling surface vortices. Thus vortex formation in the prototype may be amplified compared with observations with the DPP model. Calculation of the approach Reynolds number at the pump intake forebay is included in the Appendix.

Figure 14 is a photograph of the pump intake forebay during model operation. Table 1 is a summary of observed pump forebay water surface elevations for tested conditions.

Figure 14. Pump intake forebay during model operation
Flow conditions in photo: Stream Q = 580 ft^3/s Pump Q = 280 ft^3/s

Table 1. Water Surface Elevations in Pump Intake Forebay for Tested Conditions

Stream Q (ft^3/s)	**Pump Q** (ft^3/s)	**Water Surface Elevation in Forebay** (ft)	**Crest Gate Pos.**
200	100	6438.28	down
400	207	6438.22	down
580	280	6438.10	down
1200	280	6440.13	down
2160	280	6437.74	up
3600	280	6442.89	up

It should be noted that for stream discharges greater than 580 ft^3/s, operating the sediment-control gates to automatically adjust to a downstream level, (i.e. a target pump intake forebay water surface elevation), would effectively minimize intake of bed-load sediments while allowing maximum pumping rate. This would also mean that the minimum operational water surface elevation in the forebay would be the 6438.10 ft observed for 580 ft^3/s stream discharge.

Recreational Aspects: Model tests investigating effects of pumping on recreational activities focused on flow patterns near the inlet mouth and changes in river stage. For selected stream discharge rates, pump discharge was set at the lesser of the hydraulic pumping limit or 280 ft^3/s. Manometer board readings for the five streambed taps were recorded, and video was made of dye injection in the river just upstream of the intake mouth. For comparison, pump and fish bypass discharge was then shut off. Manometer readings for the no-diversion condition were recorded and a video of dye injection into the flow was again recorded. Table 2 shows the water surfaces observed for the tested stream flow rates, both with and without pump and fish bypass diversion.

Table 2. Animas River near DPP Intake: Water Surface Elevation Differential, with and without Pumping

			Observed Water Surface Elevations				
Streambed Tap #			**1**	**2**	**3**	**4**	**5**
Distance along stream from center of intake (ft)			*-103.6*	*4.9*	*131.9*	*210.8*	*285.1*
Stream Q *(ft^3/s)*	**Diversion** *(ft^3/s)*	**Gate Pos.**					
200	130	down	6438.45	6438.45	6438.21	6437.85	6437.67
200	0	n/a	6438.99	6438.99	6438.81	6438.33	6438.09
400	237	down	6438.93	6438.87	6438.69	6438.27	6438.09
400	0	n/a	6439.71	6439.65	6439.53	6438.81	6438.69
580	310	down	6439.29	6439.17	6439.05	6438.57	6438.33
580	0	n/a	6440.19	6440.13	6440.01	6439.17	6438.99
1200	310	down	6440.73	6440.73	6440.61	6439.53	6439.41
1200	0	n/a	6441.33	6441.33	6441.15	6440.01	6439.83
2160	310	up	6442.05	6442.05	6441.87	6440.67	6440.49
2160	0	n/a	6442.41	6442.41	6442.17	6440.91	6440.67
3600	310	up	6443.25	6443.25	6443.07	6441.57	6441.21
3600	0	n/a	6443.49	6443.49	6443.25	6441.63	6441.39

The greatest drop in water surface elevation was observed for a 580 ft^3/s stream discharge with a 280 ft^3/s pump discharge. For this test condition, drop in surface elevation near the intake was almost one foot. Figure 15 is a plot comparing water surface elevations with and without diversion for a 580 ft^3/s stream flow. Plots for all tested conditions are included in the Appendix.

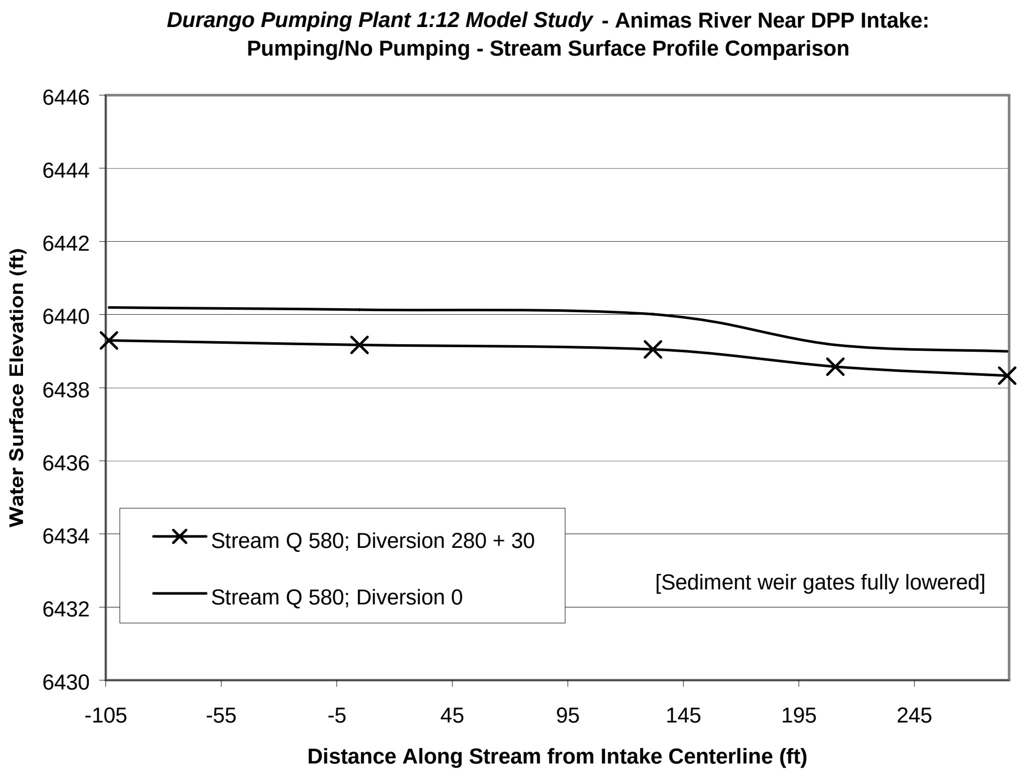


Figure 15. Diversion/no diversion - water surface elevation profiles for 580 ft^3/s stream discharge

Potential backwater effects due to outflow from the fish bypass conduit – not seen in the model – should be considered. The returning fish bypass flow would comprise approximately 10% of the downstream flow. This reach of the Animas River appears to be a pool and riffle sequence with a riffle zone between 100 and 250 feet downstream from the inlet. At lower stream discharge rates flow through the riffle would typically be near critical or supercritical. Thus fish bypass backwater effects that would be observed at the inlet for flow conditions shown in Figure 15 will likely be negligible.

A copy of video of these tests has been forwarded to Barry Longwell of the Project Team in Durango. Figure 16 is a photograph showing dye injection typical of the videoed tests. (Flow in the Animas River is right to left as shown.)

Figure 16. Dye injection upstream of DPP intake

Using the ADV equipment, near-surface velocities were measured in front of the intake trash rack. Using the same two-dimensional probe and traversing set-up that was used for measuring velocity in front of the fish screens, approach and sweeping components of velocity were obtained. The 48’ 6” width of intake mouth is configured as three bays 14’ 3” wide separated by two piers each 2’ 0” wide. Velocity data obtained in tests was segmented into nine groups, each representing a one-third span of a 14’ 3” wide bay.

Resultant velocity vectors were calculated from the mean value of points from each data group. The highest observed resultant velocities occurred for a stream discharge of 580 ft^3/s, the minimum at which the full pump discharge can be diverted. Figure 17 is a sketch of the near-surface velocity field near the trash rack for a 580 ft^3/s stream flow with a 280 ft^3/s pump discharge, (+ 30 ft^3/s fish bypass = 310 ft^3/s total intake discharge). Sketches of velocity vectors for all tested conditions is included in the Appendix.

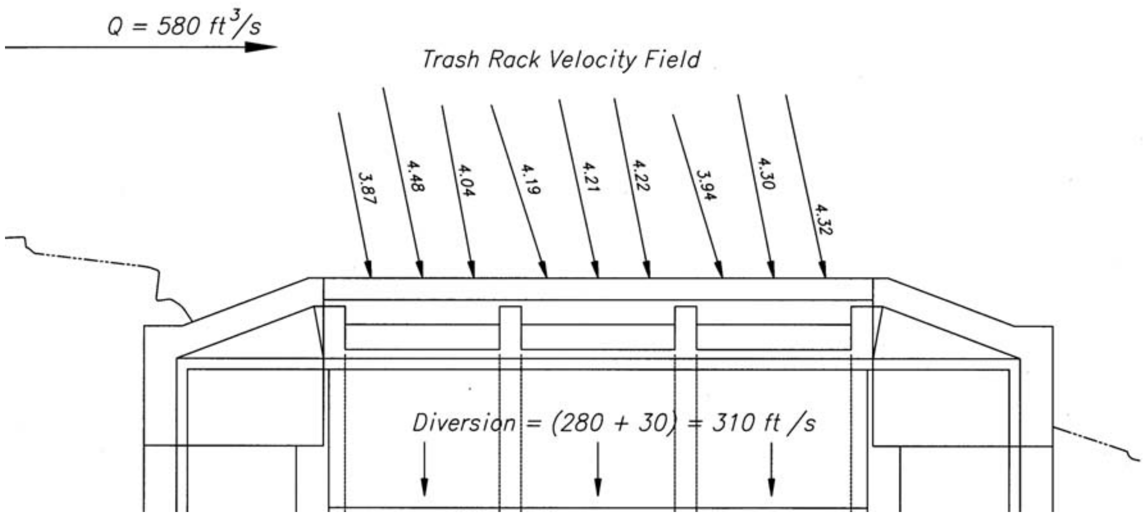


Figure 17. Near-surface velocity field in front of DPP intake trash rack Stream Q = 580 ft^3/s; Diversion = (280 [Pump] + 30 [FBP]) = 310 ft^3/s

Sediment Transport Enhancement Tests: Concern was raised in the course of the design process over the possibility that accumulation of bed-load sediments near the inlet could become a maintenance issue. Sediment loading projections are the topic of an analysis dated May, 2002, and of a revised analysis dated September, 2002 by Joesph Lyons, D-8850. Daily discharge estimates for the Animas River were examined for water years 1929 – 1993. The month of June is identified as the highest average daily flow month over the period of record. A flow of 2456 ft^3/s is identified as the median daily average June discharge and 5382 ft^3/s as the maximum average daily June discharge. Projected sediment loading for flow diverted into the DPP intake is based on these discharges. Copies of both analyses are in the Appendix.

A limited scope model study to assess the effects of placing a large boulder in the stream near the intake mouth was added to the testing schedule. A boulder of approximately 12 ft diameter was placed near the bank downstream from the intake. Tests were conducted with the boulder in two positions, and with the boulder removed. Velocity was measured with the ADV at two locations, 5.4 ft and 8.4 ft downstream of the left side of the intake opening, in line with the leading edge of the intake crest. Tests were conducted with the maximum pump diversion allowable for the respective stream discharges, with the crest gates fully lowered.

For 200 ft^3/s stream flow, velocity was measured 0.5 ft above the intake invert, (elev. 6437.34). For all greater stream flows, velocity was measured 1.0 ft above the invert, (elev. 6437.84). For discharges of 1200 ft^3/s and greater, a second velocity measurement was made 2.0 ft above the inlet invert, (elev. 6438.84). For both positions of the boulder, it was centered approximately 10.5 ft out from a vertical plane passing through the leading edge of the intake crest. In position “A”, the most upstream edge of the rock was approximately 5.5 ft downstream from the left side of the inlet mouth. In position “B”, the upstream edge of the rock was approximately in line with the left side of the inlet mouth, looking downstream into the intake.

Increases in observed velocity that result from forcing flow to pass around the boulder are more pronounced at higher flows. The tested discharge of 2400 ft^3/s is near the 2456 ft^3/s flow identified by Lyons as the median daily average June flow rate. Figures 18 – 20 show the observed velocities with no boulder, the boulder in position A, and the boulder in position B, respectively, for the 2400 ft^3/s tests. Sketches of observed velocities for all tested flow rates are included in the Appendix

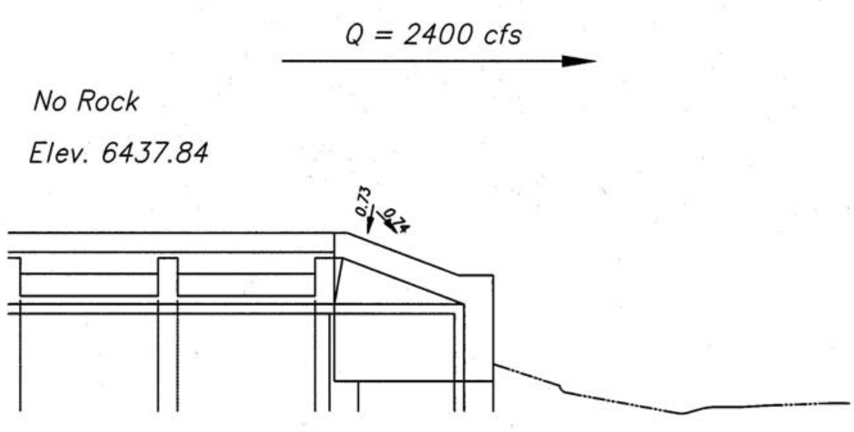


Figure 18. Velocities @ elev. 6437.84 downstream of DPP intake
Stream Q = 2400 ft^3/s, Diversion = 310 ft^3/s, no boulder

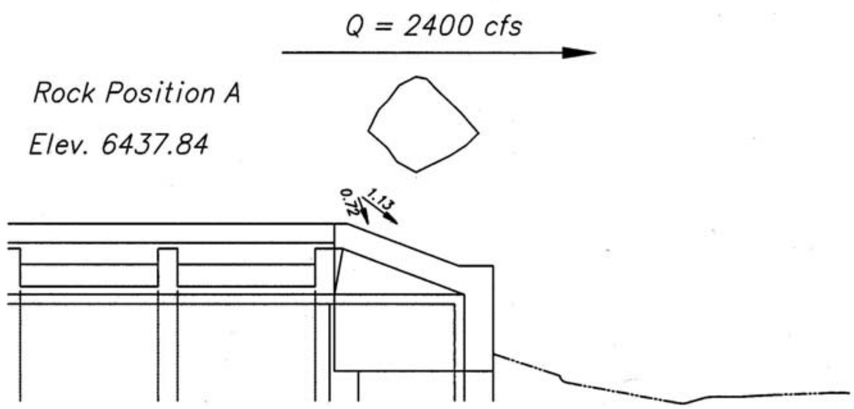


Figure 19. Velocities @ elev. 6437.84 downstream of DPP intake Stream Q = 2400 ft^3/s, Diversion = 310 ft^3/s, boulder position A

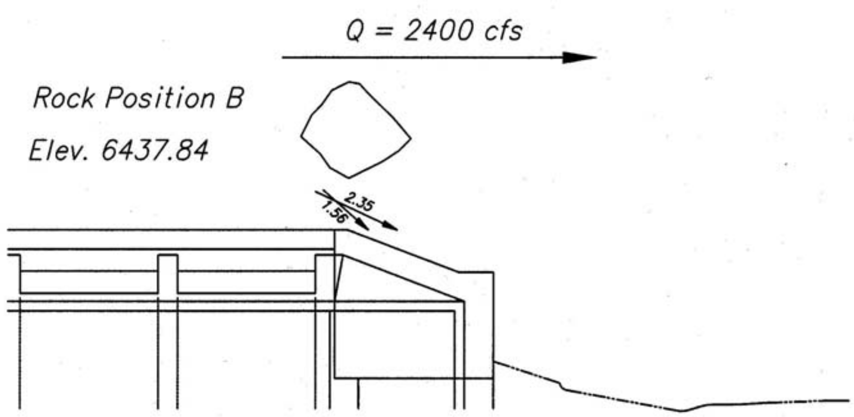


Figure 20. Velocities @ elev. 6437.84 downstream of DPP intake Stream Q = 2400 ft^3/s, Diversion = 310 ft^3/s, boulder position B

As indicated in Figures 18-20, an appreciable increase in local velocity was observed at the points of measurement with the boulder placed in the stream for a 2400 ft^3/s stream discharge. A more expansive modeling program may be warranted to accurately assess the potential of using a boulder placed near the intake to enhance sediment transport past the intake.

SUMMARY

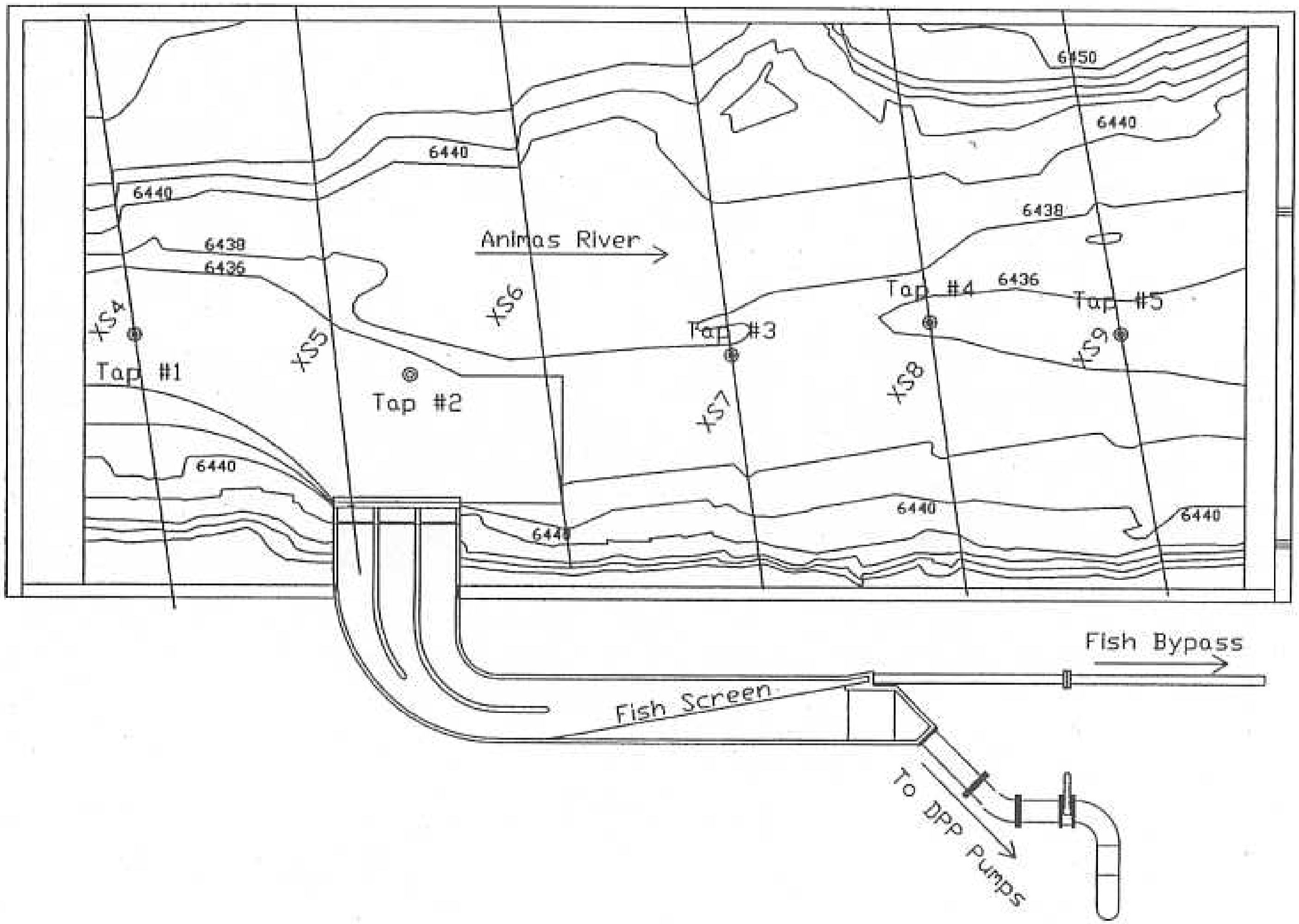
In the model testing that has been completed, investigations were carried out in the four focus areas identified at the outset of the project. Design modifications that improve intake capacity and distribution of flow through the fish screen are direct results of the modeling program being an integral part of the design process. Suitability of the design of the pump intake forebay was confirmed as a concurrent activity with tests focusing on other facets of the project. A body of information was compiled documenting impacts of diversion on stream flow and stream currents that potentially impact recreational uses of the Animas River. In an additional activity, a mechanism that may have potential for enhancement of sediment transport past the intake was studied in a limited-scope investigation.

Two modifications proposed late in the design process were tested to determine whether either would cause negative impact when compared with previous test results. Results of these tests, (see Figures 7 and 13) indicate no negative impacts on performance would result from the modifications. Due to time limits in the project schedule, it wasn't possible to repeat the full spectrum of tests with the late modifications. The similarity of data sets generated before and after the late modifications were incorporated indicates that information produced with the model as configured prior to these two changes represents model performance with the modifications with reasonable accuracy.

The 1:12 scale physical model has played an integral role in the "fast track" design process employed for the Durango Pumping Plant component of the Animas – La Plata project. It may have significant additional utility to the project should additional studies be undertaken to examine issues such as sediment accumulation in the river near the intake mouth and within the intake itself.

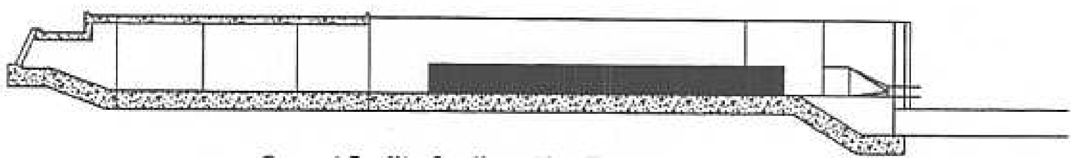
APPENDIX

DPP 1:12 PHYSICAL MODEL: PLAN SKETCH WITH REFERENCE SURVEY CROSS SECTIONS

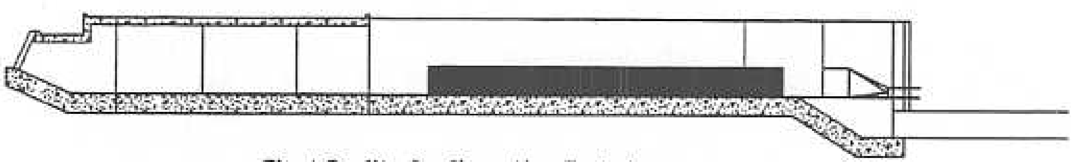


Centerline Profile of DPP Intake Structure: Tested Profile Configurations

First Profile Configuration Tested

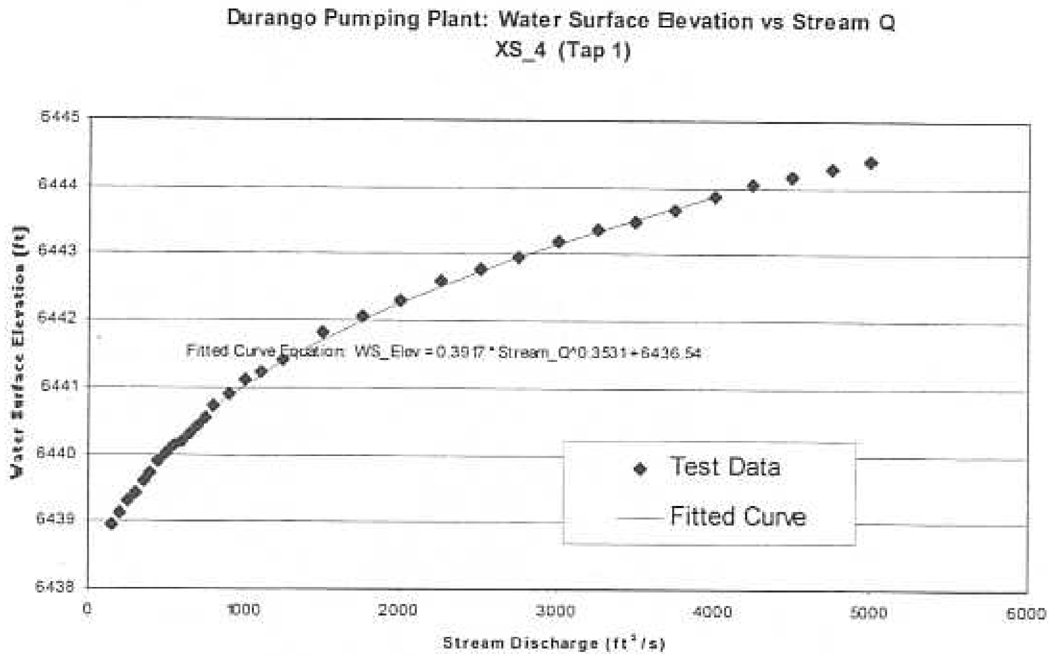


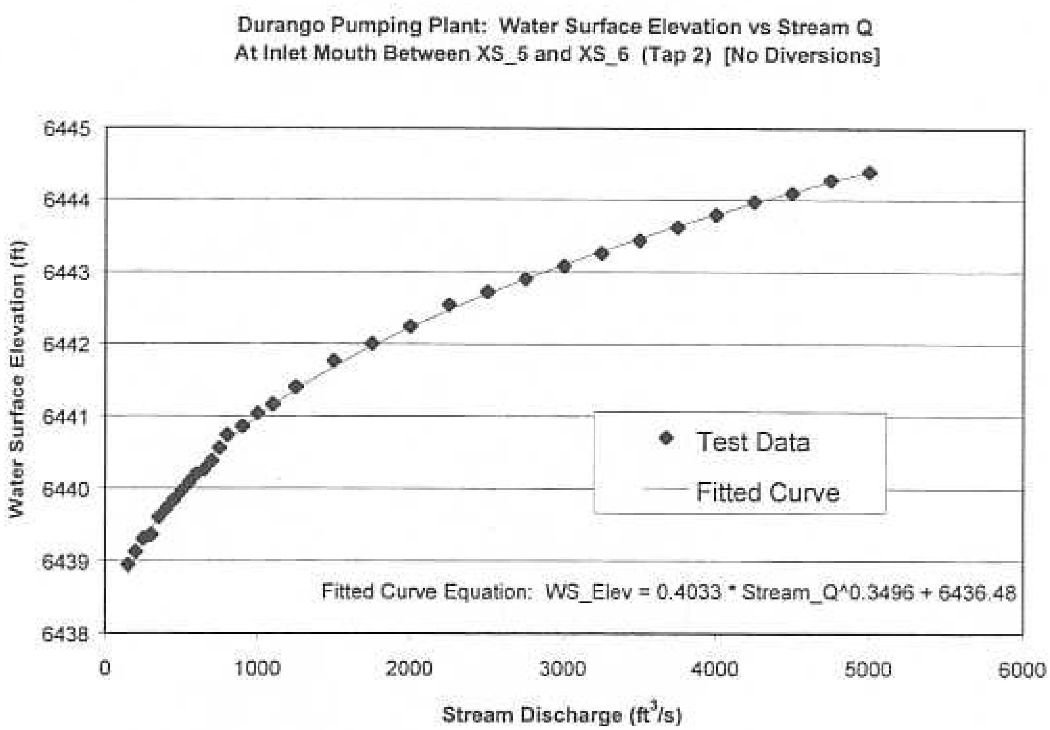
Second Profile Configuration Tested



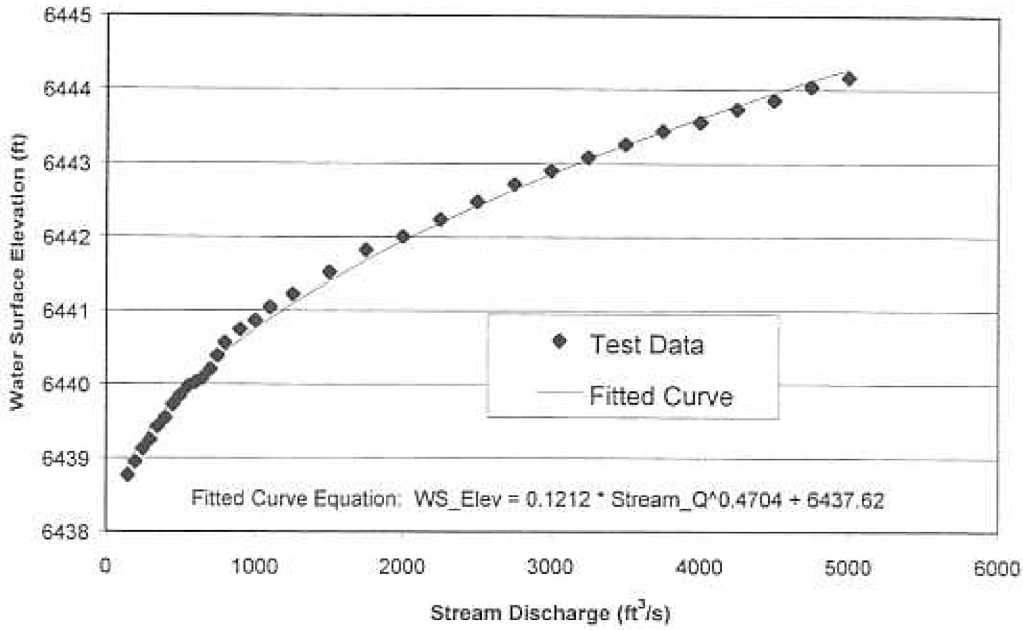
Final Profile Configuration Tested

Water Surface Elevation – Stream Discharge Curves: No Diversions

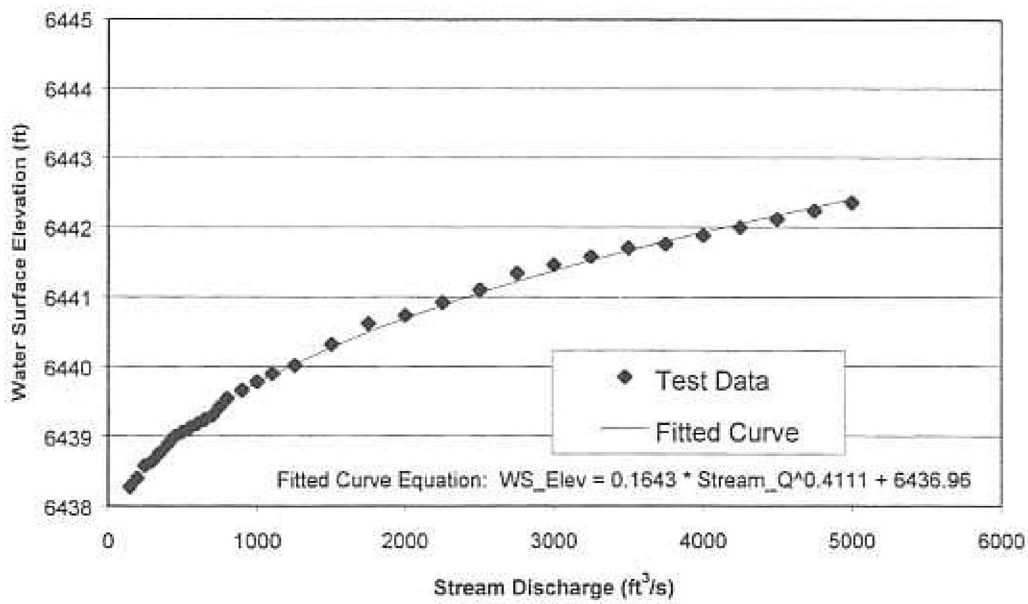




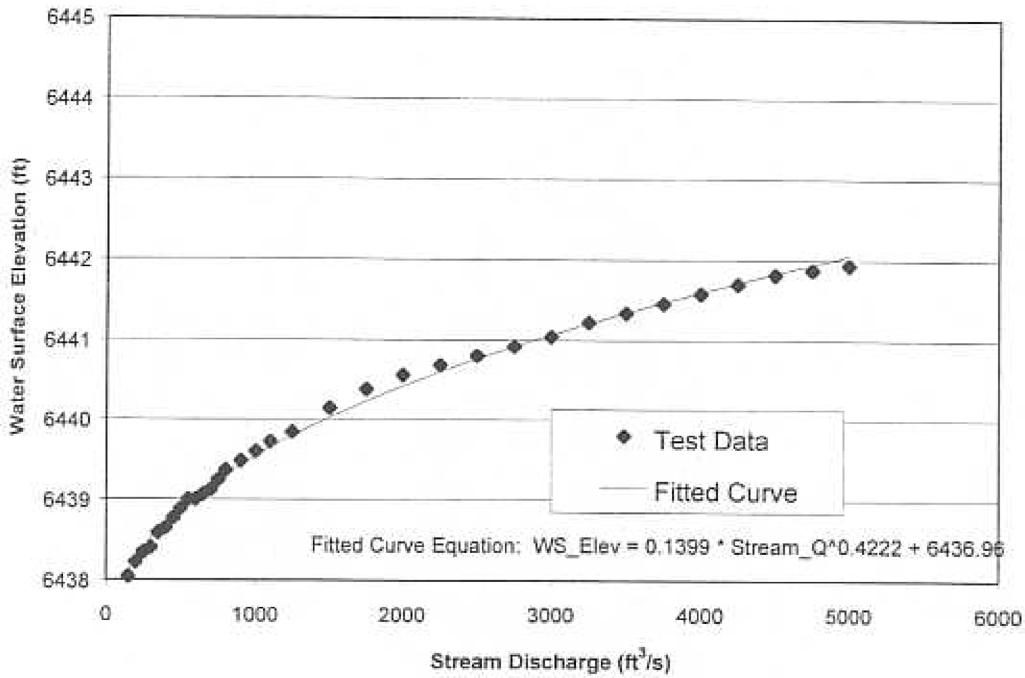
Durango Pumping Plant: Water Surface Elevation vs Stream Q
XS_7 (Tap 3)



Durango Pumping Plant: Water Surface Elevation vs Stream Q
XS_8 (Tap 4)



Durango Pumping Plant: Water Surface Elevation vs Stream Q
XS_9 (Tap 5)



PUMPING / NO PUMPING WATER SURFACE PROFILE COMPARISONS:

SUCTION INTAKE REYNOLDS NUMBER CALCULATIONS

Reference: Knauss, J. (Editor). 1987. Swirling Flow Problems at Intakes. IAHR/AIRH Hydraulic Structures Design Manual. A. A. Balkema. Rotterdam, Netherlands. pp 23 – 25.

For good modeling of submerged vortices, a limiting approach flow Reynolds number must be exceeded. The above reference notes that a Reynolds number of "about **3×10^4**" has been shown to be the limit in depressed sump tests.

A minimum Reynolds number of **2×10^5** is commonly cited as the lower limit for good modeling of surface vortices

Approach Reynolds Number: [1:12 Scale Model]

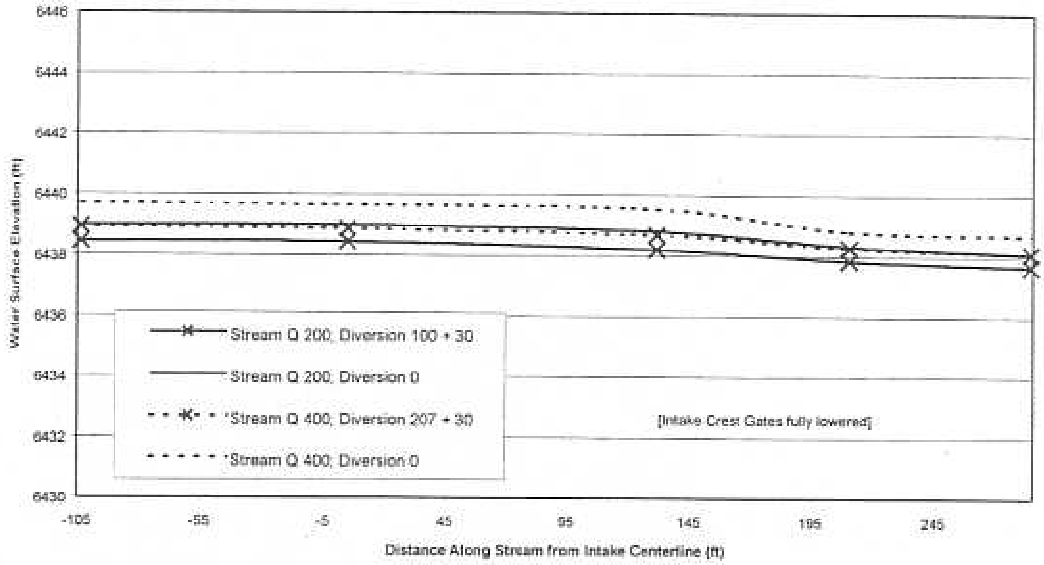
Reynolds Number: $Re = \frac{VD}{\nu}$

Where:
- Re = Reynolds Number
- V = Mean Velocity in Approach Channel (ft/s) $= Q/A$
  - Q = Discharge $= 280/12^{5/2} = 0.561$ (ft^3/s)
  - $A = W \times D$ (ft^2)
    - W = Channel Width = $19.5/12 = 1.625$ (ft)
- D = Flow Depth (ft)
- ν = Kinematic Viscosity $= 1.13 \times 10^{-5}$ (ft^2/s)

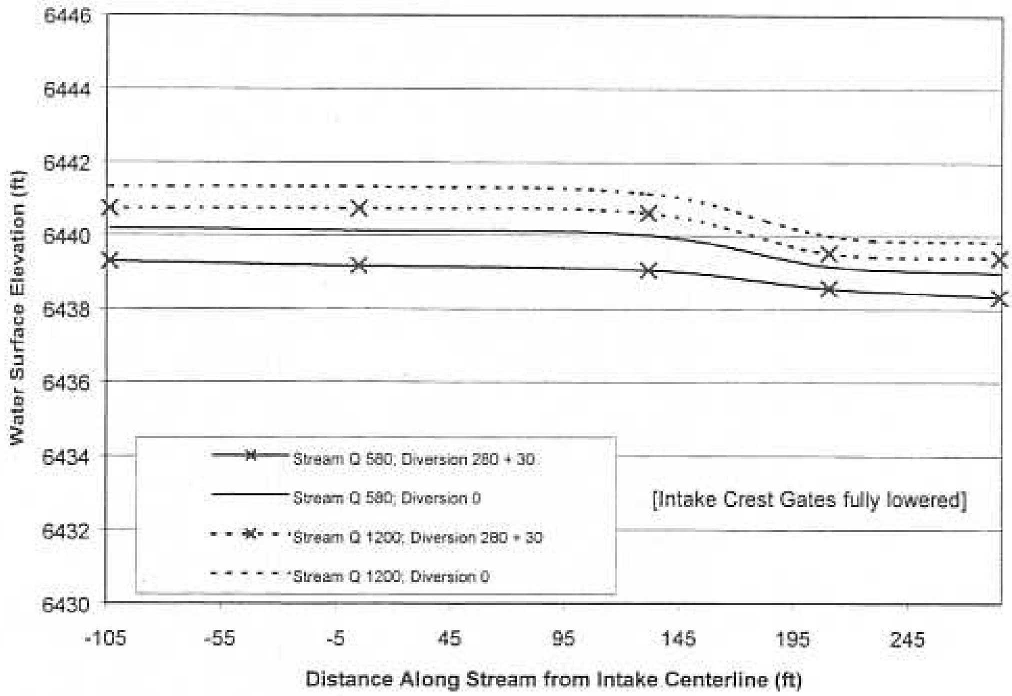
$$Re = \frac{Q * D}{W * D * \nu} = \frac{0.561}{(1.625)(1.13 \times 10^{-5})} = \mathbf{3.06 \times 10^4}$$

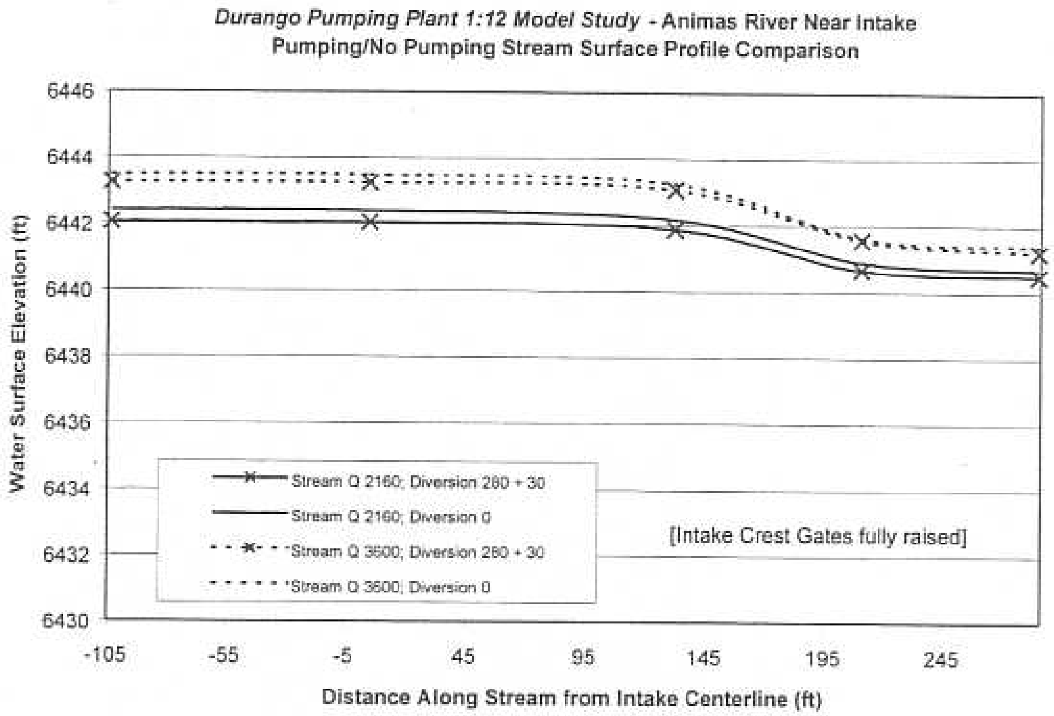
Summary of Approach Reynolds Number Analysis: The DPP model approach Reynolds Number is at the lower limit for good modeling of submerged vortices and is below the minimum value suggested for good modeling of surface vortices. Thus it is expected that formation of vortices will be amplified somewhat at prototype scale when compared with observations in the 1:12 scale model.

***Durango Pumping Plant 1:12 Model Study* - Animas River Near Intake: Pumping/No Pumping Stream Surface Profile Comparison**

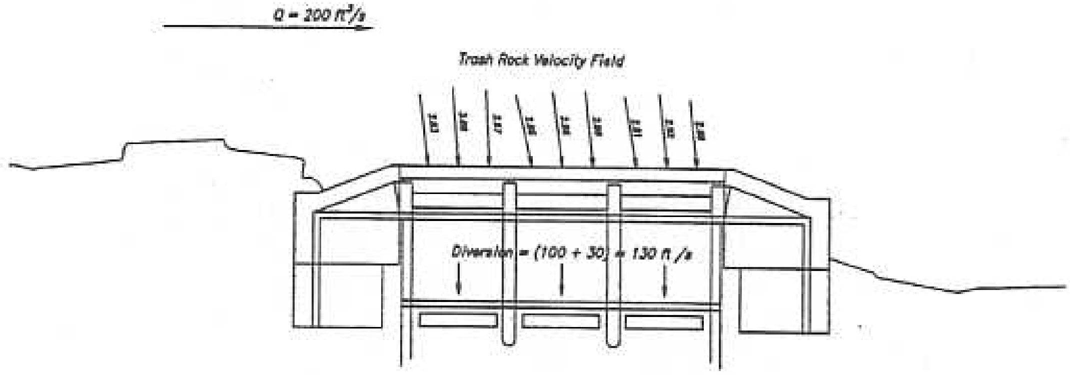


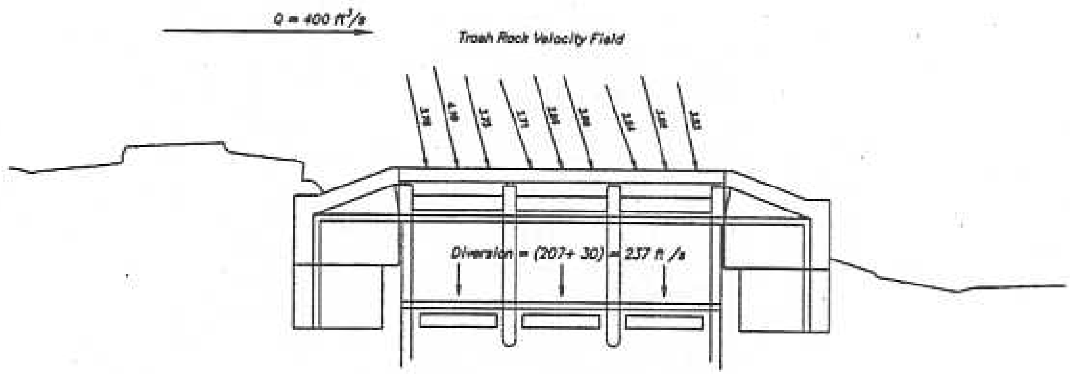
***Durango Pumping Plant 1:12 Model Study* - Animas River Near Intake: Pumping/No Pumping Stream Surface Profile Comparison**

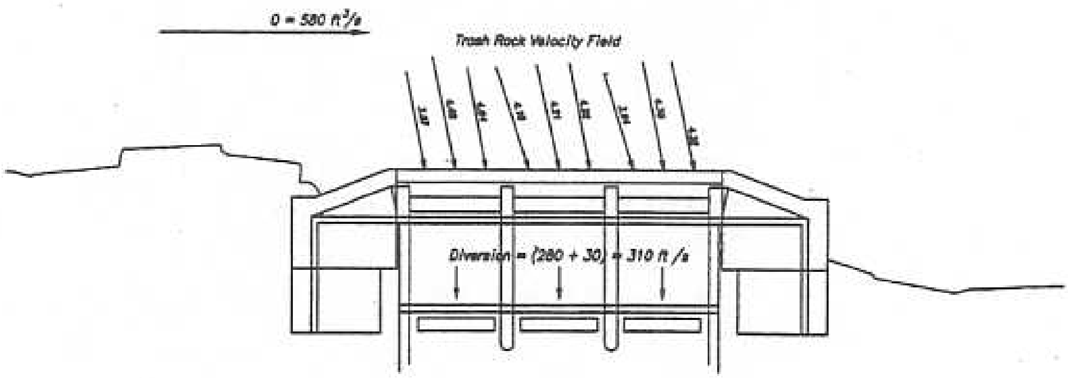


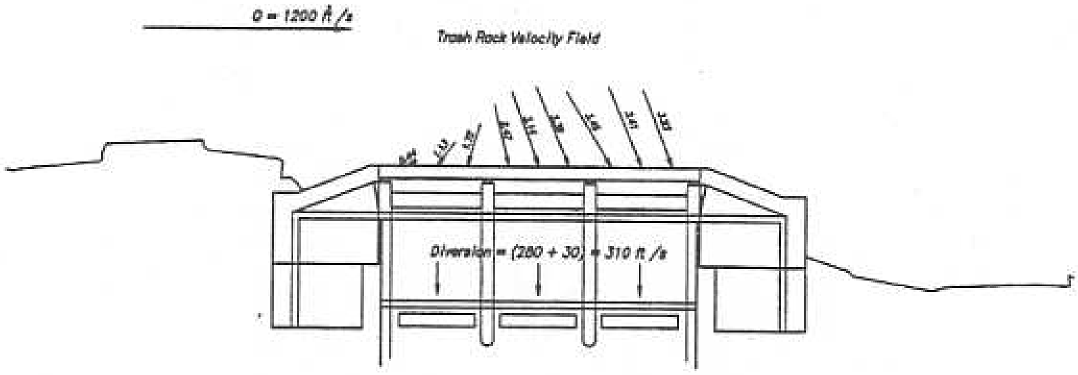


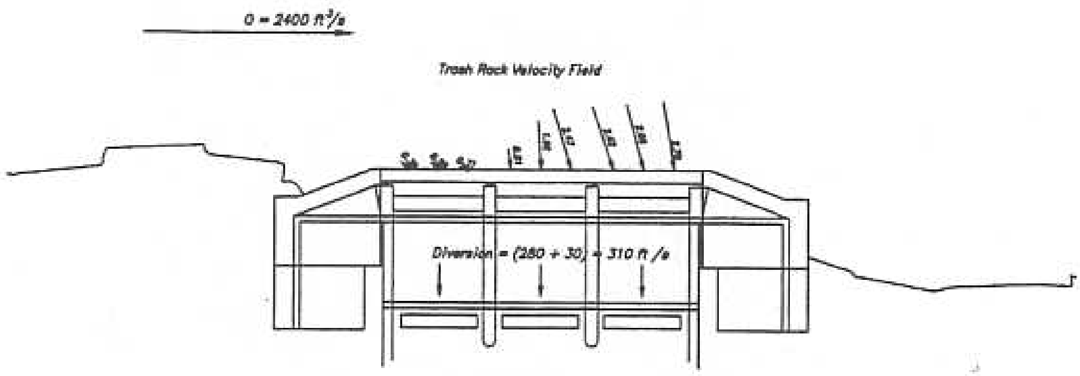
Near-Surface Velocities At DPP Trash Rack

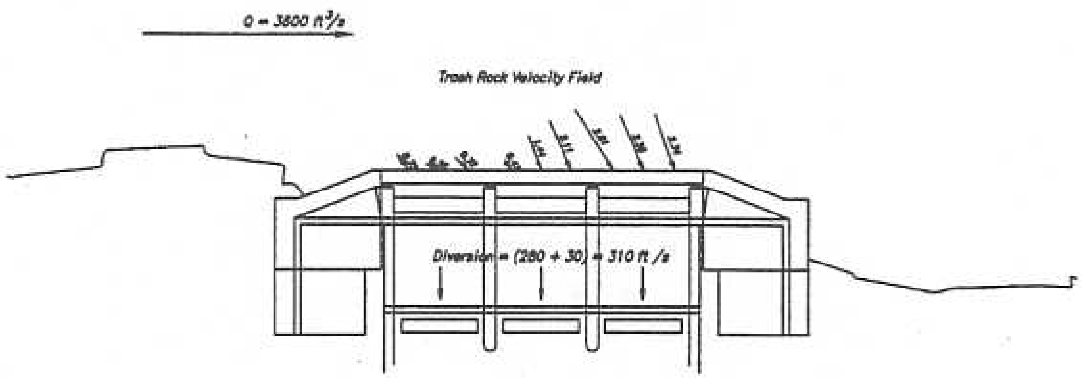

Q = 200 ft³/s
Trash Rack Velocity Field
Diversion = (100 + 30) = 130 ft /s


Q = 400 ft³/s
Trash Rack Velocity Field
Diversion = (207+ 30) = 237 ft /s


Q = 580 ft³/s
Trash Rack Velocity Field
Diversion = (280 + 30) = 310 ft /s

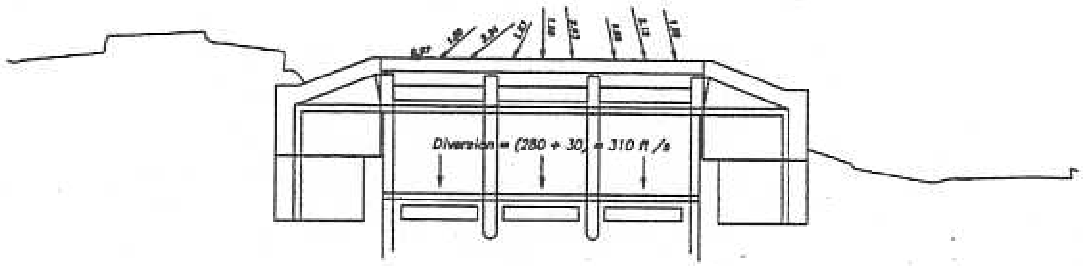

Q = 1200 ft³/s
Trash Rack Velocity Field
Diversion = (280 + 30) = 310 ft³/s


Q = 2400 ft³/s
Trash Rack Velocity Field
Diversion = (280 + 30) = 310 ft³/s


Q = 3600 ft³/s
Trash Rack Velocity Field
Diversion = (280 + 30) = 310 ft³/s

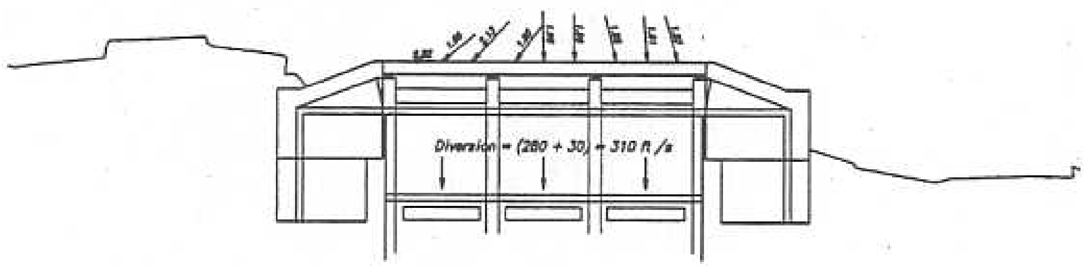
$Q = 2160 \text{ ft}^3/\text{s}$

Trash Rack Velocity Field – Crest Gates Up



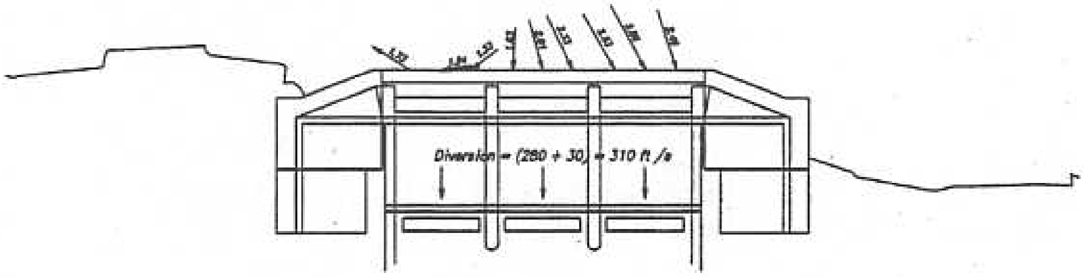
$Q = 2400 \text{ ft}^3/\text{s}$

Trash Rack Velocity Field – Crest Gates Up



$Q = 3600 \text{ ft}^3/\text{s}$

Trash Rack Velocity Field – Crest Gates Up



TECHNICAL SERVICE CENTER
DENVER, COLORADO

Durango Pumping Plant – Animas-La Plata Project
Expected Sediment Loading During Spring Runoff
COLORADO

Prepared by

Joseph K. Lyons
Hydraulic Engineer
Water Supply, Use, and Conservation Group

U.S. Department of the Interior
Bureau of Reclamation

May 2002

Durango Pumping Plant – Animas-La Plata Project
Expected Sediment Loading During Spring Runoff

BACKGROUND:

Sediment loading estimates for the Durango Pumping Plant (DPP) intake were initiated in the late 1980s and used during previous design activities for various configurations of the river intake and pumping plant designs. Recent changes in the project operations plan and the layout of the intake structure suggest that the previously calculated sedimentation load may not be representative of conditions that will exist when the DPP is operational.

The current design is markedly different in terms of intake size and expected diversion requirements than previous designs for the DPP river intake structure and intake channel. Included in this design is an Obermeyer gate. Located behind the river intake trash racks, the Obermeyer gate can be raised (by inflating an air-filled bladder) to reduce the chance of entraining bedload sediment into the intake structure.

METHOD:

Maximum seasonal sediment loading at the DPP are estimated using monthly flow and projected pumping requirements during the month of June. Also, the effects of intake channel hydraulics and the Obermeyer gate on suspended sediment loading at the DPP are estimated using an approach based upon theoretical distributions of suspended sediment particles in the vertical flow profile.

Daily estimates of Animas River flow and DPP pumping were obtained from the operation study data supplied by WCP-Durango. Median (1946) and Maximum (1957) monthly river flow conditions for June were selected based upon the 1929 - 1993 record for the Animas River at Durango gauge. DPP pumping rates for June 1946 conditions is 226 cfs and 240 cfs for June 1957 conditions.

Suspended sediment concentration was estimated from a sediment rating curve developed for measured suspended sediment data collected in the Animas River during the 1950s and 1970s. The maximum measured suspended sediment concentration is 816 mg/l from these data. An analysis of suspended sediment gradient data yielded sand-sized estimate of 56.5 percent of the total suspended load. On average, about 25.5 percent of the suspended sand load is between 0.0625 and 0.125 mm; about 18 percent is between 0.125 mm and 0.250 mm; about 9.4 percent is between 0.250 mm and 0.500 mm; about 2.9 percent is between 0.500 mm and 1.00 mm; and 0.7 percent is between 1.00 and 2.00 mm.

The effect of the proposed Obermeyer gate at the river intake structure was estimated by computing the concentration of suspended sands in the upper part of the flow relative to the measured concentration from depth-integrated samples. The approach used is that described in Report #3 of the Federal Interagency Sedimentation Project entitled Analytical Study of Methods of Sampling Suspended Sediments. The computations used in this analysis were originally prepared in the early 1990s for the DPP intake structure as proposed at that time. The current proposal is similar to the earlier proposed structure, including the presence and operation of the Obermeyer gate during spring runoff. It was assumed that the Obermeyer gate operated at an elevation 3 feet above the invert of the intake continuously during June and that sufficient river flow was available to maintain the constant diversions specified in the operations study.

RESULTS:

Intake Hydraulics

A review of estimated intake channel velocities for high flow conditions on the Animas River and implementation of the Obermeyer gate at the intake structure indicates that suspended sediments will stay in suspension upstream of the fish screens with minimal deposition. The estimated intake channel velocities provided by D-8140 were compared to Hjulstrom curves diagram for sediments less than 2.00 mm in diameter. The Hjulstrom curves diagram relates velocity and sediment size range and indicates zones of these parameters where sedimentation, transportation, and erosion of sediments will occur. Estimated mean intake channel velocity for high river flow conditions range from 0.47 to 0.97 feet per second. According to the relationships described in the Hjulstrom curves, particles less than 1.00 mm in diameter will tend to stay in suspension over this range of velocity.

It should be noted that the Hjulstrom curves diagram was prepared from data collected from flume studies using uniform-sized material. This limits the direct application of these relationships between sediment and velocity in mixed load conditions (such as the Animas River flow) but it appears that there is sufficient velocity within the intake channel to transport the anticipated sediment load from the Animas River.

Sediment Load Estimate without Obermeyer Gate

Sediment loading was estimated using daily Animas River flow and estimated suspended concentration based upon the daily flow for both median and maximum June monthly flow conditions. For the median June streamflow of 2,456 cfs (1946 conditions) and 226 cfs pumping rate, the monthly sand-sized sediment load carried into the DPP is about 704 cubic yards. For the historic maximum June streamflow of 5,382 cfs (1957 conditions) and 240 cfs pumping rate, the monthly sand-sized sediment load carried into the DPP is about 3,317 cubic yards. Sand-sized sediments constitute more than 56 percent of the anticipated sediment loading with silt and clay comprising the rest of the suspended load. Nearly half of the Animas River sand load is in the 0.0625 mm to 0.125 mm size range.

Sediment Load Estimate with Obermeyer Gate

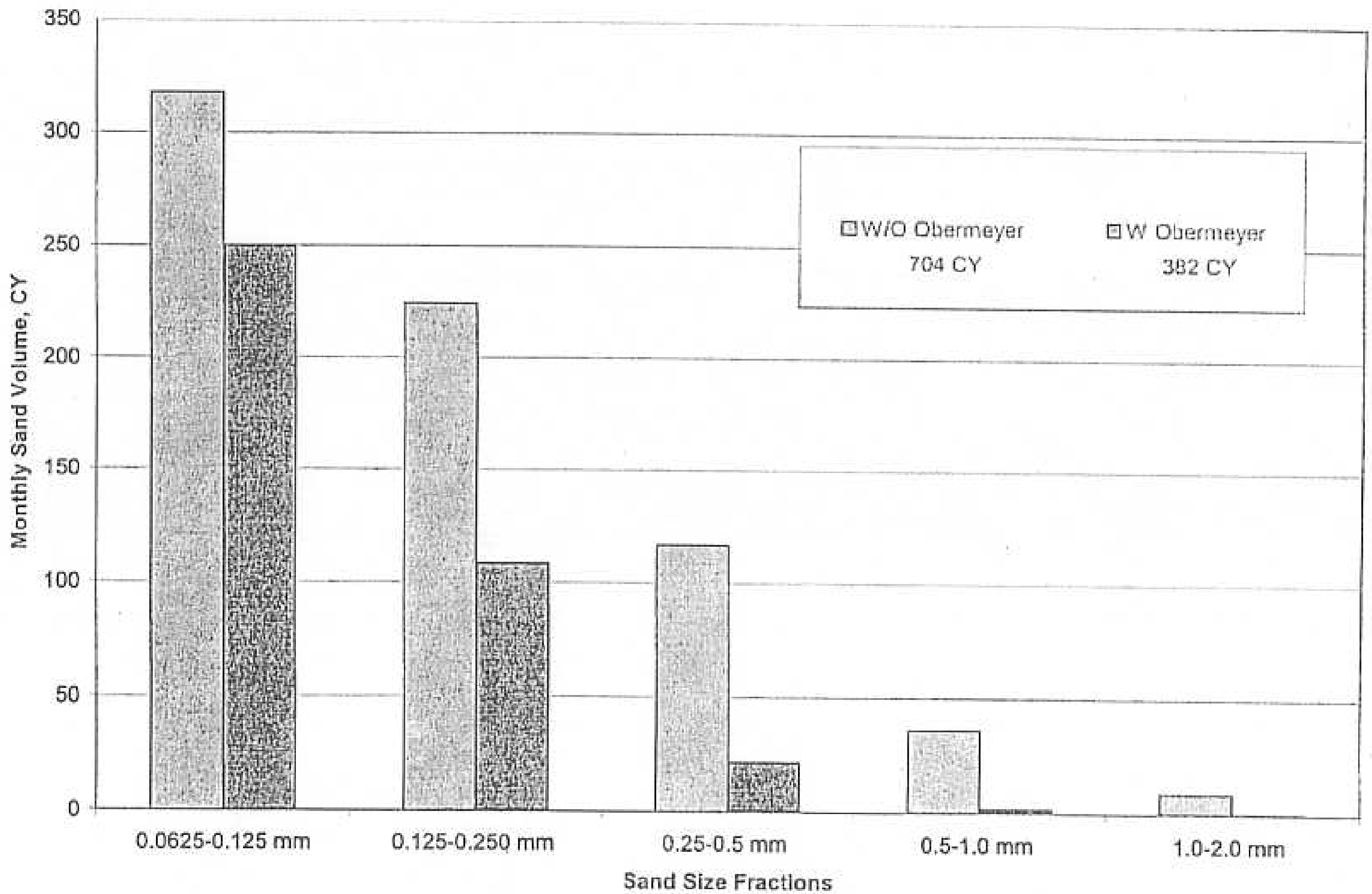
Sediment loading reduction for conditions with the Obermeyer gate in operation was estimated using daily Animas River flow, estimated suspended concentration based upon the daily flow for both median and maximum June monthly flow conditions and estimated reduction percentages based on particle size range data and flow depth. Theoretically, less sediment is transported in the upper portion of the flow column than what is represented in a sample taken from the entire flow column. As the Obermeyer gate will restrict diverted flow to the upper portion of the flow column, an approach to compute the sediment load carried in the upper part of the flow is necessary to approximate the diverted sediment load to the DPP with Obermeyer gates in place.

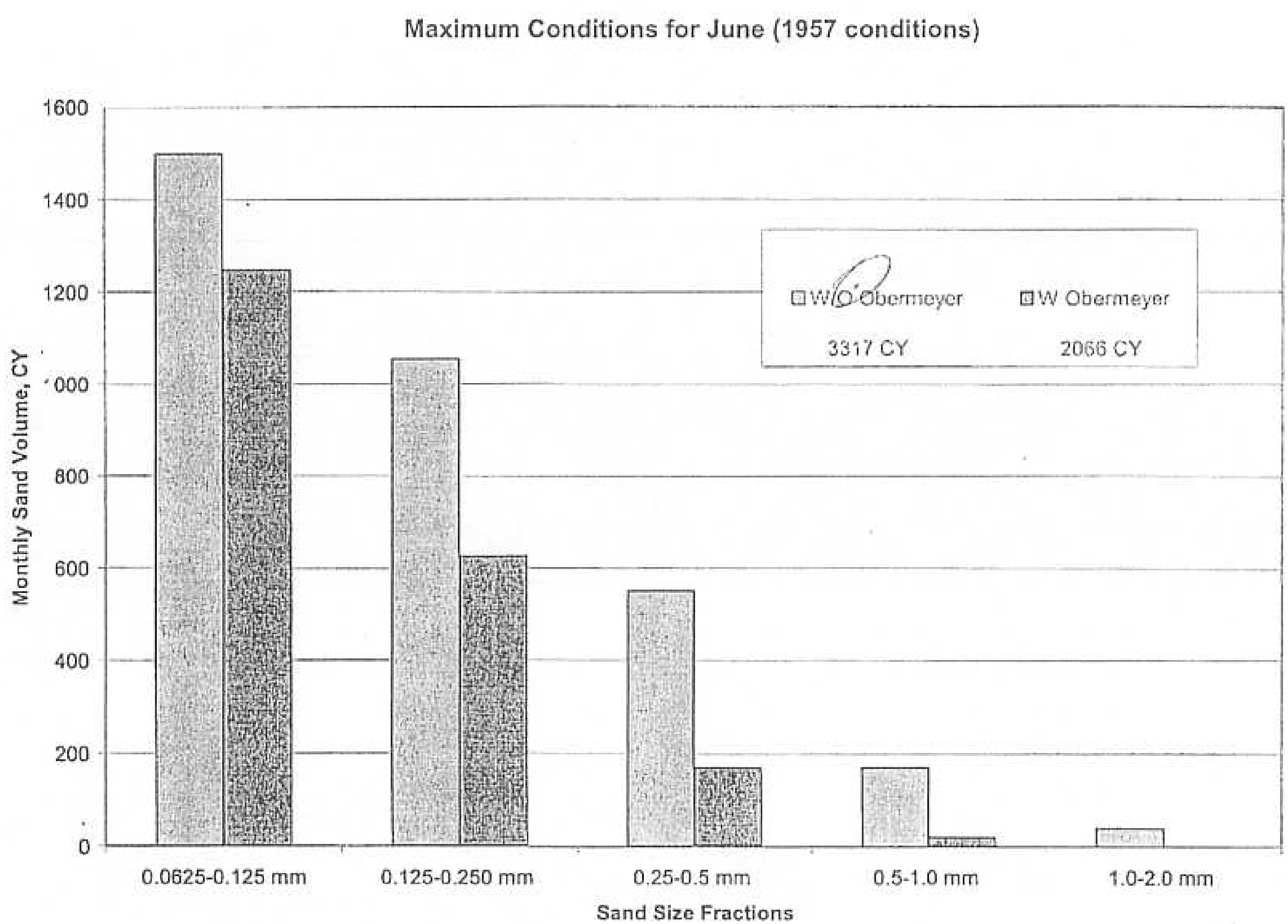
Using river stage data for the Animas River in the vicinity of the DPP intake, rating tables of percent sediment load reduction for the sand-sized fraction for river discharges ranging from 500 cfs to 10,000 cfs were estimated. Generally, as river discharge increases, the reduction percentage for sediment decreases within a given size range. Larger sands have a greater reduction percentage than finer sand particles such that less than 10 percent of the depth-integrated sampled sand in the 1.00 mm to 2.00 mm size range is found in the upper portion of the flow column. Conversely, the computed reduction percentage for fine sand ranges from 10 to about 25 percent over the 500 to 10,000 cfs range.

For median monthly flow conditions for June (1946 conditions) the projected reduction in sand load into DPP is 322 cubic yards (from 704 cubic yards without the Obermeyer gate to 382 cubic yards with the Obermeyer gate raised). This represents a 46 percent reduction in sand load with elimination of essentially all of the sand load greater than 1 mm in diameter.

For maximum monthly flow conditions for June (1957 conditions) the projected reduction in sand load into DPP is 1,251 cubic yards (3,317 cubic yards without the Obermeyer gate to 2,066 cubic yards with the Obermeyer gate raised). This represents a 38 percent reduction in sand load with elimination of essentially all of the sand load greater than 1 mm in diameter.

Median Conditions for June (1946 conditions)




Maximum Conditions for June (1957 conditions)
Monthly Sand Volume, CY
1600
1400
1200
000
800
600
400
200
0
W/O Obermeyer
3317 CY
W Obermeyer
2066 CY
0.0625-0.125 mm
0.125-0.250 mm
0.25-0.5 mm
0.5-1.0 mm
1.0-2.0 mm
Sand Size Fractions

Reductions by size fraction and discharge
Animas River at Durango

Size:0.0625-0.125		Size:0.125-0.250		Size:0.250-0.50		Size:0.50-1.00		Size:1.00-2.00	
Discharge	% of Original	Discharge	% of Original	Discharge	% of Original	Discharge	% of Original	Discharge	% of Original
500	0.74	500	0.37	500	0.09	500	0.01	500	0.00
1000	0.76	1000	0.41	1000	0.11	1000	0.03	1000	0.00
2000	0.78	2000	0.46	2000	0.16	2000	0.04	2000	0.01
3000	0.79	3000	0.50	3000	0.20	3000	0.06	3000	0.01
4100	0.81	4100	0.55	4100	0.25	4100	0.08	4100	0.02
5000	0.82	5000	0.58	5000	0.29	5000	0.10	5000	0.03
6000	0.84	6000	0.60	6000	0.32	6000	0.13	6000	0.04
7000	0.85	7000	0.63	7000	0.36	7000	0.16	7000	0.04
8000	0.87	8000	0.66	8000	0.40	8000	0.19	8000	0.05
9000	0.88	9000	0.69	9000	0.44	9000	0.22	9000	0.06
10000	0.9	10000	0.72	10000	0.48	10000	0.25	10000	0.07

Year	Month Ranked	Avg. monthly flow in cfs
1934	6	394.0
1977	6	716.7
1963	6	946.2
1940	6	1,178.5
1954	6	1,243.1
1974	6	1,255.2
1989	6	1,347.9
1939	6	1,365.8
1967	6	1,404.5
1931	6	1,419.3
1964	6	1,436.1
1936	6	1,619.6
1959	6	1,632.4
1972	6	1,685.6
1951	6	1,764.1
1981	6	1,783.7
1956	6	1,812.4
1966	6	1,822.7
1950	6	1,896.6
1937	6	1,902.3
1961	6	2,066.0
1955	6	2,094.9
1992	6	2,095.1
1988	6	2,149.3
1970	6	2,152.3
1990	6	2,212.7
1943	6	2,216.0
1991	6	2,217.5
1976	6	2,273.0
1969	6	2,316.3
1971	6	2,350.7
1953	6	2,409.9
1948	6	2,456.4
1930	6	2,553.6
1962	6	2,677.8
1945	6	2,781.2
1947	6	2,899.8
1933	6	2,921.5
1960	6	2,942.3
1984	6	3,140.6
1982	6	3,232.5
1932	6	3,473.2
1929	6	3,506.0
1978	6	3,571.9
1958	6	3,634.0
1946	6	3,745.2
1986	6	3,762.8
1968	6	3,835.6
1942	6	3,887.8
1987	6	3,907.5
1993	6	3,958.9
1935	6	3,983.9
1965	6	4,027.9
1983	6	4,185.1
1938	6	4,234.7
1985	6	4,363.4
1975	6	4,682.9
1944	6	4,713.1
1979	6	4,756.8
1973	6	4,796.6
1941	6	4,911.8
1949	6	4,999.4
1980	6	5,156.7
1952	6	5,215.1
1957	6	5,381.7

Percentile	Flow
10	1355.1
20	1874.9
30	1935.1
40	2214.7
50	2456.4
60	3021.6
70	3723.0
80	3992.7
90	4739.3

min	394.0
max	5,381.7
avg	2,823.0

Median June 1948

Date	Animas Q, CFS	DPP Q, CFS	Conc., mg/	Qs to DPP, T/day
06/01/48	759.57	226.15	13	8.0
06/02/48	767.92	226.15	13	8.1
06/03/48	1028.76	226.15	19	11.5
06/04/48	1794.59	226.15	43	26.1
06/05/48	3087.49	226.15	112	68.4
06/06/48	3630.9	226.15	156	95.5
06/07/48	3818.71	226.15	173	105.7
06/08/48	3891.74	226.15	180	109.9
06/09/48	3891.74	226.15	180	109.9
06/10/48	3537	226.15	148	90.6
06/11/48	3943.91	226.15	185	112.9
06/12/48	3923.05	226.15	183	111.7
06/13/48	3745.68	226.15	166	101.6
06/14/48	3537	226.15	148	90.6
06/15/48	3359.63	226.15	134	81.8
06/16/48	3359.63	226.15	134	81.8
06/17/48	3328.33	226.15	131	80.3
06/18/48	3077.93	226.15	113	68.9
06/19/48	2222.37	226.15	62	37.7
06/20/48	2034.56	226.15	53	32.3
06/21/48	1930.22	226.15	48	29.5
06/22/48	1690.25	226.15	39	23.7
06/23/48	1711.12	226.15	40	24.2
06/24/48	1617.21	226.15	36	22.1
06/25/48	1492.01	226.15	32	19.5
06/26/48	1481.58	226.15	32	19.3
06/27/48	1408.54	226.15	29	17.8
06/28/48	1314.63	226.15	26	16.1
06/29/48	1199.87	226.15	23	14.1
06/30/48	1126.84	226.15	21	13.0
				1632.2 Total tons sediment per month
	2458.426			

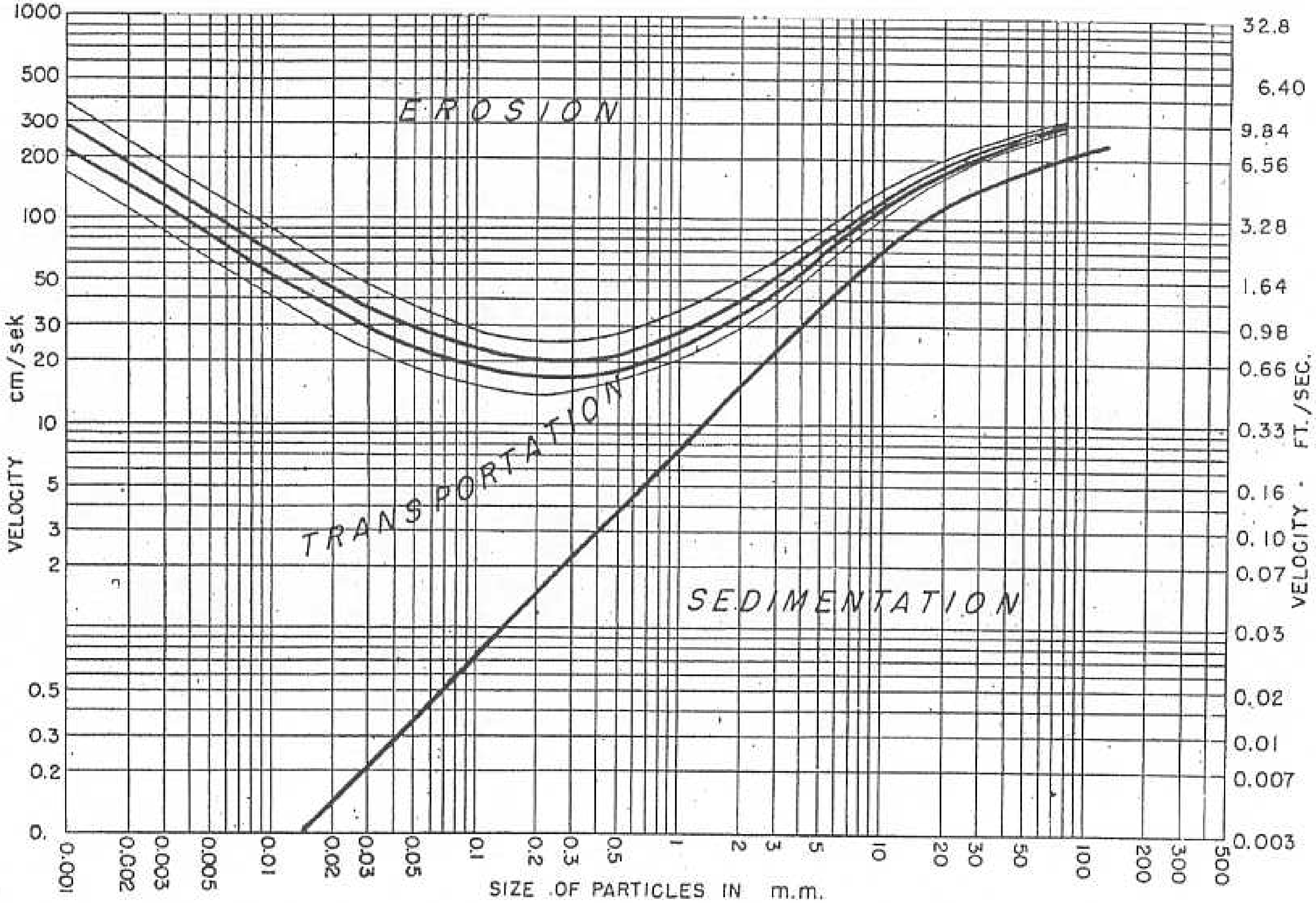
0.1	clay, AF	232 CY
0.3	silt, AF	530 CY
0.4	sand, AF	704 CY
0.9	AF sediment per month	1-

Max June 1957

Date	Animas Q, CFS	DPP Q, CFS	Conc., mg/	Qs to DPP, T/day
06/01/57	3244.1	240	125	81.0
06/02/57	4049.91	240	195	120.5
06/03/57	4949.87	240	297	192.6
06/04/57	6205.66	240	486	314.8
06/05/57	7818.41	240	816	528.8
06/06/57	8466.07	240	816	528.8
06/07/57	7084.72	240	816	528.8
06/08/57	6645.18	240	565	366.4
06/09/57	6226.58	240	489	317.2
06/10/57	6645.18	240	565	366.4
06/11/57	4781.51	240	274	177.4
06/12/57	3463.87	240	142	92.2
06/13/57	3965.72	240	186	120.5
06/14/57	4383.85	240	228	147.7
06/15/57	4112.69	240	201	130.6
06/16/57	3275.5	240	127	82.5
06/17/57	2553.43	240	79	51.4
06/18/57	2846.44	240	97	62.9
06/19/57	4206.87	240	211	136.8
06/20/57	5556.83	240	381	247.1
06/21/57	6700.29	240	588	380.8
06/22/57	5808	240	420	272.2
06/23/57	5232.43	240	335	217.0
06/24/57	5556.83	240	381	247.1
06/25/57	5975.43	240	447	289.7
06/26/57	6540.54	240	546	353.7
06/27/57	6268.45	240	497	321.9
06/28/57	6645.18	240	565	366.4
06/29/57	6707.97	240	577	374.2
06/30/57	5724.27	240	407	263.7
				7687.0 Total tons sediment per month

5381.7

0.7	clay, AF	1091 CY
1.5	silt, AF	2496 CY
2.1	sand, AF	3317 CY
4.3	AF sediment per month	6904 CY



THE CURVES FOR EROSION AND DEPOSITION OF A UNIFORM MATERIAL

FROM F. HJULSTRÖM BULLETIN GEOL. INSTITUTE OF UPSALA VOL.XXX

This sheet calculates the load reduction due to the Obermeyer gate
Reduction is based upon procedure in FISP report #3, Nov. 1941, Section II part III p.21
Report #3 is entitled Analytical Study of Methods of Sampling Suspended Sediment

Median June 1948

Date	Animas Q, CFS	DPP Q, CFS	Conc., mg/l	Qs to DPP, T/day
06/01/48	759.57	228.15	13.1	8.0
06/02/48	787.92	228.15	13.2	8.1
06/03/48	1029.74	228.15	18.8	11.6
06/04/48	1794.89	228.15	42.8	26.1
06/05/48	3067.49	228.15	112.1	68.4
06/06/48	3530.9	228.15	156.2	95.3
06/07/48	3810.71	228.15	173.1	105.7
06/08/48	3891.74	228.15	178.6	108.9
06/09/48	3891.74	228.15	178.6	108.9
06/10/48	3537	228.15	148.3	90.6
06/11/48	3943.81	228.15	184.9	112.9
06/12/48	3923.08	228.15	182.9	111.7
06/13/48	3745.84	228.15	166.5	101.6
06/14/48	3537	228.15	148.3	90.6
06/15/48	3359.63	228.15	133.9	81.8
06/16/48	3359.63	228.15	133.9	81.8
06/17/48	3328.33	228.15	131.6	80.3
06/18/48	3077.83	228.15	112.8	68.9
06/19/48	2222.37	228.15	61.7	37.7
06/20/48	2034.64	228.15	52.9	32.3
06/21/48	1930.23	228.15	48.1	29.5
06/22/48	1890.25	228.15	38.8	23.7
06/23/48	1711.12	228.15	39.6	24.2
06/24/48	1657.21	228.15	36.2	22.1
06/25/48	1492.01	228.15	31.8	19.5
06/26/48	1481.58	228.15	31.8	19.5
06/27/48	1438.54	228.15	29.2	17.8
06/28/48	1314.89	228.15	26.4	16.1
06/29/48	1198.87	228.15	23.2	14.1
06/30/48	1125.84	228.15	21.2	13.0

Sandload by size fraction, Tons per day

0.0625-0.125 mm	0.125-0.250 mm	0.25-0.5 mm	0.5-1.0 mm	1.0-2.0 mm	
2.0	1.4	0.8	0.2		
2.1	1.5	0.8	0.2		
2.8	2.1	1.1	0.3		
6.7	4.7	2.5	0.8		
17.5	12.3	6.4	2.0		
24.3	17.2	9.0	2.8		
27.0	19.0	9.9	3.1		
28.0	19.8	10.3	3.2		
28.0	19.8	10.3	3.2		
23.1	16.3	8.5	2.6		
28.8	20.3	10.6	3.3		
28.5	20.1	10.5	3.2		
25.9	18.3	9.6	2.9		
23.1	16.3	8.5	2.6		
20.8	14.7	7.7	2.4		
20.8	14.7	7.7	2.4		
20.5	14.4	7.5	2.3		
17.6	12.4	6.5	2.0		
9.6	6.8	3.5	1.1		
8.2	5.8	3.0	0.9		
7.5	5.3	2.8	0.9		
6.0	4.3	2.2	0.7		
6.2	4.4	2.3	0.7		
5.6	4.0	2.1	0.6		
5.0	3.5	1.8	0.6		
4.9	3.5	1.8	0.6		
4.5	3.2	1.7	0.5		
4.1	2.9	1.5	0.5		
3.6	2.6	1.3	0.4		
3.3	2.3	1.2	0.4		
418.2	293.8	153.4	47.3		912.7 Sandload, T/month
0.20	0.14	0.07	0.02		0.4 AF sand per month
318	224	117	36		704 CY of sand per month

Obermeyer reduction
Sandload by size fraction

0.0625-0.125 mm (.253)	0.125-0.250mm (.18)	0.25-0.5 mm (.094)	0.5-1.0 mm (.029)	1.0-2.0mm (.007)	
1.5	0.5	0.1	0.0	0.0	
1.6	0.5	0.1	0.0	0.0	
2.2	0.9	0.1	0.0	0.0	
6.1	1.9	0.3	0.0	0.0	
13.8	5.2	1.3	0.1	0.0	
19.3	8.5	1.8	0.2	0.0	
21.4	9.5	2.0	0.2	0.0	
22.2	9.8	2.1	0.2	0.0	
22.2	9.8	2.1	0.2	0.0	
18.3	8.2	1.7	0.2	0.0	
22.8	10.2	2.1	0.2	0.0	
22.6	10.1	2.1	0.2	0.0	
20.5	9.2	1.9	0.2	0.0	
18.3	8.2	1.7	0.2	0.0	
16.5	7.4	1.5	0.1	0.0	
16.5	7.4	1.5	0.1	0.0	
16.2	7.2	1.5	0.1	0.0	
13.9	6.2	1.3	0.1	0.0	
7.6	3.1	0.5	0.0	0.0	
6.4	2.6	0.5	0.0	0.0	
5.7	2.2	0.3	0.0	0.0	
4.6	1.7	0.2	0.0	0.0	
4.7	1.8	0.3	0.0	0.0	
4.3	1.8	0.2	0.0	0.0	
3.9	1.4	0.2	0.0	0.0	
3.7	1.4	0.2	0.0	0.0	
3.3	1.3	0.2	0.0	0.0	
3.1	1.2	0.2	0.0	0.0	
2.7	1.0	0.1	0.0	0.0	
2.5	1.0	0.1	0.0	0.0	
327.3	142.2	28.5	2.5	0.1	500.4 Sandload, T/month
0.15	0.07	0.01	0.00	0.00	0.2 AF sand per month
250	109	22	2	0	382 CY of sand per month

Max June 1957

Date	Animas Q, CFS	DPP Q, CFS	Conc., mg/l	Qs to DPP, T/day
06/01/57	3244.1	240	125.0	81.0
06/02/57	4069.91	240	185.2	120.5
06/03/57	4949.87	240	297.3	192.8
06/04/57	6206.88	240	483.8	314.8
06/05/57	7818.41	240	818.0	529.8
06/06/57	8468.07	240	818.0	529.8
06/07/57	7064.72	240	818.0	529.8
06/08/57	6643.18	240	585.5	380.4
06/09/57	6226.68	240	480.4	311.2
06/10/57	6643.18	240	585.5	380.4
06/11/57	4781.31	240	273.7	177.4
06/12/57	3459.87	240	143.3	92.2
06/13/57	3853.72	240	185.0	120.3
06/14/57	4363.86	240	227.9	147.7
06/15/57	4112.99	240	201.8	130.8
06/16/57	3275.3	240	127.4	82.6
06/17/57	2623.43	240	79.3	51.4
06/18/57	2846.44	240	97.1	62.9
06/19/57	4208.87	240	211.2	136.8
06/20/57	5454.83	240	381.4	247.1
06/21/57	6780.28	240	587.8	380.8
06/22/57	5808	240	420.0	272.2
06/23/57	5232.43	240	334.9	217.0
06/24/57	5506.83	240	381.4	247.1
06/25/57	5815.43	240	447.0	289.7
06/26/57	6540.54	240	545.8	353.7
06/27/57	6268.45	240	496.7	321.9
06/28/57	6643.18	240	565.5	368.4
06/29/57	6107.87	240	577.5	374.2
06/30/57	5724.37	240	406.8	263.7

Sandload by size fraction, Tons per day

0.0625-0.125 mm (.250)	0.125-0.250mm (.18)	0.25-0.5 mm (.094)	0.5-1.0 mm (.029)	1.0-2.0mm (.007)	
20.7	14.6	7.8	2.3	0.8	
32.3	22.8	11.9	3.7	0.9	
48.1	34.7	18.1	5.6	1.3	
80.3	56.7	29.6	9.1	2.2	
134.9	95.2	49.7	15.3	3.7	
134.9	95.2	49.7	15.3	3.7	
134.9	95.2	49.7	15.3	3.7	
93.4	66.0	34.4	10.6	2.6	
80.9	57.1	29.8	9.2	2.2	
93.4	66.0	34.4	10.6	2.6	
45.2	31.9	16.7	5.1	1.3	
23.5	16.6	8.7	2.7	0.6	
30.7	21.7	11.3	3.5	0.8	
37.7	26.6	13.9	4.3	1.0	
34.3	23.5	12.3	3.8	0.9	
21.0	14.8	7.8	2.4	0.6	
13.1	9.2	4.8	1.5	0.4	
16.0	11.3	5.9	1.8	0.4	
34.9	24.6	12.9	4.0	1.0	
63.0	44.5	23.2	7.2	1.7	
97.1	68.5	35.8	11.0	2.7	
69.4	49.0	25.6	7.9	1.9	
55.3	39.1	20.4	6.3	1.5	
63.0	44.5	23.3	7.2	1.7	
73.9	52.1	27.2	8.4	2.0	
90.2	63.7	33.2	10.3	2.5	
82.1	57.9	30.3	9.3	2.3	
93.4	66.0	34.4	10.6	2.6	
95.4	67.4	35.2	10.9	2.6	
67.2	47.5	24.8	7.6	1.8	
1960.3	1383.7	722.8	222.9	53.8	4343.2 Sandload, T/month
0.93	0.65	0.34	0.11	0.03	2.1 AF sand per month
1497	1057	552	170	41	3317 CY of sand per month

Obermeyer reduction
Sandload by size fraction

0.0625-0.125 mm (.253)	0.125-0.250mm (.18)	0.25-0.5 mm (.094)	0.5-1.0 mm (.029)	1.0-2.0mm (.007)	
18.4	7.3	1.5	0.1	0.0	
26.8	11.4	2.4	0.2	0.0	
39.8	18.1	4.5	0.4	0.0	
67.4	34.3	8.6	1.2	0.1	
113.2	60.3	15.0	2.5	0.2	
117.2	63.0	20.2	2.8	0.2	
115.2	60.3	19.3	2.5	0.2	
78.4	32.8	11.2	1.4	0.1	
67.9	34.5	9.7	1.2	0.1	
78.4	30.8	11.2	1.4	0.1	
36.8	17.8	4.2	0.4	0.0	
18.6	8.3	1.7	0.2	0.0	
24.4	10.9	2.3	0.2	0.0	
30.5	14.5	3.5	0.3	0.0	
27.0	12.9	3.1	0.3	0.0	
16.7	7.4	1.8	0.1	0.0	
10.2	4.2	0.7	0.1	0.0	
12.8	5.2	0.9	0.1	0.0	
28.3	13.5	3.2	0.3	0.0	
51.9	25.6	6.6	0.7	0.0	
81.5	41.4	11.8	1.5	0.1	
57.2	28.3	7.3	0.8	0.1	
45.8	22.5	5.8	0.8	0.0	
51.9	25.6	6.6	0.7	0.0	
60.8	30.0	7.8	0.9	0.1	
75.7	38.5	10.8	1.2	0.1	
68.9	35.0	9.8	1.2	0.1	
78.4	39.8	11.2	1.4	0.1	
80.1	40.7	11.4	1.4	0.1	
55.4	21.3	7.1	0.8	0.1	
1633.2	819.3	323.2	37.2	1.8	2704.8 Sandload, T/month
0.77	0.39	0.15	0.01	0.00	1.3 AF sand per month
1247	626	170	31	1	2066 CY of sand per month

Prepared by JLYONS 4/10/02

TECHNICAL SERVICE CENTER
Denver, Colorado

Durango Pumping Plant -- Animas-La Plata Project
Expected Sediment Loading During Spring Runoff

Supplemental Sediment Deposition Analysis
For the Proposed Intake Channel Design

Prepared by

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U.S. Department of the Interior
Bureau of Reclamation

September 2002

Durango Pumping Plant -- Animas-La Plata Project
Expected Sediment Loading During Spring Runoff
Supplemental Sediment Deposition Analysis
for the Proposed Intake Channel Design

BACKGROUND:

Recent (May 2002) analysis of hydraulic conditions within the intake channel indicated the potential for transportation of the incoming sediment during high river flows. Subsequently, design changes (specifically the lowering of the channel invert immediately behind the intake structure) in the intake channel have warranted additional study of hydraulic conditions relative to the incoming sediment load. The analysis reported herein identifies river stage and corresponding river discharge where significant sedimentation effects will likely commence during spring runoff (sediment-laden) conditions. This report can serve as the basis for DOC (Designers Operation Criteria) recommendations regarding operations of the intake structure and channel during high, sediment-laden river flows that may occur in June.

METHOD:

For the May 2002 report prepared by the Bureau of Reclamation (Lyons, 2002), maximum seasonal sediment loading at the Durango Pumping Plant (DPP) was estimated using monthly flow and projected pumping requirements during the month of June. A review of flow conditions during May through July demonstrated that snowmelt runoff for the Animas River generally peaks during June. Also, the effects of intake channel hydraulics and the overflow crest gate on suspended sediment loading at the DPP were estimated using an approach based upon theoretical distributions of suspended sediment particles in the vertical flow profile. These approaches are also used in this analysis.

Daily estimates of Animas River flow and DPP pumping were obtained from the operation study data supplied by WCP-Durango. Median (1946) and Maximum (1957) monthly river flow conditions for June were selected based upon the 1929 - 1993 record for the Animas River at Durango gauge in the data supplied by WCP-Durango. DPP pumping rates for all days in June 1946 are 226 ft^3/s (cubic feet per second) and 240 ft^3/s for all days in June 1957.

Suspended sediment concentration was estimated for the Animas River from a sediment rating curve developed for measured suspended sediment data collected during the 1950s and 1970s. The maximum measured suspended sediment concentration was 816 mg/l (milligrams per liter) from these data. An average sediment size gradation was computed from these data and adjusted to reflect the exclusionary effect of the planned overflow crest gate.

The effect of the proposed overflow crest gate at the river intake structure was estimated by computing the concentration of suspended sands in the upper part of the flow relative to the measured concentration from depth-integrated samples. This approach is described in Federal Interagency Sedimentation Project (1941). The proposed overflow crest gate is designed to operate four feet above the invert of the intake during June. Water surface elevation in the intake channel will vary with the daily river flow.

Sediment transport in the intake channel during these anticipated conditions of river flow and sediment transport was evaluated with a numerical model of sediment settling (SETSIZE model documented by Randle, 1984). A daily value for water surface elevation in the intake channel, daily sediment concentration, daily sediment size gradation, daily intake channel flow, and the dimensions of the intake channel were evaluated for the median June(1946) and maximum June (1957) conditions. The quantity of sediment deposited under these conditions is estimated in this numerical simulation of flow and sediment transport using the computational procedures described for settling basins in Pemberton and Lara (1971).

A review of Animas River daily stream flow data for the month of June for the years 1929 through 1993 was completed to determine the percent of time flows of a given magnitude could be expected to occur during the snowmelt runoff period. This flow-duration analysis can be used to quantify the amount of time that flow in the Animas River exceeds an identified threshold level. This threshold flow level can be related to a river stage and associated depth of flow over the overflow crest gate and in the intake channel that results in conditions of excessive sedimentation in the intake channel. The recommended river flow and at which pumping should be curtailed was made by inspecting the record of flows and associated sediment settling with the goal of minimizing the number of days with no pumping. Choosing a lower threshold river flow would further reduce the seasonal sediment deposition but decrease the pumping volume for the season.

RESULTS:

Estimating Sediment Deposition With A Settling Basin Numerical Model

Notwithstanding the beneficial effects of the overflow crest gate in reducing sediment concentration of the incoming flow, sediment deposition within the intake channel will likely occur during the snowmelt runoff season because of the slow velocities and depth of flow found in the channel. Sediment loading was estimated using daily Animas River flow and estimated suspended concentration based upon the daily flow for both median and maximum June monthly flow conditions. For the median (1946 conditions) June, when daily streamflow averaged about 2,456 ft^3/s (ranging from about 760 ft^3/s to 3,944 ft^3/s), about 226 ft^3/s is scheduled to be pumped at the DPP, and the monthly calculated sand-sized sediment load carried into the DPP was about 704 cubic yards. Of this incoming sediment load, about 160 cubic yards of material is predicted, by the SETSIZE model, to be deposited in the intake channel upstream of the fish screen structure. Comparable analyses for years with similar runoff conditions (1930 and 1953) yielded similar deposition predictions.

For the historic maximum (1957 conditions) June, when daily streamflow averaged about 5,382 ft^3/s (ranging from 2,553 ft^3/s to 8,466 ft^3/s) 240 ft^3/s is scheduled to be pumped at the DPP, the monthly sand-sized sediment load carried into the DPP is about 3,317 cubic yards. Of this incoming sediment load, about 650 cubic yards of material is predicted, by the SETSIZE model, to be deposited in the intake channel upstream of the fish screen structure.

A review of additional hydrologic data for years with runoff volume during June similar to those found in1957 demonstrated no significant differences in the settling basin analysis results.

Reduction of the Sediment Deposition Estimate for Maximum June Conditions

Reducing sediment deposition during maximum anticipated runoff (June 1957) conditions can be accomplished by eliminating pumping during periods of high river flow and high sediment concentration. By eliminating pumping on 14 days when mean daily Animas River flow is greater than 5,800 ft^3/s, the estimated deposition in the intake channel is reduced from 650 cubic yards to 234 cubic yards. Although nearly 50 percent greater than the expected sediment deposition during average (as represented by 1946) conditions, this diminishment of deposition illustrates that sediment loading can be managed during the peak snowmelt runoff while allowing some pumping.

Average daily flows exceeding 5,800 ft^3/s occurred on 144 days, or 7 percent of the time, during 1929-1993. About two-thirds of these days, or 5 percent of the time, occurred during the month of June. Thus, during June, there is a five percent chance that flows will exceed the 5,800 ft^3/s-threshold identified as important to consider relative to sediment deposition conditions.

This analysis indicates that when river discharge approaches or is greater than 5,800 ft^3/s, sediment transport conditions in the Animas River and the depth of flow in the intake channel result in a significant depositional environment upstream of the fish screens. Curtailment of pumping at DPP should be considered at this river stage threshold if sediment deposition is deemed to be a significant factor in plant maintenance. Other factors to consider in decisions to suspend pumping could include the synoptic runoff forecast, observed river discharge and sediment concentration, reservoir storage requirements, and the maintenance requirements and costs associated with removing sediment from the intake channel.

CONCLUSION:

The conclusion reached in the May 2002 report regarding sediment deposition within the DPP intake channel has been re-examined using numerical modeling of flow and sediment transport during the snowmelt runoff season for the Animas River. The current studies reported here indicate that plant operations could be impacted during periods of peak runoff when sediment concentrations in the Animas River are relatively high. Curtailment of pumping during above-average runoff conditions is recommended for operation planning. A threshold discharge of 5,800 ft^3/s in the Animas River is recommended as an upper limit for pumping so as to avoid substantial sediment deposition in the intake channel.

REFERENCES:

Federal Interagency Sedimentation Project (1941). A study of the methods used in measurement and analysis of sediment loads in streams - Report 3 - Analytical study of methods of sampling suspended sediments. St. Paul, Minnesota, 82 p.

Lyons, J. K. (2002, May 14). Durango Pumping Plant -- Animas-La Plata Project. Expected Sediment Loading During Spring Runoff. Bureau of Reclamation, Denver Technical Service Center, 3 p. plus attachments.

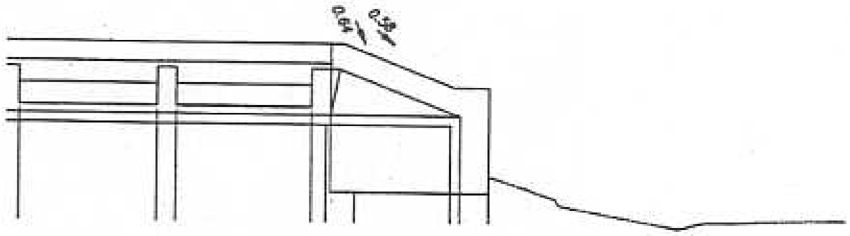
Pemberton, E. L. and Lara, J. M.(1971). A procedure to determine sediment deposition in a settling basin - Sedimentation investigations technical guide series: Section E, Intake works and desilting basins, Part 2, Settling basins. Bureau of Reclamation, Denver Technical Service Center, 8 p.

Randle, T. J. (1984). User's guide to computer modeling of settling basins. Bureau of Reclamation, Denver Technical Service Center, 4 p. plus attachments.

Sediment Transport Enhancement Near Inlet Mouth

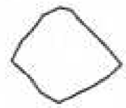
No Rock
Velocities @
Elev. 6437.34

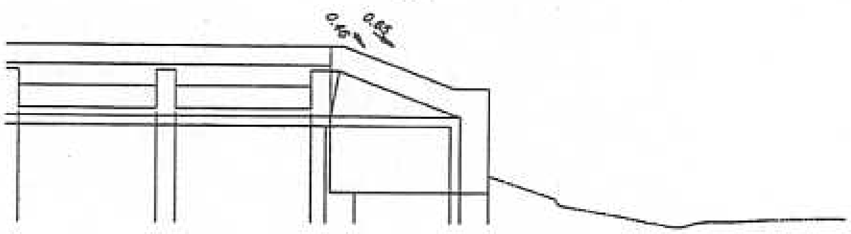
Q = 200 cfs



Rock Position A
Velocities @
Elev. 6437.34

Q = 200 cfs





Rock Position B
Velocities @
Elev. 6437.34

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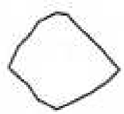

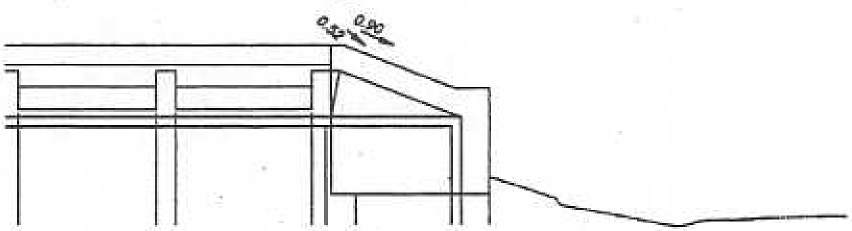

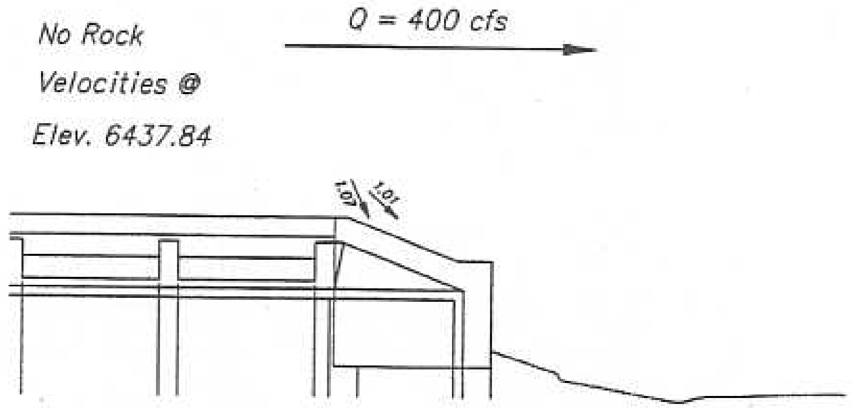
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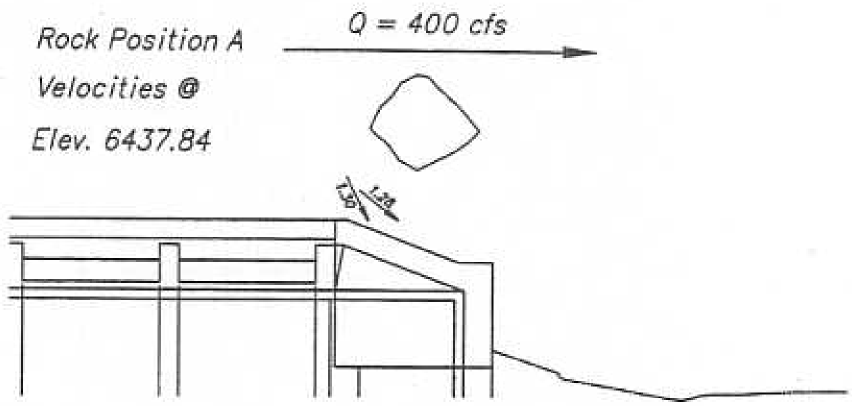
Velocities @
Elev. 6437.84
Q = 400 cfs
1.07
1.01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Velocities @
Elev. 6437.84
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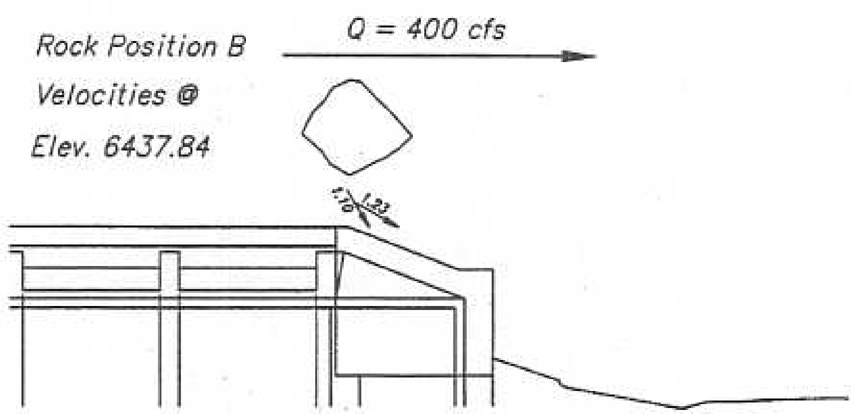
1.30
1.28


Rock Position B
Velocities @
Elev. 6437.84
Q = 400 cfs
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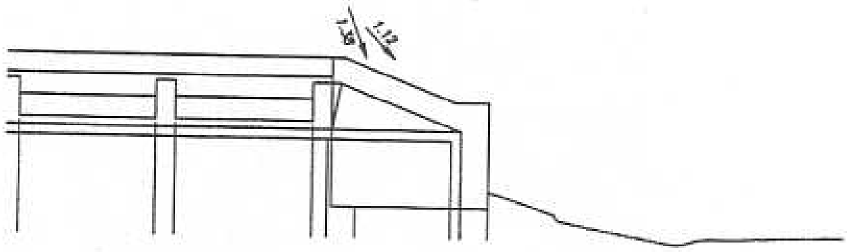
1.23

No Rock

Q = 580 cfs

Velocities @

Elev. 6437.84

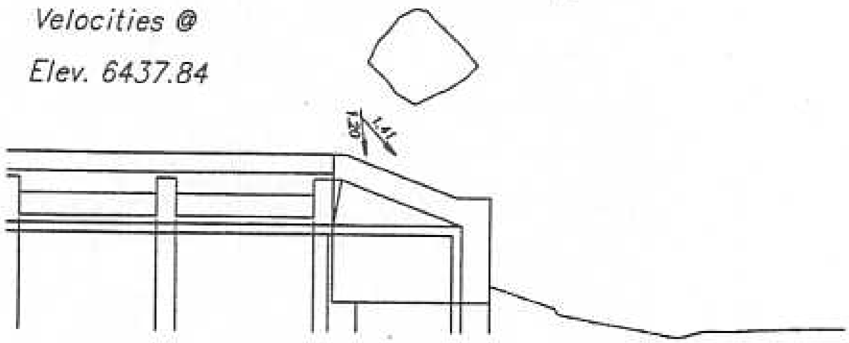


Rock Position A

Q = 580 cfs

Velocities @

Elev. 6437.84



Rock Position B

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Velocities @

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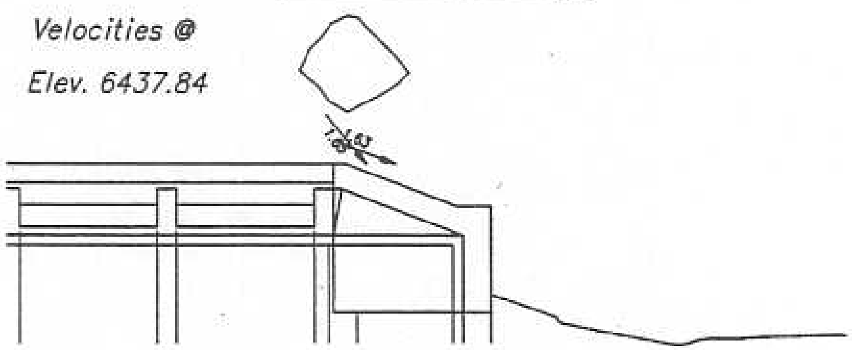
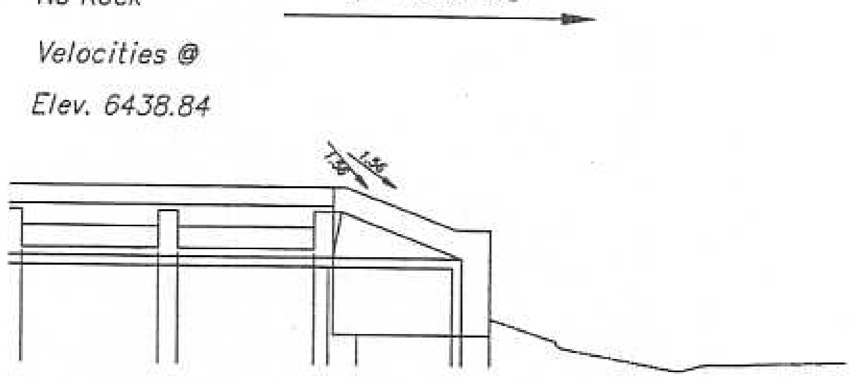


No Rock

Q = 1200 cfs

Velocities @

Elev. 6438.84

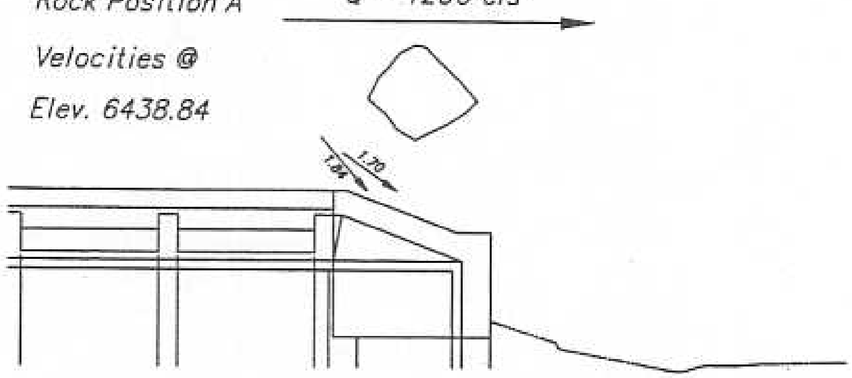


Rock Position A

Q = 1200 cfs

Velocities @

Elev. 6438.84

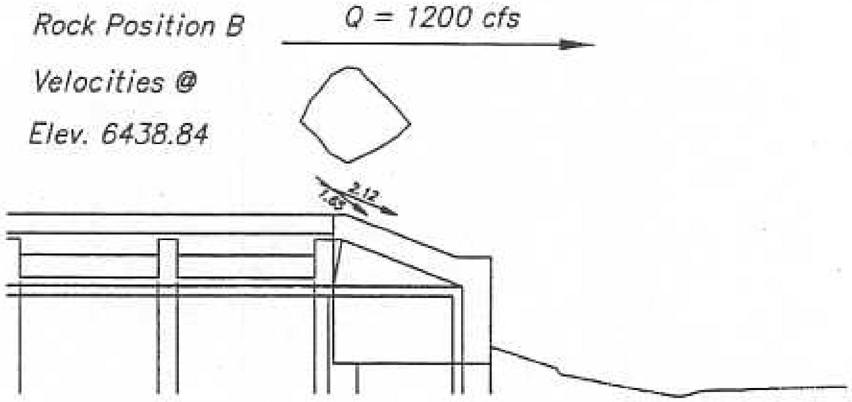


Rock Position B

Q = 1200 cfs

Velocities @

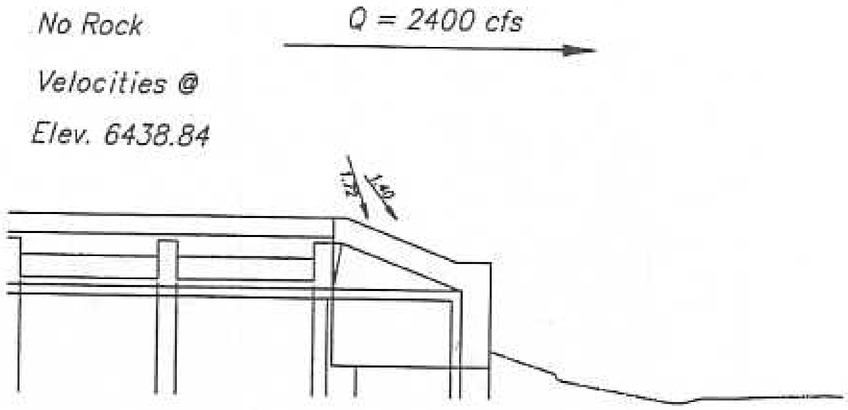
Elev. 6438.84



No Rock

Velocities @

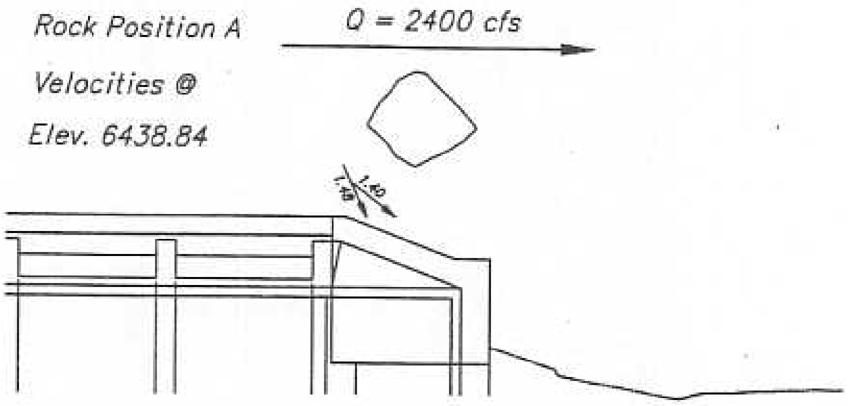
Elev. 6438.84



Rock Position A

Velocities @

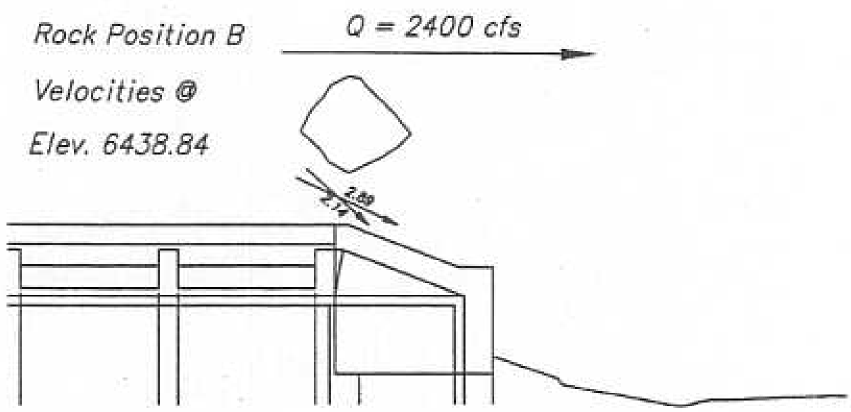
Elev. 6438.84



Rock Position B

Velocities @

Elev. 6438.84

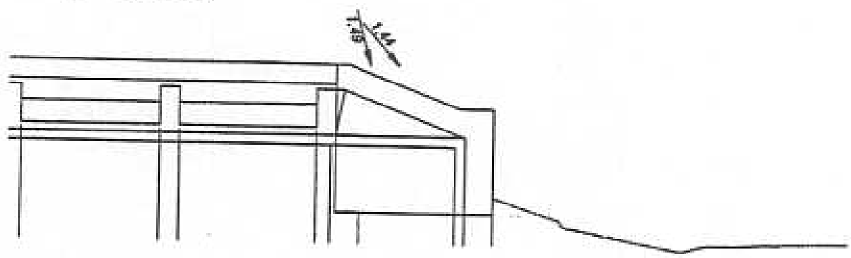


No Rock

Q = 3600 cfs

Velocities @

Elev. 6438.84

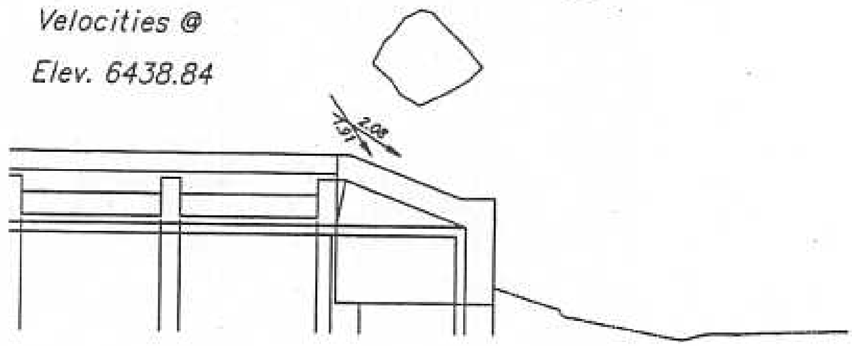


Rock Position A

Q = 3600 cfs

Velocities @

Elev. 6438.84



Rock Position B

Q = 3600 cfs

Velocities @

Elev. 6438.84