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Managing Water in the West

Hydraulic Laboratory Report HL-2004-01

Physical Model Study: City of Albuquerque Drinking Water Project

Sediment Management at the Proposed Rio Grande Diversion

U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
Water Resources Research Laboratory
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14. ABSTRACT

A 1:24 scale physical model study was conducted to qualitatively investigate sediment management issues at the City of Albuquerque NM Drinking Water Project's proposed Rio Grande diversion structure. Discussion of sediment scaling methodologies employed is presented along with descriptions of model verification tasks performed. A variety of sediment exclusion systems were examined in a comparative testing scheme to identify alternatives for limiting diversion of sediments while maintaining positive attraction conditions for a fish bypass that would continuously enable fish passage around the diversion structure. Surface velocity fields were mapped for selected flow conditions near the diversion intake with the aid of digital overhead photography and computer aided design and drafting (CADD) technologies.

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Physical Scale Hydraulic Modeling, Sedimentation Modeling, River Diversion, Sediment Exclusion, Surface Velocity Map, Sediment Diversion, Fish Bypass

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

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This work was conducted by Reclamation for the City of Albuquerque, New Mexico, under agreement with URS Corporation.

Table of Contents

Table of Figures

Background

The city of Albuquerque, NM (City) proposes to construct a diversion structure on the Rio Grande River, to be located on the stream reach between the Paseo Del Norte and Alameda Boulevard bridges. The proposed structure will feature pneumatically operated overshot gates spanning the width of the channel that – when fully opened – lay nearly flat along the channel invert. Diversion will be made from the left (east) of the channel from where water will be pumped to the City's nearby water treatment facility. A physical scale-model study to investigate aspects of sediment management for the proposed structure was performed by Reclamation's Water Resources Research Laboratory (WRRL) under a contract agreement with URS Inc. (URS). A 1:24 scale physical model was constructed at the WRRL laboratory in Denver, CO in which tests were conducted.

Scope of the Model Study

This model study was projected to be capable of providing qualitative feedback regarding aspects of sedimentation issues and sediment management in operation of the proposed diversion structure. The estimated model test matrix at the outset of the study included thirteen tests over five river flow rates. Objectives of the model study included:

- Investigation of operational methods and/or structural alternatives that will limit the amount of bed-load sediments entering the diversion.
- Investigation of how various gate operating scenarios for a selected range of stream discharges impact both the velocity field and sediment deposition near the downstream end of a fishway that will be constructed to allow fish passage around the structure
- Investigation of how various operating scenarios for selected stream discharges might limit the impact of accumulation of sediments in the pooled reach immediately above the structure.

Physical Hydraulic Modeling

General Hydraulic Similitude

Scale model investigations of hydraulic structures have proven a cost-effective means of studying performance of a proposed structure or of proposed structure modifications, provided requirements for hydraulic similitude are observed. In theory, this requires matching the ratio of inertial forces to viscous forces (represented by a dimensionless parameter known as the Reynolds Number) and the ratio of inertial forces to gravity forces (represented by a dimensionless parameter known as the Froude Number) in both model and prototype. The stream Reynolds Number (Re) for open channel flow is calculated as the product of fluid velocity (V) and flow depth (y) divided by the fluid's kinematic viscosity (ν), or Re = Vy/ν. The Froude Number (Fr) is calculated as the fluid velocity (V) divided by the square root of the product of the gravitational constant (g) and the flow depth (y), or Fr = V/ $\sqrt{gy}$.

In practical application meeting both criteria would require scaling of not only physical dimensions, but scaling of fluid properties (i.e. viscosity, fluid density) – which can almost never be achieved due to the fact that fluids with suitably scaled properties almost never exist. In most physical model studies of water conveyance and control systems, water is both the model and prototype fluid for economic reasons. If turbulent flow conditions exist in both model and prototype for the aspect(s) of a system being examined, viscous force effects are significantly diminished and observations from model performance will relate to prototype performance within a useful degree of accuracy. Hence physical open channel flow hydraulic models are commonly designed to adhere to Froude number scaling and to maintain turbulent flow conditions for the modeled aspects of interest in order to avoid having viscous forces (commonly referred to as "Reynolds effects") impact model performance. A stream Reynolds number of 2000 represents the minimal range for turbulent flow conditions.

Additional Sediment Modeling Considerations

Studies to date of the hydraulics of sediment transport with the extensive associated complexities has yielded a diverse group of empirically derived predictive methodologies that are often applicable only for a limited range of conditions. Franco (1978) describes alluvial channel engineering as ". . . a matter of experience and general judgment." When attempting to account for the impacts of scaling on predicted performance characteristics of a physical scale model, the degree of imprecision is magnified.

For sediment movement, the hydraulic scale of interest is at the bed sediment particle diameter. Particle movement is a function of shear force – or the drag force – exerted by fluid moving past bed particles exceeding forces holding the particles in place. Bed shear (τ_o) is calculated as the product of fluid density (ρ) [or more correctly density of the fluid and suspended particle mixture] and the square of the shear velocity (u_*). Shear velocity is calculated as the square root of product of the gravitational constant (g), the channel's hydraulic radius (R) [for wide shallow channels like the modeled reach of the Rio Grande, hydraulic radius is approximated by the depth of flow (y)], and slope (S). Thus $u_* = \sqrt{gyS}$ and $\tau_{o} = \rho\, u_*$. The magnitude of drag force exerted depends on degree of turbulence present and thus is a function of the Reynolds Number. The form of the Reynolds Number used for consideration at the bed particle scale is known as the "Grain" Reynolds Number, (Re_*), defined as the product of the shear velocity (u_*) and grain size (d_s) divided by the fluid kinematic viscosity (ν) or $Re_* = u_* d_s/\nu$.

Normally, it is not feasible to simply reduce particle size according to geometric model scale. An obstacle frequently encountered is that non-cohesive prototype particles (i.e. fine sands) geometrically scale into clay-size particles exhibiting a highly dissimilar cohesiveness properties. It is therefore necessary to distort the scaling of grain size by using model sediments greater than called for using geometric scaling. Model particle size in excess of scaled value may require using a lower density bed material, an increase in bed slope, or a combination of both to produce transport rates with a reasonable degree of similarity with prototype rates.

The U.S. Army Corps of Engineers (USACE) has probably been more actively involved in moveable bed (sediment) modeling that any other entity. In a booklet entitled *Guidelines for the Design, Adjustment and Operation of Models for the Study of River Sedimentation Problems*, Franco (Franco, 1978), identifies extensive field data needs along with a rigorous process of validation against field performance parameters. Costs associated with the manpower and time frame necessary to follow this model calibration process are of a magnitude that renders this method feasible only for modeling projects with extended life expectancy. [i.e. Some USACE sediment model studies have been ongoing over a period of years.]

For studies of a more limited scope, an approach that streamlines design and adjustment process for a moveable bed physical scale model has been implemented in previous studies at the USBR WRRL (Pugh and Dodge, 1991). This approach is based on an apparent relationship between dimensionless parameters of bed shear (τ_*) – known as Shields' parameter – and dimensionless unit sediment transport (q_{*s}) – known as Taylor's Function. Shields' parameter (τ_*), is defined as the bed shear (τ_o) divided by the product of buoyant specific weight (γ_s - γ) and particle size (d_s), or

$\tau_* = \tau_o /((\gamma_s - \gamma)\, d_s)$.

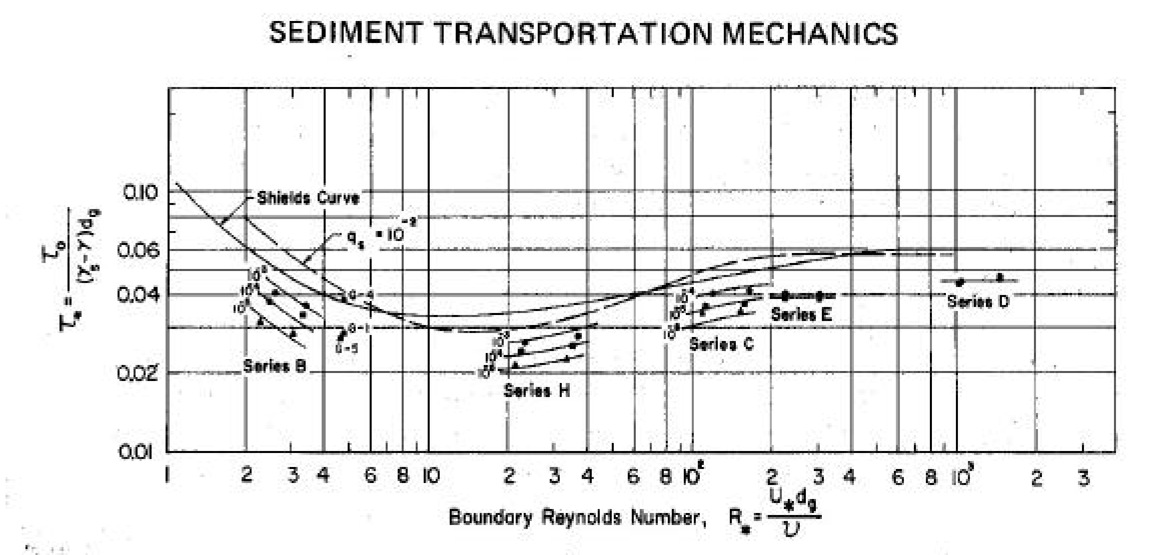
The threshold condition for either mobilization or deposition of a given particle is commonly referred to as the condition of incipient motion. Using data from laboratory flume studies, Shields was able to show that for a given grain Reynolds number, there is a unique dimensionless shear value at which the state of incipient motion exists. Dimensionless shear values representing the incipient motion state plotted against grain Reynolds number produce a curve for the condition of incipient motion. Dimensionless shear values that lie on this curve are known as "critical" Shields parameter values. (Vanoni, 1975)

Incipient motion is not regarded as a state of no transport for a given particle class. Rather it is a state where that class of particle is equally likely to be mobilized if it is initially stationary, or to be deposited if it is initially in motion. Thus some small rate transport occurs at the state of incipient motion. In subsequent studies with low sediment discharge rates by Taylor (as discussed in Vanoni, 1975), the amounts of sediment being transported from the flume were measured and results analyzed in terms of dimensionless unit sediment transport. Dimensionless unit sediment transport (q_{*s}

— known as Taylor's Function) is defined as the volume of sediment discharge per unit of flow width (q_s) divided by the product of the shear velocity (u_*) and sediment grain size (d_s) or $q_{*s} = q_s/(u_* d_s)$. Taylor determined Shields' parameter values and Taylor function values for each data point. He found that when Shields' values for data points of constant Taylor's function value were plotted against grain Reynolds number, curves approximately parallel to the critical Shields parameter were produced. (Vanoni, 1975).

When plotted on the Shields' diagram, Taylor's data appears below the critical Shield's parameter suggesting that the critical Shields' curve represents a constant Taylor's function value of some small value of dimensionless unit sediment transport. Figure 1 is the Shields' diagram showing the apparent parallels Taylor found for Shields' parameter values associated with constant Taylor's function values, and the critical Shield's parameter curve.

Figure 1



Dimensionless shear vs. grain (a.k.a. boundary) Reynolds number showing the approximate parallel relationship between dimensionless shear (Shields' parameter) for constant values of dimensionless unit sediment discharge (Taylor's function) and the dimensionless shear for the condition of incipient motion (critical Shields' parameter). [from Vanoni 1975]

In keeping with theory relating similitude and dimensionless parameters it follows that when Taylor's function values for scale model and prototype are equivalent, similitude would exist in sediment transport. Pugh and Dodge proposed that the parallel relationship Taylor had shown between Shields' parameter values associated with constant Taylor's function values for small rates of sediment discharge and the critical Shields' values might hold for higher sediment discharge rates. If so, the target Shields' value for the model and the corresponding prototype Shields' value would lie on the same curve paralleling the critical Shields' values curve. By equating the differentials between actual Shields values and critical Shields' values at the respective grain Reynolds numbers for both model and prototype, this relationship could be achieved.

For cases where prototype grain Reynolds' number is greater than 100, but where geometrically scaled particle size produce grain Reynolds numbers below 100, Pugh and Dodge reported success achieving target differentials between model dimensionless shear and critical Shield's values by increasing model particle size. A guideline used was to attempt to equate model and prototype particle fall velocity (ω_o). [Fall velocity is terminal velocity of a particle of given shape, density and size falling through a fluid of given properties. Fall velocities are commonly approximated using empirically derived relationships based on laboratory observations.] Issues encountered in applying this methodology are discussed below in the Design of the Physical Model section below.

The parallel relationship between grain shear for constant value Taylor's functions and critical Shields' values provides a design guideline for similitude in sediment transport capacity. Actual transport rates are a function of both transport capacity and sediment availability. A scaled sediment load relationship was derived by applying an empirical bed load transport equation to both model and prototype over a range of discharges. A mathematical relationship was then identified between corresponding predicted loads.

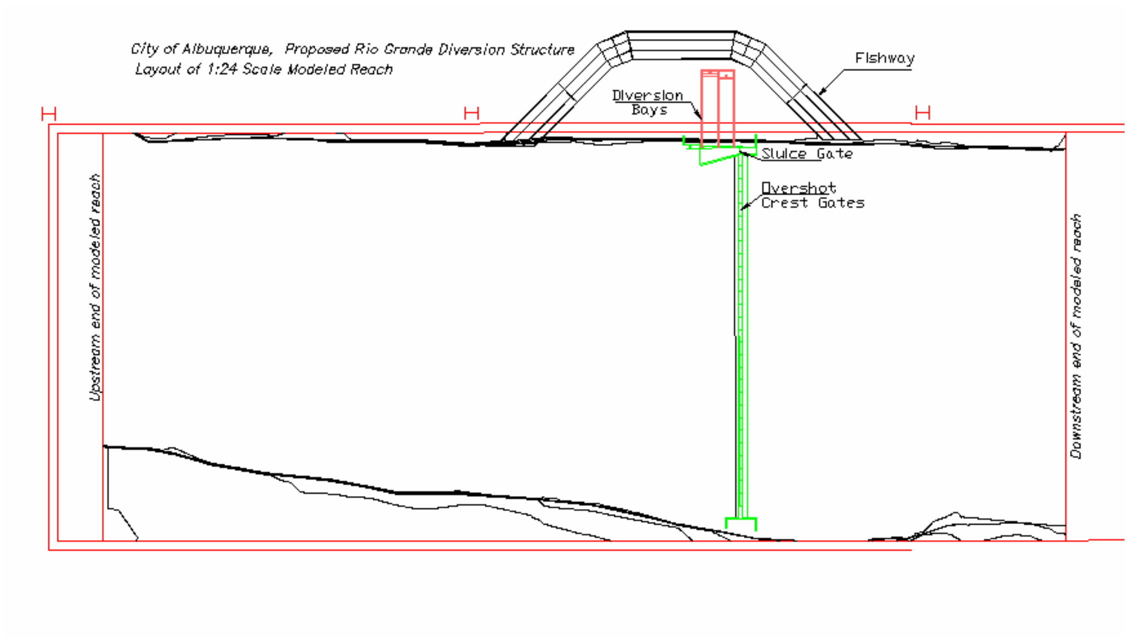
A formulation of the Meyer-Peter & Mueller (M-P&M) equation as presented in Vanoni, (1975) was utilized for this purpose. M-P&M was developed for sand grain sediments and can be applied to sediments of varying density. The derived relationship was utilized to compare measured sediment feeding rate in the model with prototype bed load field data.

Design of the Physical Model

Selection of Scale

Selecting an appropriate scale was a function of space availability, including as many features as possible and maintaining a large enough scale to limit the effect of viscous forces. The available model box was approximately 27 ft wide and 90 ft long. A 1:24 scale factor was identified as near the upper limit for scale that would enable construction of the entire diversion structure and bank-full model of the upstream channel within the 27 ft box width. Even at this scale, the Reynolds number in the diversion bays (assuming 65 cfs diversion per bay and 3 ft depth) is approximately 1600. This falls below a minimum value of 2000 for turbulent flow conditions. Thus model flows in the diversion bays are subject to some degree of Reynolds' effects. Due to higher velocities, flows through gate openings on the diversion structure should be in the turbulent range and be less impacted by Reynolds effects. Figure 2 is a plan sketch of the initial layout of the 1:24 scale physical model.

Figure 2.

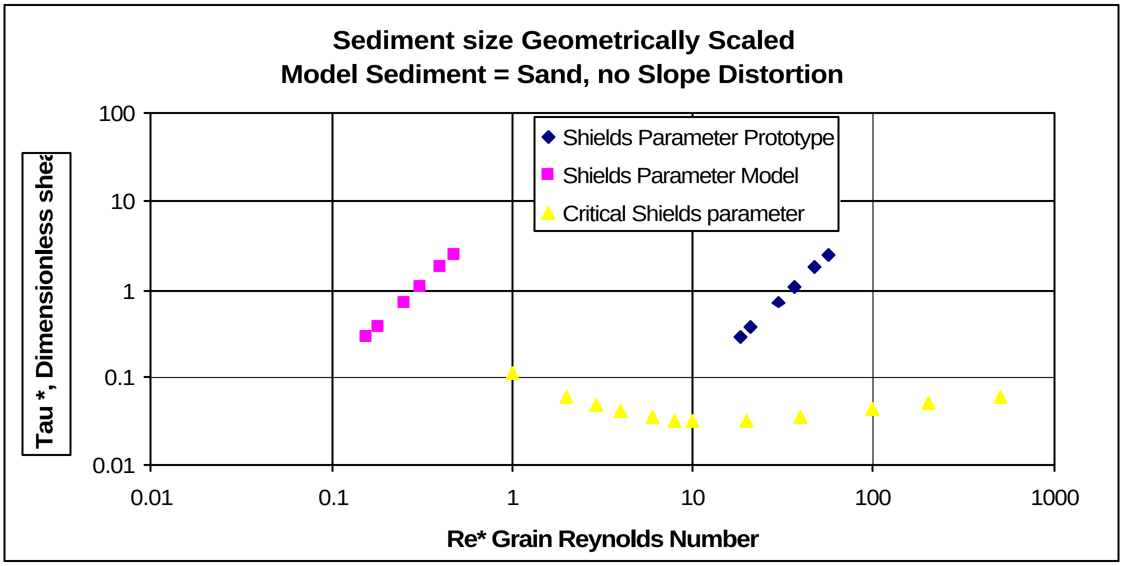


Plan sketch of the initial configuration of the 1:24 scale physical model. The modeled reach includes just over 1000 ft of the Rio Grande channel upstream from the diversion structure and just more than 500 ft of the downstream channel. The plan layout of the fishway was altered to fit available laboratory space while leaving fishway entrance and exit unchanged.

Examination of Sediment Transport Similarity

For this study model adjustments were identified following a sediment transport model scaling methodology described by Pugh (Pugh 2000). The first step was to look at Shields' values for particles of prototype density and of geometrically scaled size with equivalent slope in both model and prototype. This produced equivalent Shields values for both model and prototype. A comparison of Shields' values for selected prototype grain Reynolds values and for corresponding model grain Reynolds values using geometrically scaled sand grain particles is shown in Figure 3.

Figure 3

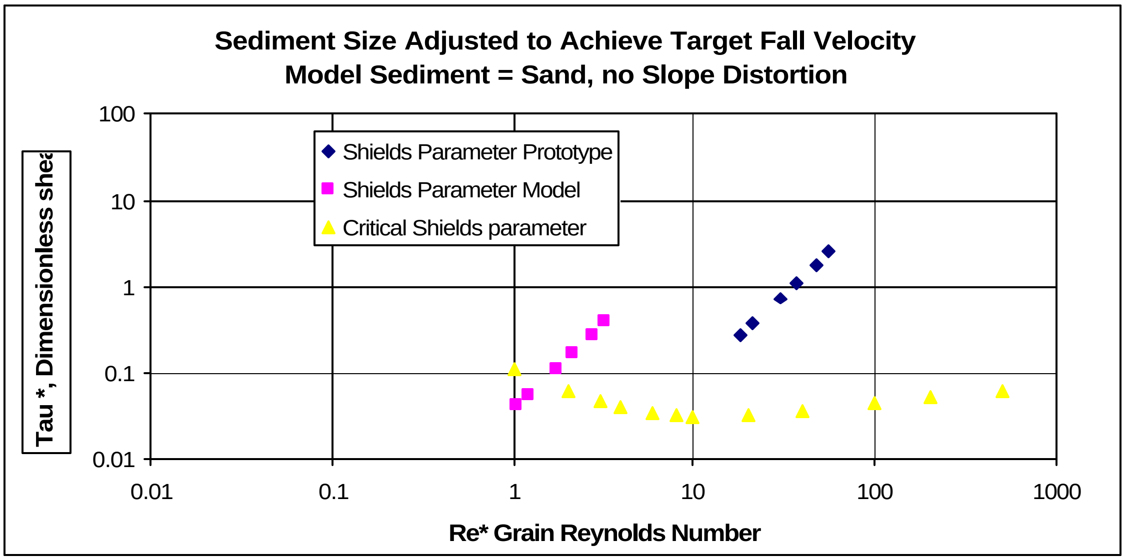


Shields' diagram comparing prototype dimensionless shear with geometrically scaled sand grained model sediments with no slope distortion. Note the extremely low calculated model grain Reynolds' numbers.

A grain Reynolds number of 100 is a threshold for turbulent flow. In cases where both prototype and model are well above this threshold (i.e. grain Reynolds number for both > 10^2) the critical Shields' value will be approximately constant and little if any adjustment to model characteristics would have been needed. It is readily apparent from Figure 3 that grain Reynolds numbers for both model and prototype are below this range.

Studies reported by Pugh and Dodge featured prototype grain Reynolds numbers greater than 100 with corresponding model grain Reynolds numbers below the turbulent threshold. For this condition, model operations fell in the range where the model critical Shields value is below that of the prototype. For these conditions an adjustment was made by increasing model grain size in order to match the Froude scaled fall velocity of prototype grain size. [Fall velocity defined as the terminal velocity a particle reaches when falling through a fluid. Fall velocities based on laboratory studies have been identified for a range of sand particles falling through clear water. Froude number scaled model velocity is obtained by dividing prototype velocity by the square root of the geometric scale value.] For this study, the fall velocity adjustment yielded the relationship shown in Figure 4

Figure 4

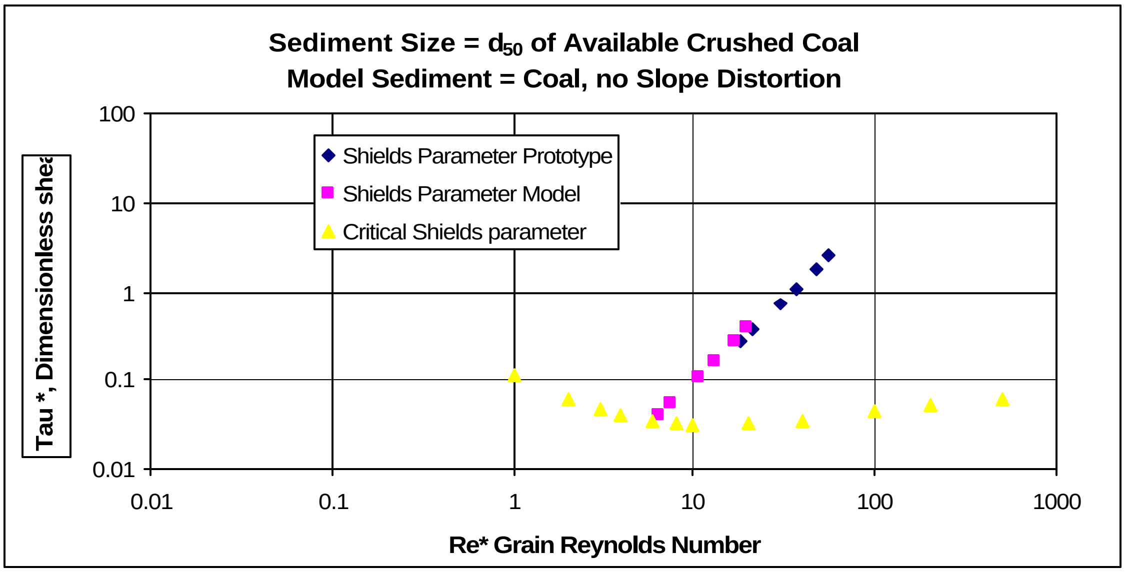


Grain size in model is adjusted (increased) to achieve equivalent fall velocity with prototype. The resulting model grain Reynolds' numbers are increased almost an order of magnitude, but this adjustment fails to locate corresponding dimensionless shear values on curves parallel to the critical Shields' parameter.

As is apparent in Figure 4, making the fall velocity adjustment was insufficient for achieving the targeted equivalent differentials for both model and prototype between actual and critical Shields' parameter values. The fall velocity adjustment results in a shift down and to the right on the Shields' diagram for model values. For this study it became necessary to examine means to extend/adapt application of the apparent parallel relationship between dimensionless shear for constant Taylor's function values and critical Shields curve beyond previously reported methods.

Figure 4 suggests that even after sediment size is increased to meet scaled fall velocity criteria model sediment transport would be non existent or well below transport rates for corresponding prototype stream discharge rates. A model sediment of lower density needed to be considered. The next step taken was to consider an available supply and gradation of crushed coal. Specific weight of the coal was measured at 79 lbs/ft^3 and a gradation test revealed a grain d_{50} of 0.88mm. The resulting shift in model Shields values with the coal as the model sediment is shown in Figure 5.

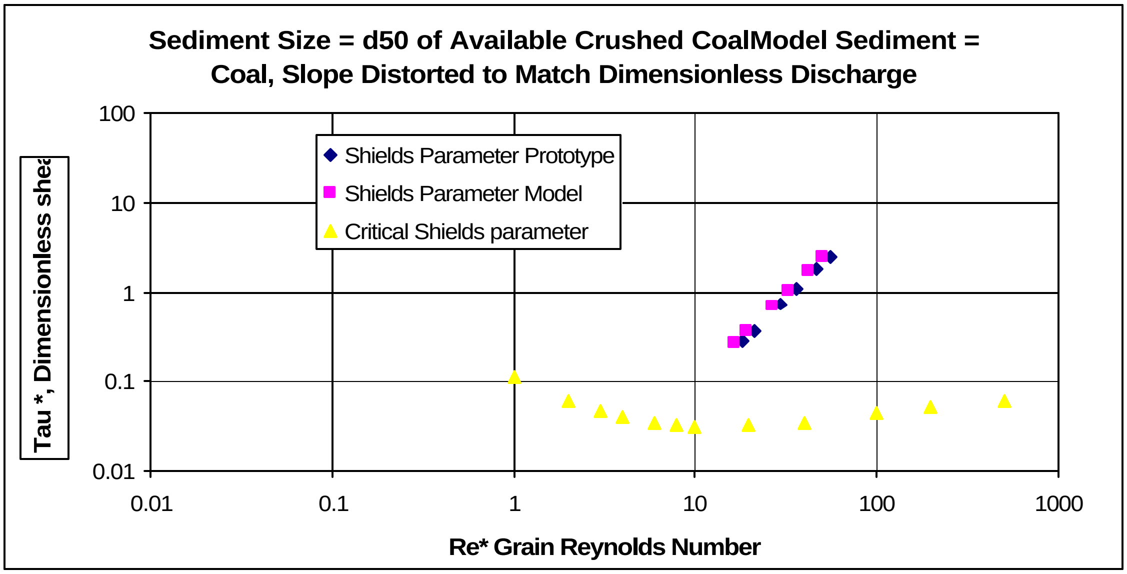
Figure 5



Using an available supply of crushed coal for the model sediment (γ_s = 79 pcf, d_{50} = .88 mm), grain Reynolds' number values are again increased and now all dimensionless shear points lie above the critical Shields' value. Yet corresponding dimensionless shear points for the model are still well below prototype values.

In comparison with figure 4, transport capability for all model conditions is increased, but remains below targeted values. At this point slope distortion was the most practical alternative available to consider for further enhancement of model transport capability. Model slope was adjusted iteratively until target model Shields values were approached. A model slope distortion of 6.5 (model):1(prototype) produced the relationship shown in Figure 6.

Figure 6



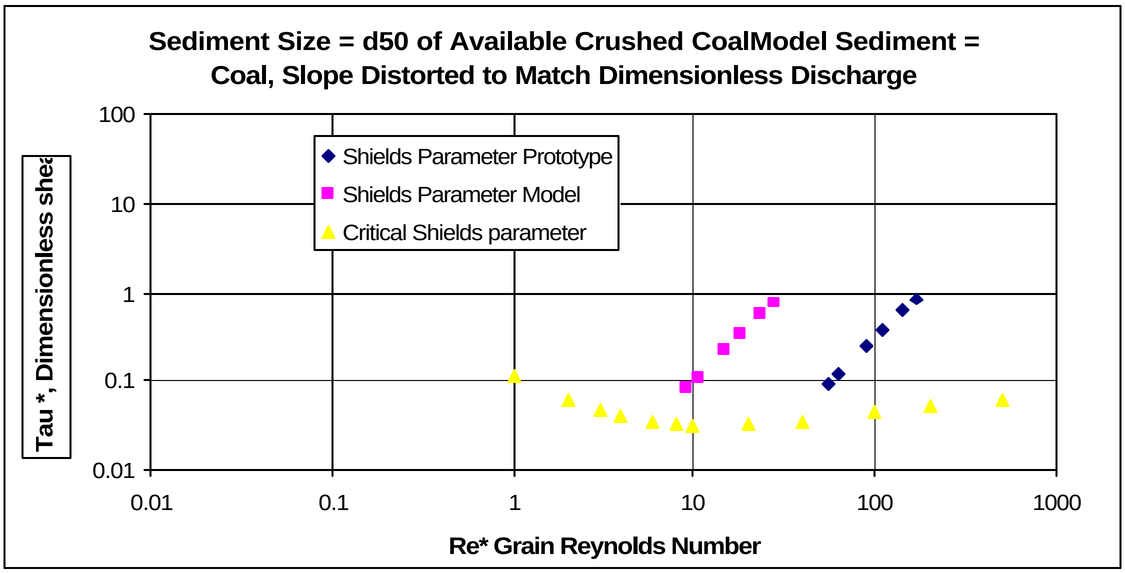
Plot comparing model to prototype dimensionless shear adjusted for smaller prototype sediment grain d_{50} (0.51 mm) indicated by Reclamation sedimentation studies of the modeled reach. A slope distortion factor of 6.5 (model slope = 0.0065) was necessary to achieve suitable shift in dimensionless shear for the model.

From Figure 6 it appears that the adjustments in material density and slope have produced conditions near the target values. If assumptions underlying the design process followed above are valid, using crushed coal with a specific weight of 79 lbs/ft^3 and with a with a grain size d_{50} of 0.88 mm in the model together with a model slope that is increased by a factor of 6.5 times prototype slope, similitude in sediment transport would be expected, given prototype sand grain d_{50} of 0.51 mm and slope of 0.001.

At this point it should be noted that the prototype bed sediment D_{50} value indicated in the design analysis report (Boyle 2003) was approximately 1.5 mm. Field data subsequently obtained from USBR's Sedimentation and River Hydraulics Group (Sedimentation Group) which is involved in an ongoing study that includes this reach of the Rio Grande indicated a prototype grain size of just over 0.5 mm, or approximately one-third the size initially used for design considerations.

The design process for sediment transport similitude had already been worked through using the 1.5 mm prototype sediment size. A significantly smaller slope distortion factor of 2.0 was determined appropriate using the available crushed coal as model sediment. Figure 7 shows the comparative model and prototype Shields values on which model design and construction were based.

Figure 7



A comparison of model and prototype dimensionless shear values using client-supplied value for prototype bed-load sediment size (1.5 mm). Using a slope distortion factor of 2.0 and the available crushed coal as model sediment, corresponding dimensionless shear values are approximately equidistant from critical Shields' values.

At the point the prototype bed load D_{50} value to be used was modified, concrete placement of channel features in the model had been completed using the slope distortion factor of 2.0. To achieve the slope distortion factor of 6.5 needed for the updated D_{50} value of 0.51 mm, a sand wedge was placed atop the concrete in the reach upstream from the diversion structure.

Extension of the increased slope in the reach below the diversion site would have required removing concrete topograpy. Sedimentation issues upstream from the diversion as well as diversion of sediments were the focal points of the model study. It was determined that the impact of leaving the existing downstream slope could be minimized by keeping tailwater elevation immediately below the structure at target levels and manually removing the excess sediment deposition that could be expected in the lower modeled reach before tailwater level was impacted.

Sediment Discharge Scaling Methodology for Model Verification

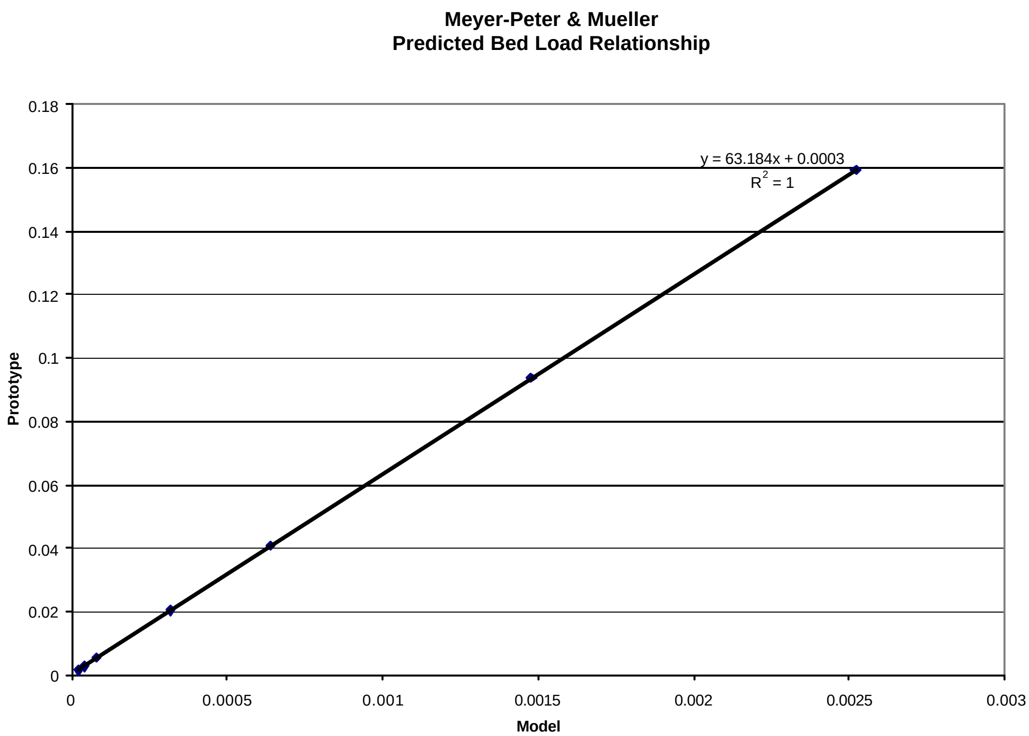
An approximation of the relationship between the discharge rates of model and prototype sediment given the material properties and particle sizes of the respective prototype and model sediments was determined by applying the Meyer-Peter and Muller (M-P&M) bed load transport equation to each case for corresponding Froude-scaled stream discharges. The Meyer-Peter and Mueller equation was developed from studies of sand-sized bed particles with particles of varying densities and provides unit sediment discharge (g_s) in metric tons/meter/second. The formulation of this equation presented by Vanoni (Vanoni, 1975) is as follows:

$$(k_r/k'_r)^{3/2}\gamma r_b S = 0.047(\gamma_s-\gamma)d_m + 0.25(\gamma/g)^{1/3}((\gamma_s-\gamma)/\gamma_s)^{2/3}g_s^{2/3}$$

Where:
- k_r = roughness coeff. (= 1/n where n = Manning's roughness coeff.)
- $k'_r = 26/d_{90}^{1/6}$ (d_{90} in meters)
- γ = specific weight of water (metric tons/cubic meter)
- γ_s = specific weight of sediment (metric tons/cubic meter)
- r_b = hydraulic radius (~ depth for wide channel)
- S = channel slope
- d_m = effective sediment diameter (= $\Sigma_i\, p_i d_{si}$ where p_i = % by wt. of size d_{si})
- g = gravitational constant
- g_s = bed load (metric tons/meter/second)

This equation was manipulated to calculate unit bed load (g_s) for both model and prototype, then multiplied by respective channel widths, as a basis for scaling transport rate given differing model/prototype sediment densities as well as model slope distortions. A numerical approximation of the relationship between M-P&M predicted transport for prototype and model was obtained as linear fit of calculated values. This relationship is shown in Figure 8.

Figure 8



The derived relationship [prototype bed load = 63.184 * model bed load] was used to project the measured rate of sediments fed into the model to prototype scale for comparison with field sediment discharge data which is discussed following section on model verification.

Bed Scour Immediately Downstream from the Diversion Structure

Preliminary structure designs called for placement of approximately a 12.5 ft. wide band of riprap across the river immediately downstream from the diversion structure as scour protection. In order to obtain a qualitative look at potential for scour problems an additional 12.5 ft wide strip of the channel bed beyond the riprap placement initially filled with fine sand that would be susceptible to scour erosion.

Sediment Handling Systems

Sediments were “force fed” into the flow at the upper end of the model. Four 30” wide units adapted from lawn fertilizer spreaders were used to drop crushed coal into the channel. These units were driven by single-speed worm-gear reduction electric motors.

Sediment feed rate was a function both of gate opening at the bottom of the units and of rate of water application via nozzles across the top of each unit.

A series of settling basins was constructed at the lower end of the model using a series of over-flow weirs between basins. After each day of tests, the settling basins were allowed to de-water overnight. Coal was then manually collected from the basins and carted back to the upper end of the model for reuse as part of the daily test set-up regimen. [Prior to initial use, the crushed coal was washed in an effort to separate out fine fractions could be expected to remain in suspension in the model.]

Data Acquisition Equipment and Procedures

Flow Measurement

Flow entering the model is measured using venturi meters in the laboratory pipe system. These meters are recalibrated on 24 month schedules. Calibration histories indicate that typically there has small if any shift in calibration curves over each 24 month time interval.

Flow in each of the diversion bays was measured using Controlotron ultrasonic transit time meters attached to PVC return pipes. These instruments have proven accurate and reliable in previous use in the WRRL lab. After installation, a spot check of calibration of each was performed by passing two discharges of known value from a flexible hose into downstream segment of each diversion bay. Agreement within 3% was observed.

Sediment Diversion

Sediments entering a diversion bay either passed through the diversion and return pipe system, or settled in the bay. Screen-bottomed boxes were placed under flow exiting the return pipes to intercept sediments. After each sediment test, the diversion bays and return systems were thoroughly flushed and diverted sediments were collected in the screen-bottomed boxes. A “wet” weight of collected sediments was obtained using an electronic scale after each test.

Photographic Documentation

Three modes of photographic documentation were employed for various processes in the model study, including still digital photographs, and digital video and magnetic video. Introduction of floating objects and dye injection were used at various times to enhance photographic documentation of tested conditions. More detailed description of some of the utilization of photographic documentation is provided below.

Model Verification

For the limited scope of this model study – dictated by budget and time constraints – model verification consisted of:

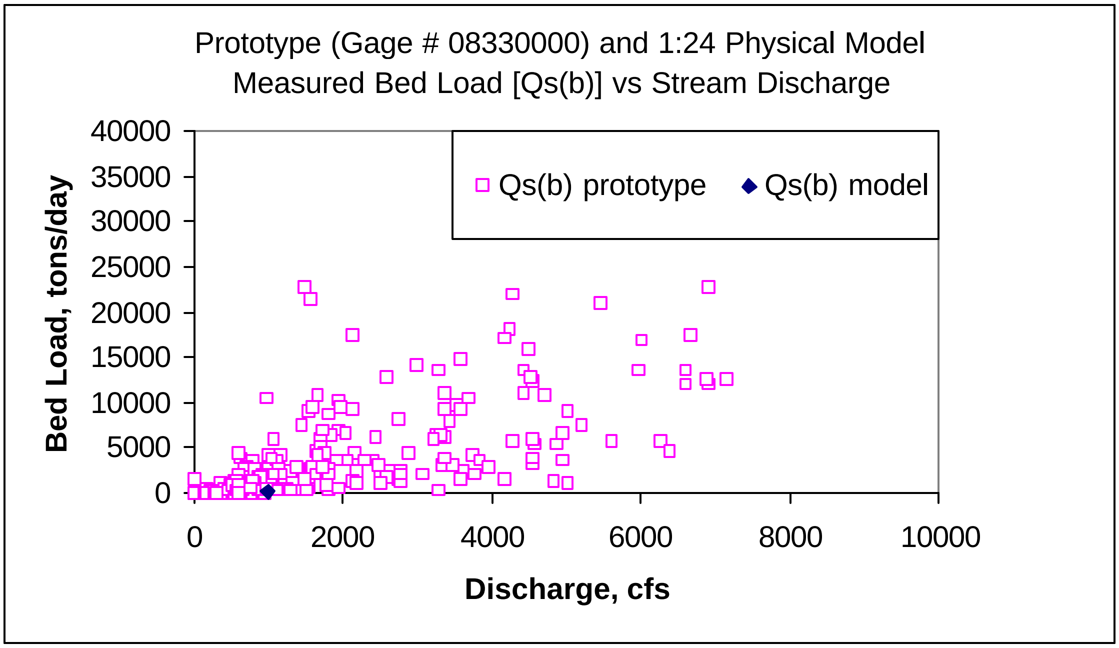
- A timed collection of model sediments (crushed coal) from each of the four model sediment feeding boxes
- Visual assessment of sediment transport processes

Sediments were collected for 60 seconds from each of the four sediment feeding boxes for a modeled stream discharge of 1000 ft^3/s. The sediments were dried and found to weigh 742.8 g (= 1.64 lbs). Converting lbs/min this equates to 1.179 tons/day.

This value was scaled to prototype using the relationship developed above using the Meyer-Peter & Mueller equation. The projected corresponding prototype sediment discharge was calculated as 63.184 * 1.179 = 74.5 tons/day.

When compared with field measurements of bed load discharge at the nearby USGS gage # 08330000, this projected prototype loading appears to be a modest rate, but fall within the observed range. This rate of bed-load transport appeared within reason for meeting the objectives of the model study. The prototype projection of the measured model sediment load comparison with field data is shown below in Figure 9.

Figure 9



Sediment Discharge vs Stream Discharge: Measured field data exhibits a broadly distribution which reflects variability in sediment loads that are introduced from different tributaries and the impacts of differing storm intensities. As seen in the plot modeled sediment load (74.5 tons/day @ 1000 cfs stream flow) is representative of modest loading, but falls within the range of observed field conditions.

The primary criteria used for visual assessment of sediment transport were:

1) Sediments could be observed moving along the bed for stream discharge rates where an appreciable degree of bed load transport would be expected in prototype
2) The channel did not appear to be attempting to change the slope, (i.e. no evidence of extensive aggradation or degradation for pre-dam flow conditions).

In preliminary runs, the model was first operated for 1 hour at a discharge of 1000 ft^3/s with all gates of the diversion structure fully open, with no sediments fed into the model. A moderate degree of channelization was observed along the reach above the diversion structure where a wedge of sand had been placed to achieve channel slope distortion. This was followed by a two hour run (also at a discharge of 1000 ft^3/s) during which crushed coal was fed into the channel. At the end of the two hour run coal was being actively transported along the length of the modeled reach with no significant degree of accumulation near the upstream sediment feeding boxes. Deposition was evident along the channel in low velocity areas. A higher rate of deposition was observed in the reach below the diversion structure – the reach that remained at a 0.002 slope. Much of the coal was carried beyond the modeled reach to the settling basins. Figure 10 is a photograph of the model after both preliminary runs had been completed.

Figure 10

A view of the steam bed (looking downstream) after "pre dam" condition tests. All gates were fully opened to simulate the channel without the diversion structure. Some channelization can be seen in the sand wedge portion in the foreground. Coal deposition has occurred in regions of lower velocity flow in the foreground, and in the reduced slope reach downstream from the diversion structure.

Observed model performance for the channel reach upstream from the diversion structure simulated expected prototype behavior within a reasonable extent. Below the diversion structure (where the increased slope distortion adjustment necessary due to the prototype d_{50} change from 1.5 mm to 0.51 mm d_{50} could not be implemented) the transport capacity appears to be significantly lower, hence the comparatively higher rate of sediment deposition. This observation suggests that the methodology employed of adjusting model parameters to achieve target sediment transport similitude was effective for conditions present in this study.

Stream Discharge for Model Tests

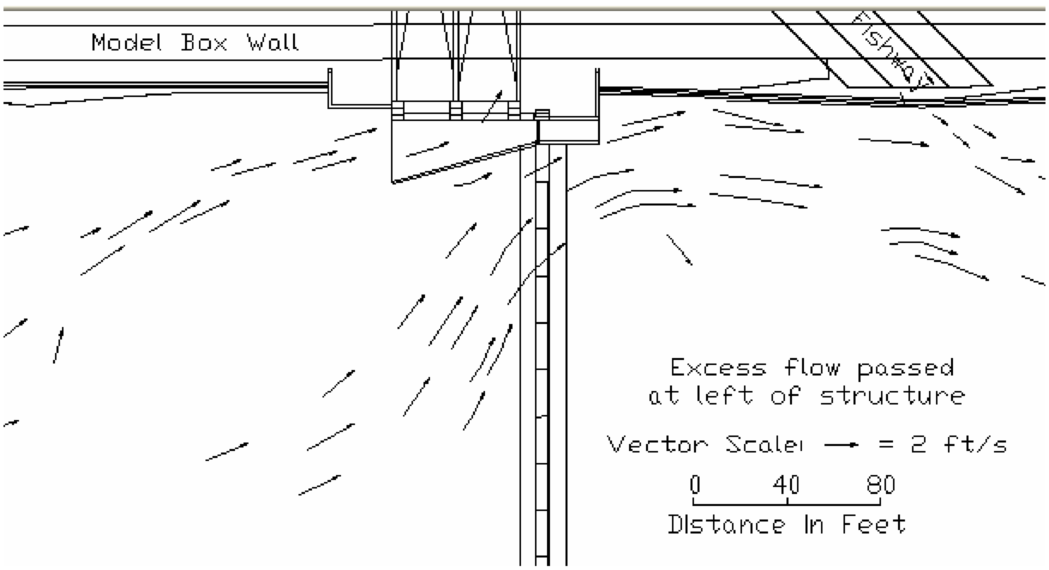
The discharge selected for the initial model tests was 1000 ft^3/s. An analysis of daily recorded flows at the nearby USGS gage @ 08330000 showed that for water years 1990 – 2002, the median daily discharge is 847 ft^3/s. A discharge of 1000 ft^3/s was exceeded on approximately 38% of the days over the same period. It was concluded that 1000 ft^3/s represents a mid- to upper mid-range discharge which should provide a characterization of expected of sediment transport conditions.

Flow-Field Tests

Subsequent to the "pre-diversion" verification, tests with three gate operating scenarios were performed at 1000 ft^3/s river discharge, 130 ft^3/s diversion and a pool surface elevation upstream of the diversion structure of 4995 ft. to examine velocity fields in the proximity of the diversion mouth and fishway entrance. First, excess flow was passed at river left, near the diversion. In the second test, excess flow was passed near the center of the channel. For the third test, excess flow was passed at river right adjacent to the right structure abutment.

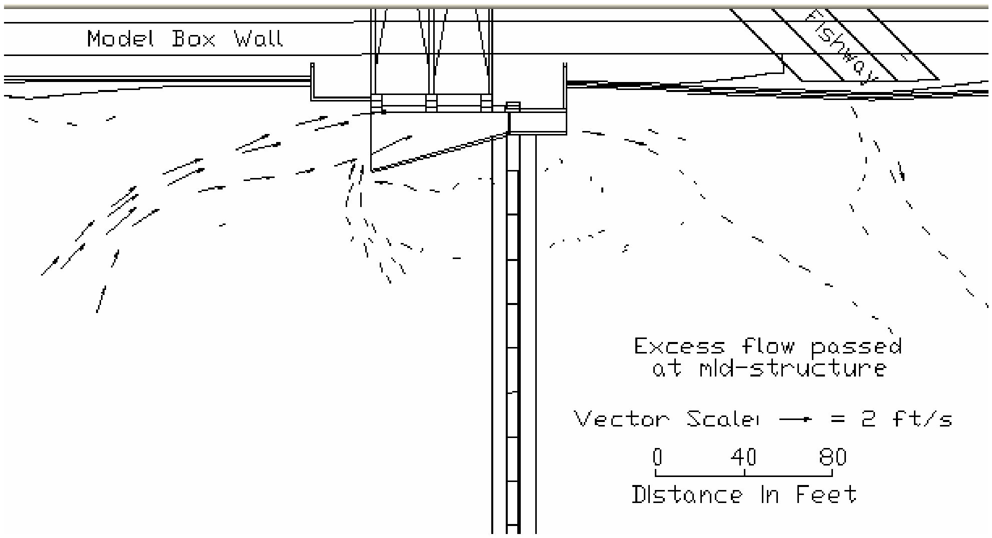
A digital video camera positioned directly overhead approximately 20 ft above the model was used to document surface velocity field near the diversion off-take and near the entrance (downstream end) of the fishway. Video was made of 0.75 in. diameter Styrofoam objects floating in the channel. A 4 ft X 4 ft grid of nylon cord was suspended approximately 8 inches above the channel bed to serve as a geo-referencing grid for analysis of video frames using Autocad software. Using this technique, surface velocity vectors were produced for the flow field in the vicinity of the diversion and fishway entrance. Figures 11-13 are sketches produced in Autocad that show the near-field surface velocity vectors for a 1000 ft^3/s river discharge with excess flow passed at river left (near the diversion and fishway), near mid channel, and at river right, respectively.

Figure 11



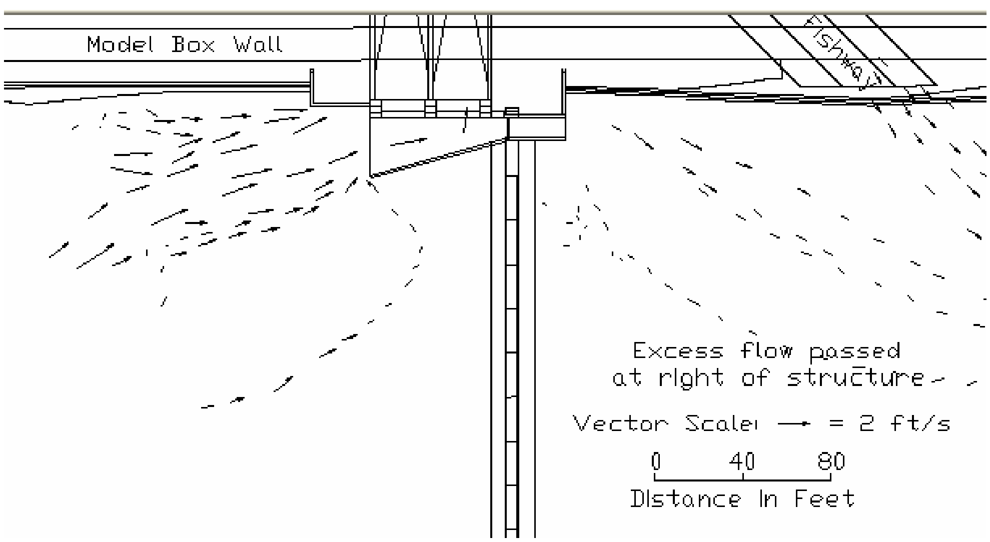
Surface velocity vector map for the velocity field along the left bank of the Rio Grande near the proposed diversion off-take and the fishway entrance. River Discharge = 1000 ft^3/s, diversion discharge = 130 ft^3/s, excess flow passed at river left. Shown in the drawing is approximately 190 ft stream width nearest the left bank (of a total structure width of approximately 590 ft).

Figure 12



Surface velocity vector map for the velocity field along the left bank of the Rio Grande near the proposed diversion off-take and the fishway entrance. River Discharge = 1000 ft^3/s, diversion discharge = 130 ft^3/s, excess flow passed near mid-channel. Shown in the drawing is approximately 190 ft stream width nearest the left bank (of a total structure width of approximately 590 ft). The section of the structure where excess flow is being passed is not shown.

Figure 13



Surface velocity vector map for the velocity field along the left bank of the Rio Grande near the proposed diversion off-take and the fishway entrance. River Discharge = 1000 ft^3/s, diversion discharge = 130 ft^3/s, excess flow passed at river right. Shown in the drawing is approximately 190 ft stream width nearest the left bank (of a total structure width of approximately 590 ft). The section of the structure where excess flow is being passed is not shown.

Significant bed scour was observed in the model immediately downstream from the diversion structure where fine sand had initially been placed. Concentration of flow passing through the limited gate openings in each of the three gate operating scenarios for passing excess flow produced localized bed scour. For the nature bed material present in this reach of the Rio Grande, attempting to prevent bed scour may be more difficult than designing the diversion structure with adequate toe protection (i.e. cutoff wall) to eliminate potential problems due to bed scour.

At the completion of the flow-field tests, representatives of URS-Denver visited the lab to view the model in operation and view demonstration of the three gate operating scenarios. As seen displayed during this visit, passage of excess flows at river left resulted in comparatively strong attraction flows near the fishway entrance, but also resulted in diversion of a significant amount of bed load sediments. When excess flow was passed near mid channel, flow near the fishway entrance was noticeably more tranquil and diversion of bed load sediments significantly diminished. Passing excess flow at river right further diminished attraction flow near the fishway entrance while possibly offering improved reduction in sediment diversion compared with passing excess flow at mid-channel.

An additional observation brought to the attention of URS personnel was that for each of the scenarios tested, it questionable whether a discharge of 65 ft^3/s in each diversion bay could be attained. Overshot crest gates had been designed for installation at the mouth of each diversion bay, atop an 18" sediment exclusion sill. Planned operation called for these gates to be raised as high as possible – thus allowing the least sediment-laden water near the surface to be diverted. In the model tests, these crest gates had to be fully lowered in order to approach the normal diversion of 130 ft^3/s (65 ft^3/s per bay). This, despite the fact that trash rack and fish screen losses were not being modeled.

At the conclusion of the visit, URS personnel expressed a desire to focus on means of improving diversion operations while passing excess flows at river left – the scenario most conducive to strong fishway attraction flows and maintenance of the channel thalweg near the diversion mouth. They requested a series of tests to investigate means of reducing sediment diversion for this flow passage scenario. While a limited body of knowledge is available regarding preferences of some of the species that are targeted for fishway usage and hence desirable magnitude for attraction flows is unknown, identification of viable means of limiting sediment diversion would provide maximum flexibility in operational abilities.

The suggested test criteria was to continue testing with a stream discharge of 1000 ft^3/s. With the various types of sediment exclusion equipment installed, the model would be operated for a specified length of time. At the conclusion of each test, the diversion bays would be flushed and a “wet” weight of diverted sediments would be measured. From these tests, a comparative evaluation of sediment exclusion performance could be obtained.

Sediment Exclusion Tests

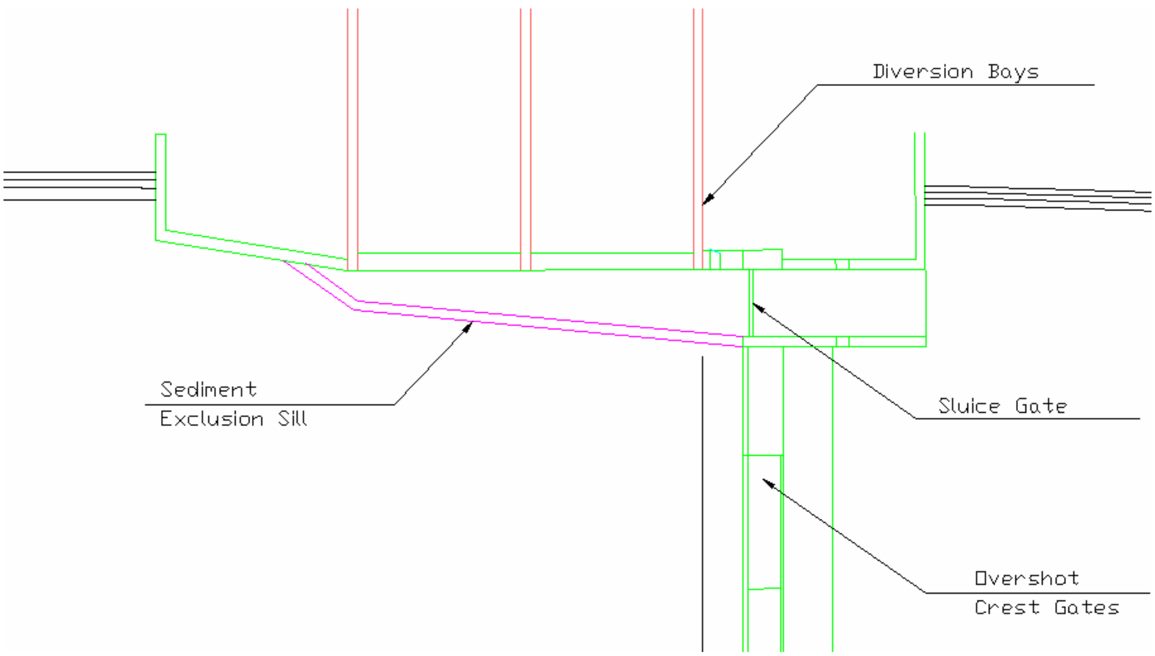
As discussed URS personnel, tests would be conducted both with the wing wall that extends upstream from the sluiceway in front of the diversion mouth in place and with the wing wall removed. Sediment exclusion mechanisms for inclusion in these tests were selected after reviewing literature including: Melone, 1975; Odgaard, 1990; and Vanoni, (1975). Structures included for testing include:

- A sediment exclusion sill (for use without the wing wall)
- Straight vanes placed on the channel invert angled outward from the bank in the downstream direction
- Long, curved vanes placed on the channel invert that initially parallel the bank and curve outward from the bank in the downstream direction
- “Hanging” vanes mounted a distance above the invert and angled outward from the bank in the upstream direction
- “Iowa” vanes – a system of short vanes angled outward from the bank in the downstream direction. [This vane system has been studied and refined by the University of Iowa. A U.S. Patent has been awarded for a double curve vane shape developed at the University. Vanes used in the model study were a generic flat vane.]

Sediment Exclusion Sill

This structure was perceived to be relatively low cost and easily constructed. It was anticipated that the sill might perform as a primary bed load exclusion structure by preventing the more sediment laden flow near the invert from entering the area immediately in front of the diversion bays. A similar sill already part of the design for the mouth of the diversion would act as a secondary bed load exclusion structure. Sediments excluded by the primary sill would be routed around the sluiceway, thus installation of this structure required removal of the wing wall. The configuration tested was rectangular, 1.5 ft high (half the design flow depth) and 1.5 ft wide. Figure 14 is a sketch showing the layout of the sediment exclusion sill.

Figure 14

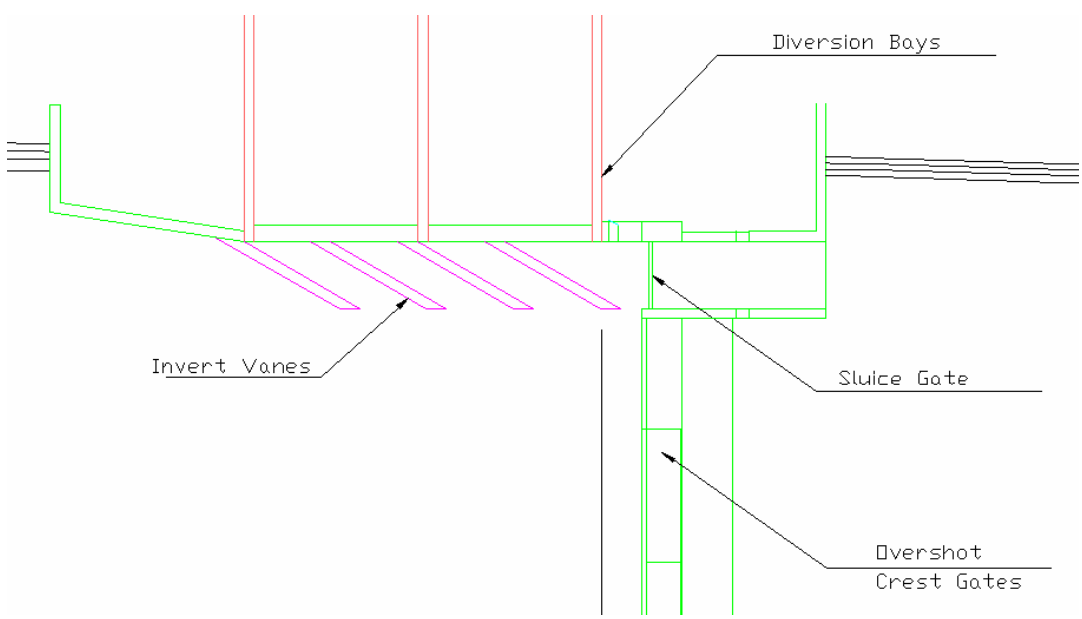


Shown in the sketch is the plan layout of the tested sediment exclusion sill. Sill was 1.5 ft high X 1.5 ft wide and was installed with the sluice wing wall removed. [Flow in the Rio Grande is left to right, direction of flow in the intake bays is toward the top of the page.]

Straight Vanes on Channel Invert

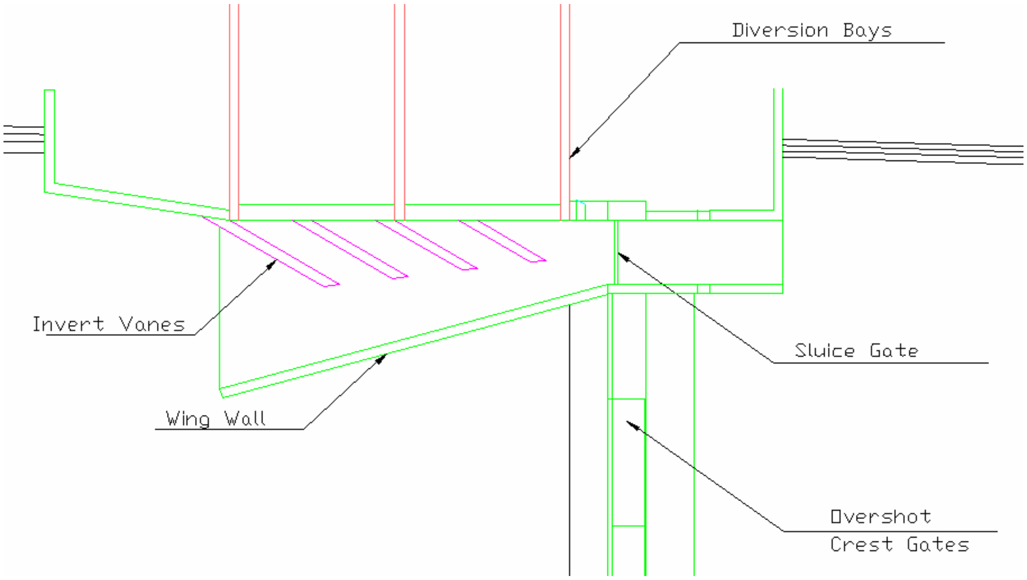
Four vanes angled at 30 degrees to the bank in the downstream direction were installed. Three vane heights were tested with no wing wall, 1.5 ft, 1.0 ft and 0.75 ft. For the tests with no wing wall all vanes were approximately 20 ft long in order to extend to a distance outward from the bank equal to the width of the sluice (10 ft). The 1.5 ft vanes were square in cross section. Both the 1.0 ft and the 0.75 ft vanes featured a rounded top downstream edge in an effort to diminish the impacts of flow separation seen with the 1.5 ft square vanes. Performance of the respective vanes is discussed in greater detail below in the *Results* section. The 1.0 ft vanes were also tested with the wing wall in place. For this test each vane in the downstream direction was shortened to maintain a proportional distance between the end of each vane and the wing wall. Figure 15 shows vane layout with no wing wall. Figure 16 shows vane layout with the wing wall in place. Cross sections of the three types of invert vanes tested are shown in Figure 17.

Figure 15



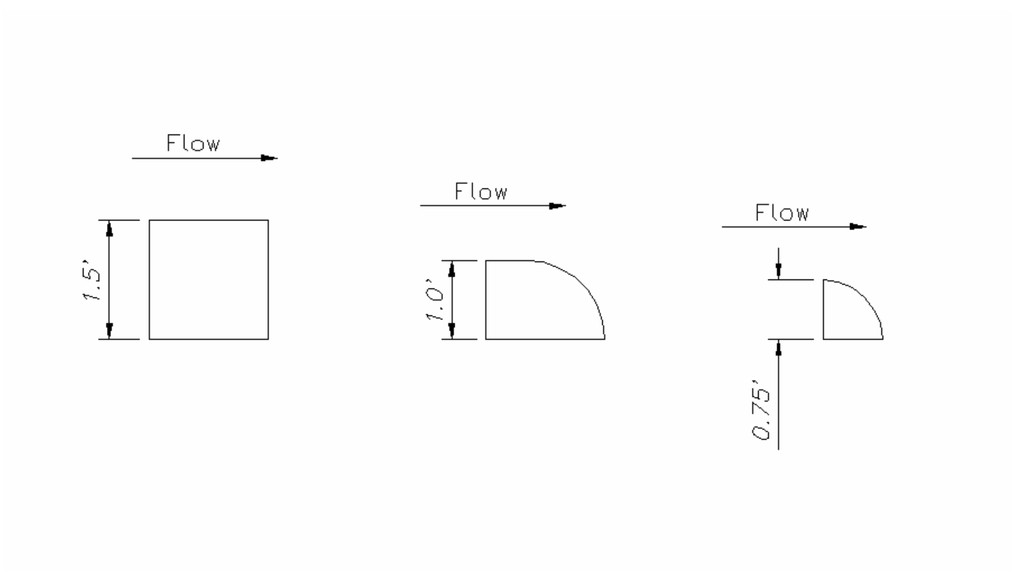
Plan layout of straight vanes on the channel invert with wing wall removed. Three vane sizes were tested.

Figure 16



Layout of straight bottom vanes with the wing wall installed. Vane lengths vary from approximately 20 ft to approximately 13 ft. The 1.0 ft vanes were tested in this configuration.

Figure 17

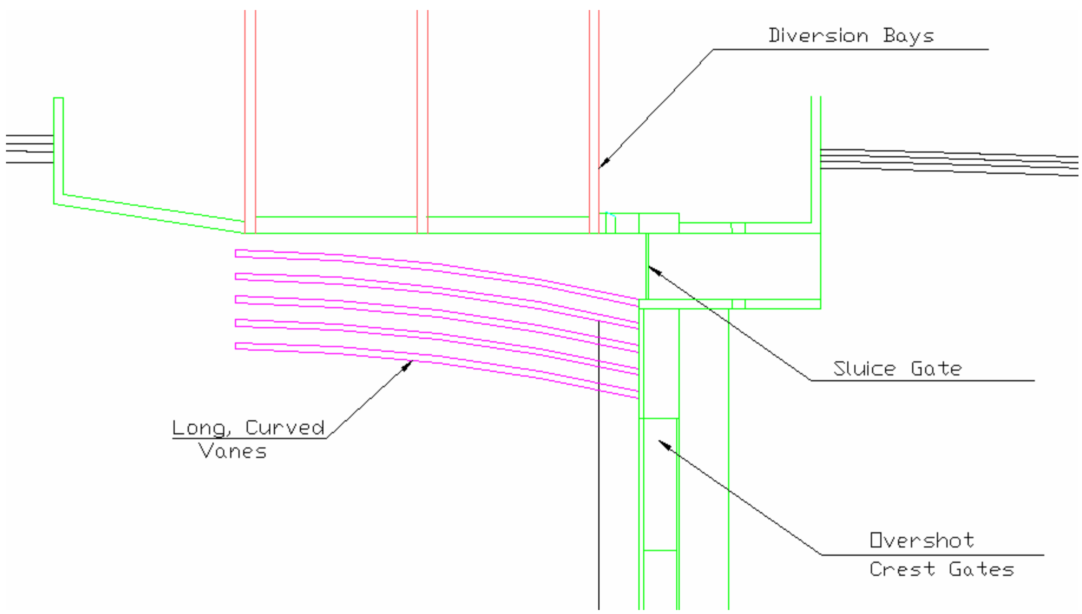


Cross sections of the three types of invert vanes tested. The 1.5 ft vane was tested first. Separation of flow as it passed over the square downstream edge of this vane style prompted use of the rounded downstream edge shown for the 1.0 and 0.75 ft vanes.

Long, Curved Vanes

A configuration of five long, curved vanes 1 ft high X 1 ft wide were tested with the wing wall removed. The vanes were approximately 61.5 ft long with a radius of curvature of 290 ft. Placement was such that the vane nearest the bank was approximately 2.5 feet out from the front of the diversion on the upstream end and in line with the inner sluiceway wall at the downstream end. Vanes were spaced 3.5 ft on center. The left (~ downstream) upper edge of the tested vanes was rounded at a radius of 0.5 ft. Layout for the long, curved vanes is shown in Figure 18.

Figure 18

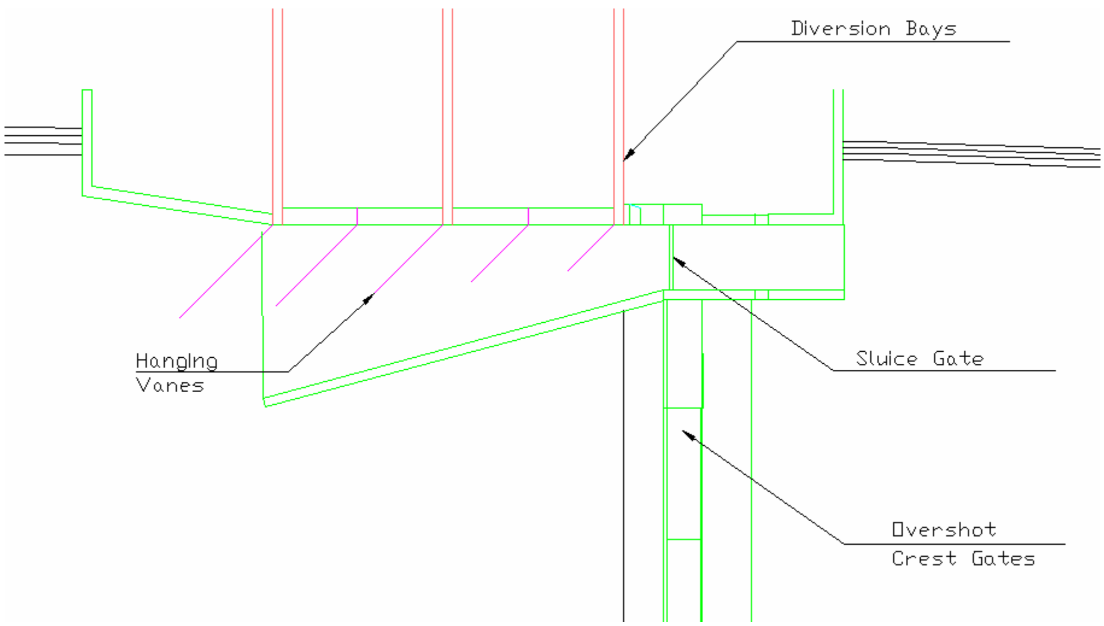


Layout of the long curved vanes attached to the channel invert. Tested vanes were 1.0 ft tall and spaced 3.5 ft on center.

Hanging Vanes

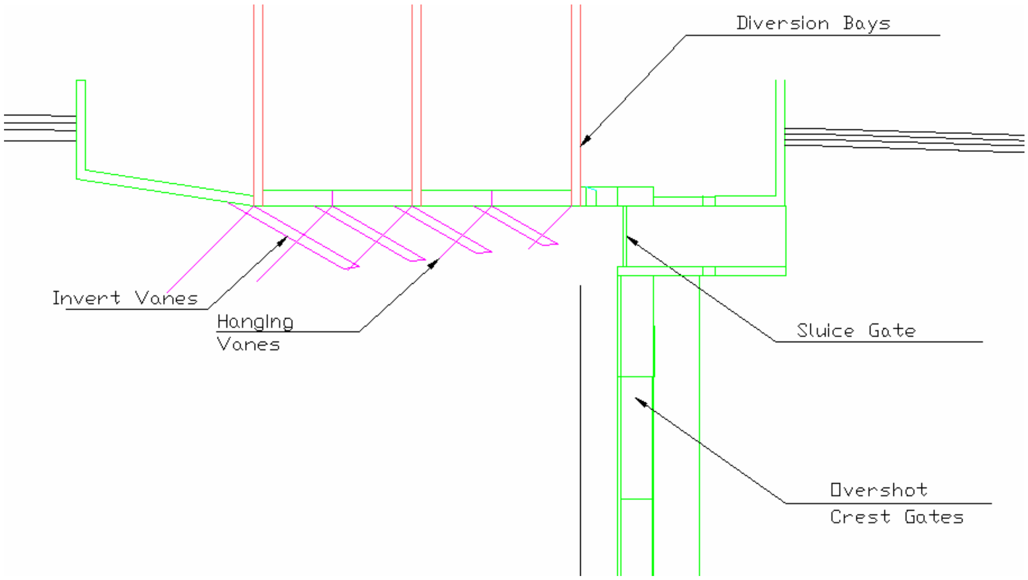
Hanging sediment exclusion vanes were fabricated and installed such that they angled 45 degrees outward from the bank in the upstream direction. The vanes were suspended 1.5 ft above the channel invert. Five vanes were spaced to align with the walls and mid span of the two diversion bays. Length was varied from approximately 20 ft for the furthest upstream vane to approximately 10 ft for the furthest downstream in order to maintain a proportional distance between the vanes and the wing wall. This hanging vane configuration was tested both with and without the wing wall. Two tests were performed without the wing wall with both hanging vanes and 1.0 ft high invert vanes. Figure 19 shows the layout of the hanging vanes with wing wall installed. Figure 20 shows the layout of the combination of hanging vanes and 1.0 ft invert vanes.

Figure 19



Layout for the hanging vanes angled 45 degrees upstream and suspended 1.5 ft above the channel invert. (Shown with wing wall in place)

Figure 20

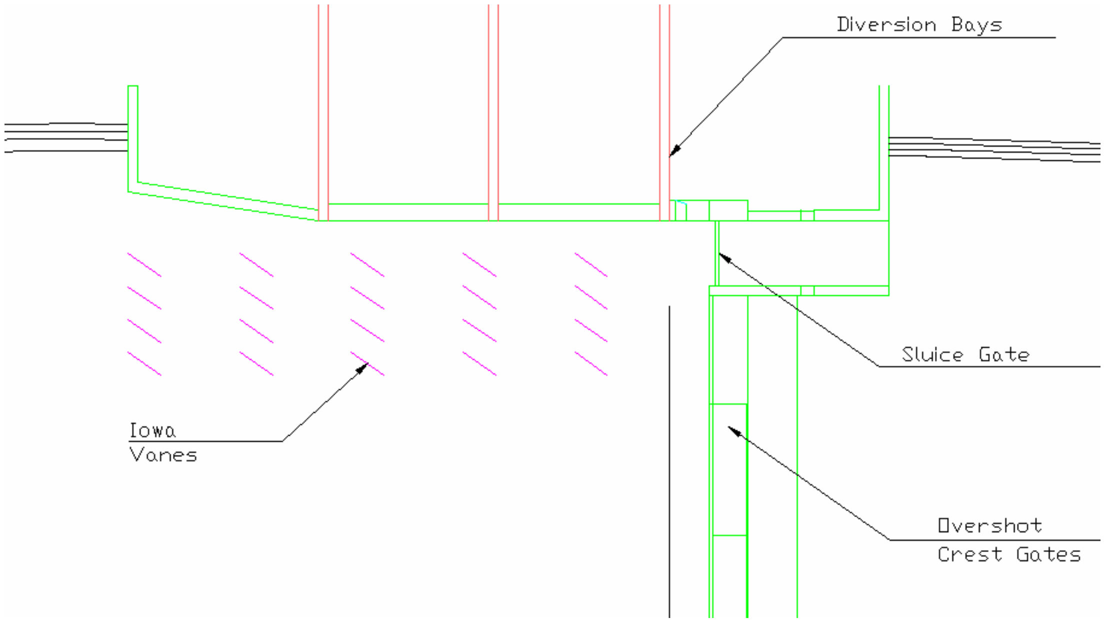


Layout for the tests combining hanging and 1.0 ft invert vanes. This layout was altered slightly for a second test by shifting the invert vanes downstream such that points where the leading (upstream) edge of the invert vanes meet the front of the diversion bay coincided with the respective points where hanging vanes meet the front of the diversion.

Iowa Vanes

Iowa vanes tested were approximately 6 ft long and 1.5 ft high. Vanes were arranged in four rows outward from the bank and in five groups moving upstream to downstream. The vanes were angled at 35 degrees in the downstream direction with respect to the bank. Moving outward from the front of the diversion bays, the upstream end of the nearest row of vanes is spaced 5 ft from the leading edge of the sill at the diversion mouth. Successive rows are space 5 ft on center. The downstream edge of the furthest downstream group of vanes is spaced 9.5 ft relative to the upstream edge of the concrete structure spanning the river. Successive groups of vanes moving in the upstream direction are spaced 17 ft on center. Figure 21 shows the layout of the tested configuration of Iowa Vanes.

Figure 21

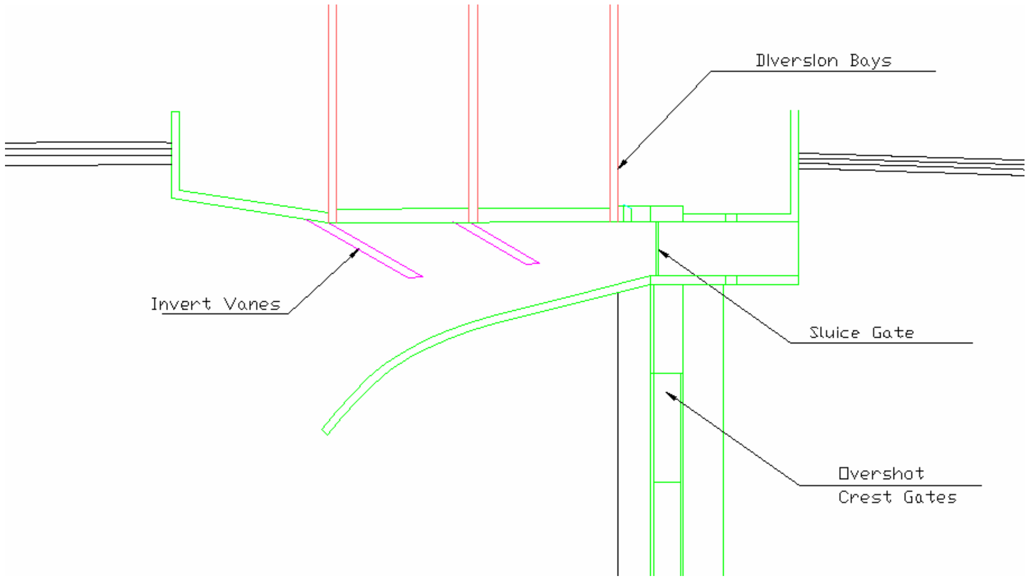


Layout for the tested configuration of Iowa Vanes. Vanes were arranged in four rows moving outward from the bank and in five rows moving upstream to downstream. Vanes were angled at 35 degrees with respect to the front of the diversion bays.

Additional tested configuration

At the request of URS Denver personnel a configuration sketched by Frank Lan and transmitted via FAX featuring a curved wing wall plus two invert vanes was tested in the model. Layout of this configuration is shown in Figure 22.

Figure 22



Layout for tests with modified curved wing wall. Two 1.0 ft invert vanes were tested in conjunction with the curved wing wall.

Results of comparative evaluations of sediment exclusion mechanisms

Sixteen tests with the sediment exclusion systems described and diagramed above, plus tests with no sediment exclusion systems in place were and documented by video taping and by obtaining a wet weight of sediment diverted into the diversion bays over a 30 minute run period. As noted above, each of the tests was conducted with a 1000 ft^3/s river discharge, passage of excess flow at river left and an upstream water surface elevation of 4995. The sluice gate opening was 0.5 ft for all tests.

Targeted diversion rate was 65 ft^3/s per diversion bay for a total diversion of130 ft^3/s. For some of the tests, this targeted rate of diversion could not be achieved. An edited copy of the test video accompanies this report. Measured data and summarized observation notes for each of the tests are as follows. Tests are numbered in the order they were performed.

Sediment Exclusion Test 1 – No sediment exclusion system, wing wall in place.

Wet weight of diverted sediments 92.4 g

Left diversion Q 65 ft^3/s

Right diversion Q 65 ft^3/s

Noted observations: A separation of flow from left wall at the mouth of both diversion bays sets up a counter-clockwise eddy in each bay. (This was also observed in subsequent tests.) [Direction of flow in the Rio Grande in this and in subsequent photos is left to right.]

Sediment Exclusion Test 2 – No sediment exclusion system, no wing wall.

Wet weight of diverted sediments 731.3 g

Left diversion Q 65 ft^3/s

Right diversion Q 65 ft^3/s

Noted observations: none

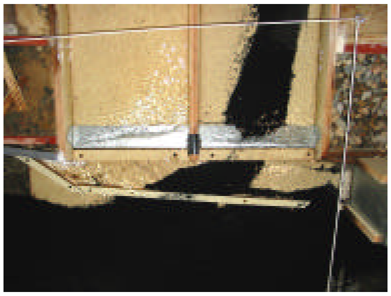
Sediment Exclusion Test 3 – 1.5 ft sediment sill, no wing wall.

Wet weight of diverted sediments 903.1 g

Left diversion Q 51.6 ft^3/s

Right diversion Q 65 ft^3/s

Noted observations: Unable to get full targeted diversion in left (upstream bay).



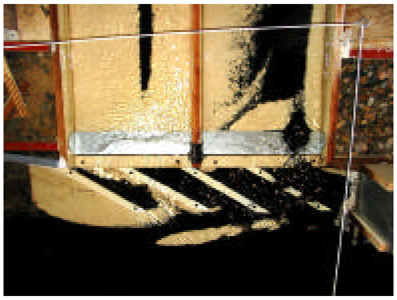
Sediment Exclusion Test 4 – 1.5 ft (square cross section) invert vanes, no wing wall.

Wet weight of diverted sediments 1574.4 g

Left diversion Q 48 ft^3/s

Right diversion Q 65 ft^3/s

Noted observations: Separation of flow passing over the sharp downstream edge of the vanes creates a low pressure zone that causes bed load sediments to be pulled around the end of



the vanes and back toward the diversion bays. Diversion in left bay is just over 70% of target. Rounding downstream edge of vanes to approximate an ogee crest may diminish the low pressure zone behind the vanes.

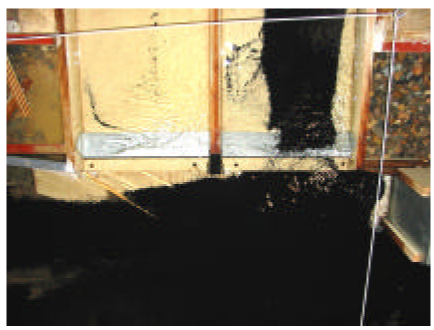
Sediment Exclusion Test 5 – 1.0 ft (rounded downstream edge) invert vanes, no wing wall.

Wet weight of diverted sediments	779.6 g
Left diversion Q	65 ft^3/s
Right diversion Q	65 ft^3/s

Noted observations: A significant reduction in sediment diversion compared with the previous test. The ability to divert is also improved. When diversion flow was stopped the vanes effectively self-cleaned.

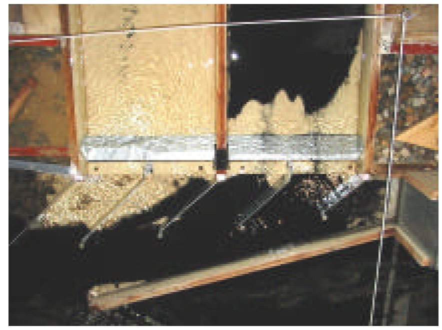
Sediment Exclusion Test 6 – 0.75 ft (rounded downstream edge) invert vanes, no wing wall.

Wet weight of diverted sediments	4189.5 g
Left diversion Q	60.3 ft^3/s
Right diversion Q	65 ft^3/s



Noted observations: A comparatively large amount of sediment was diverted. These vanes appeared to be too small in relation to depth of flow and were soon covered over by sediment. The reason for the drop in left bay diversion is not readily apparent. In reviewing previous test results and from transport in the model, it appears model performance may be in a dynamic state. Rate of deposition upstream from the diversion may be tapering off, resulting in increased transport near the diversion.

Sediment Exclusion Test 7 – Hanging vanes 1.5 ft above invert, wing wall in place.



Wet weight of diverted sediments 2498.3 g

Left diversion Q 57 ft^3/s

Right diversion Q 65 ft^3/s

Noted observations: Sediment diversion is just over half the previous test, but still comparatively high. The hanging vanes significantly diminished the eddy current observed in the diversion bays. Low left bay diversion persists.

Sediment Exclusion Test 8 – Hanging vanes 1.5 ft above invert, no wing wall.

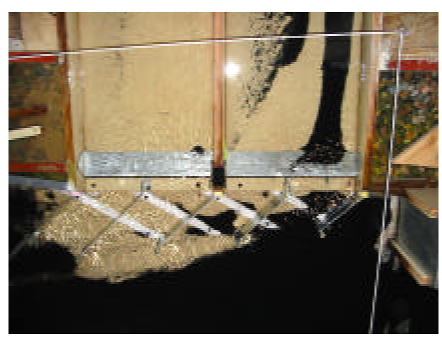
Wet weight of diverted sediments 308.7 g

Left diversion Q 65 ft^3/s

Right diversion Q 65 ft^3/s

Noted observations: A marked improvement in sediment diversion – 12% of the amount diverted with the wing wall in place in the previous test. The targeted diversion was also achieved which was not the case in the previous test.

Sediment Exclusion Test 9 – Hanging vanes 1.5 ft above invert and 1.0 ft invert vanes (rounded downstream edge), no wing wall.



Wet weight of diverted sediments 36.5 g

Left diversion Q 65 ft^3/s

Right diversion Q 65 ft^3/s

Noted observations: This combination of sediment exclusion systems appears quite effective.

Sediment Exclusion Test 10 – Hanging vanes 1.5 ft above invert and 1.0 ft invert vanes (rounded downstream edge), no wing wall.

Wet weight of diverted sediments	29.1 g
Left diversion Q	65 ft^3/s
Right diversion Q	65 ft^3/s

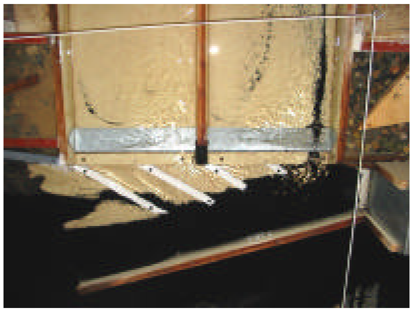
Noted observations: This test is a slight modification from the previous test. The bottom vanes were moved approximately 3 feet in the downstream direction in order to align the points at which the upstream edge of the invert vanes and the hanging vanes meet the leading edge of the sill at the mouth of the diversion bays. Performance looks like an improvement compared with the previous test, but both results are quite good.

Sediment Exclusion Test 11 – No sediment exclusion system, wing wall in place.

Wet weight of diverted sediment	1012.4 g
Left diversion Q	65 ft^3/s
Right diversion Q	65 ft^3/s

Noted observations: This is a repeat of the setup from Test 1. Sediment diversion for this test is almost 11 times the amount diverted in test 1 which confirms the suspicions noted after Test 6 regarding the dynamic state of sediment transport in the model. Ramifications of this dynamic behavior will be reviewed at the conclusion of this series of tests.

Sediment Exclusion Test 12 – 1.0 ft (rounded downstream edge) invert vanes, wing wall in place.



Wet weight of diverted sediments	238.5 g
Left diversion Q	65 ft^3/s
Right diversion Q	65 ft^3/s

Noted observations: This test configuration is shown above in Figure 16. The positive impact of the bottom vanes appears to be significant when sediment diversion is compared with Test 11.

Sediment Exclusion Test 13 – 1.0 ft long curved vanes, no wing wall.

Wet weight of diverted sediments	90.3 g
Left diversion Q	65 ft^3/s
Right diversion Q	65 ft^3/s

Noted observations: This sediment exclusion appears to more effective than any yet tested except for the combination of hanging vanes and 1.0 ft invert vanes.

Sediment Exclusion Test 14 – 1.0 ft long curved vanes, no wing wall.

Wet weight of diverted sediments	11.4 g
Left diversion Q	59.3 ft^3/s
Right diversion Q	65 ft^3/s

Noted observations: Test 14 is a repeat of Test 13. After Test 13 had been completed, a colleague working in the lab inadvertently sent a high flow through the model that was intended for a different model. This high flow flushed much of the sediment accumulated in the upper reach of the model through to the settling basins. Prior to the Test 14 run, the model was run for 2 hours with sediment feeding in an attempt to replenish upstream sediment deposits. The significant drop in sediment diversion between Tests 13 and 14 again points out the dynamic behavior of the model previously observed. The reduced ability to divert flow in the left bay also suggests a significant degree of sensitivity to existing bed forms and associated current patterns.

Sediment Exclusion Test 15 – Two 1.0 ft (rounded downstream edge) invert vanes, curved wing wall in place.

Wet weight of diverted sediments	4989.5 g
Left diversion Q	42 ft^3/s
Right diversion Q	65 ft^3/s

Noted observations: The increased upstream wing wall width of this configuration appeared to act as a funnel pulling in bed load sediments. This was the highest diverted sediment magnitude and the most limiting scenario for diversion of flow into the left bay of all tested configurations.

Sediment Exclusion Test 16 – Iowa Vanes – five clusters of four rows of vanes 1.5 ft high and 6.0 ft long oriented at 35 degrees with the front of the diversion bays

Wet weight of diverted sediments	3.4 g
Left diversion Q	65 ft^3/s
Right diversion Q	65 ft^3/s

Noted observations: The effectiveness of the Iowa vanes in keeping bed load sediments away from the diversion was impressive.

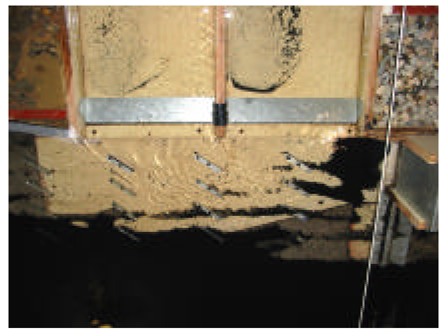
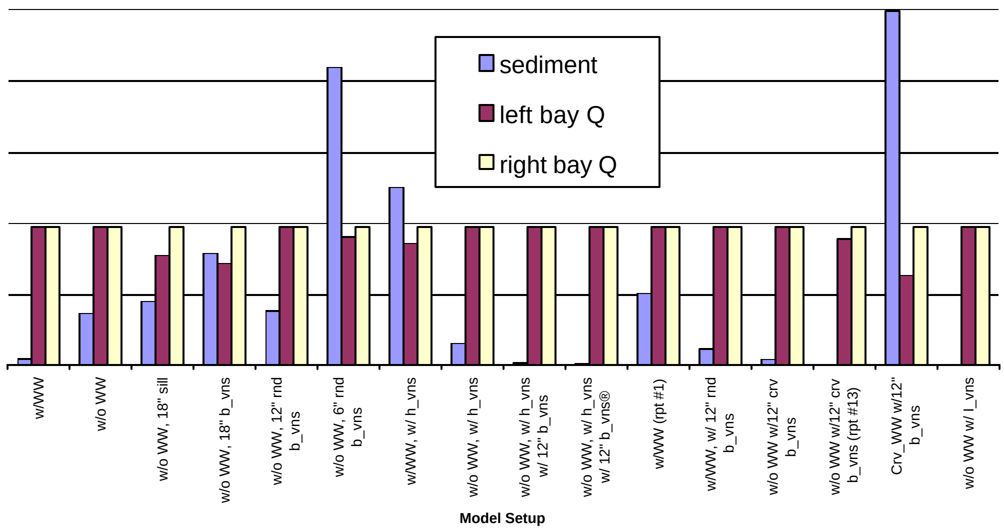


Figure 23 is a plot showing the comparative sediment diversion as well as the comparative ability to divert flow for the sixteen sediment exclusion tests.

Figure 23

Albuquerque Sediment Model Comparative Sediment Diversions and Diversion Discharges



Shown in this plot are comparative rates of sediment diversion and comparative ability to divert flow. High rates of sediment diversion (blue bars) are undesirable. For all tests shown, the target flow diversion of 65 ft$_3$/s was observed in the right bay (yellow bars) while the 65 ft^3/s target was not achieved in some cases in the left bay (maroon bars).

Evaluation of Initial Sediment Exclusion Testing

Results of the sixteen sediment exclusion tests were reviewed with URS-Denver personnel. Despite the impacts of dynamic behavior of the model, data derived from the tests – including the recorded video – provided clear indications of which sediment exclusion systems offered the greatest promise. Information derived from these tests also brought into focus other issues of concern.

Sediment exclusion systems that exhibited varying degrees of promise included the Iowa vane, the long-curved vanes, the 1.0 ft invert vanes with rounded downstream edge and the hanging vanes – particularly in combination with the 1.0 ft invert vanes. Despite positive results seen with the hanging vanes, the potentially high cost of construction, potential maintenance problems along with safety concerns associated led to a decision to drop this system from further consideration.

The wing wall was not compatible with either the long, curved vanes or the Iowa vanes and the results from tests including the wing wall provided evidence of negative impact on both on sediment exclusion and the ability to divert flow. It was decided that further testing would not include a wing wall.

Sediment Exclusion Testing Modifications and Modified Test Results

After viewing video and summary data from the completed sediment exclusion tests, URS representatives requested two modifications for subsequent tests. First, it appeared that the sluiceway might be negatively impacting both sediment exclusion and the ability to divert flow. They suggested a limited series of tests be conducted with the sluiceway removed and replaced by an additional span of overshot crest gate.

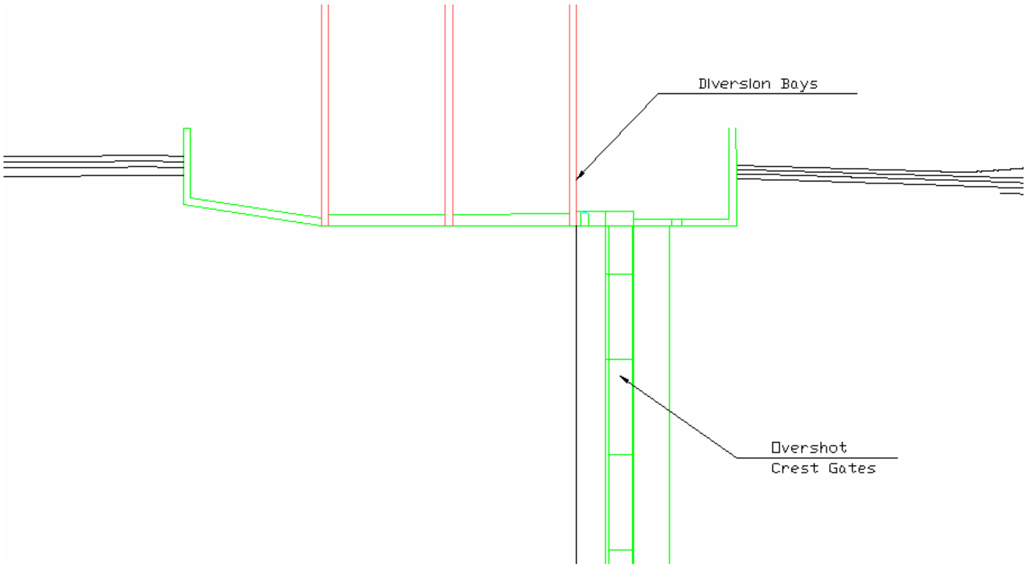
Next, to address the problems seen in the ability to divert the targeted flow rate, the URS staff indicated a desire to look at rotating the diversion bays 45 degrees. This would reduce the amount of direction change needed for flow entering the diversion bays. If the mouth of the diversion bays remained on the same line as that for the 90 degree orientation, the opening into each diversion bay would be lengthened by a factor of $\sqrt{2}$.

The follow up test program suggested by URS consisted of two series of four tests each. First the sluiceway would be removed and replaced by an additional overshot crest gate. In this configuration sediment diversion tests using the regimen employed for previous tests would be conducted with:

- No sediment exclusion system
- 1.0 ft invert vanes (with rounded downstream edge)
- long-curved vanes
- Iowa Vanes

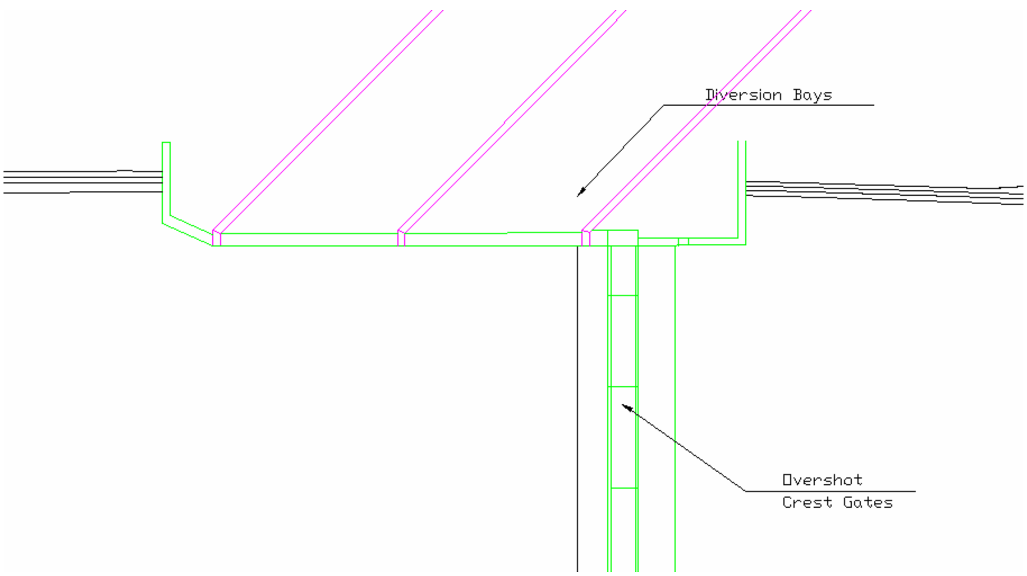
At the conclusion of these tests the diversion bays would be rotated 45 degrees and tests repeated for the same four scenarios. Figures 25 and 25 show the plan layout for tests first with the sluiceway removed and second with sluice way removed and diversion bays rotated.

Figure 24



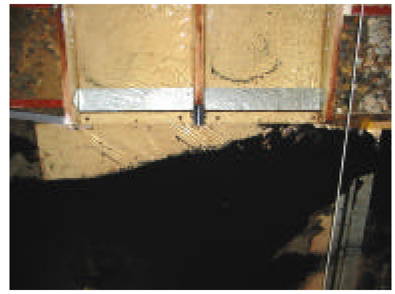
Layout with sluice gate removed and replaced by an additional span of overshot crest gate

Figure 25



Layout with sluice gate removed and diversion bays rotated 45 degrees

Sediment Exclusion Test 17 – Sluice removed, 90 degree diversion, no sediment exclusion system



Wet weight of diverted sediments	23.3 g
Left diversion Q	65 ft^3/s
Right diversion Q	65 ft^3/s

Noted observations: Removal of the sluice appears to be an improvement with regard to reduced sediment diversion

Sediment Exclusion Test 18 – Sluice removed, 90 degree diversion, 1.0 ft invert vanes

Wet weight of diverted sediments	9.1 g
Left diversion Q	65 ft^3/s
Right diversion Q	65 ft^3/s

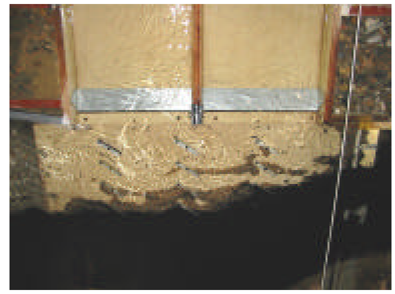
Noted observations: The 1.0 ft invert vanes are visibily effective at moving transport of the bulk of bed-load sediments somewhat farther out into the channel compared with no exclusion system in the previous test.

Sediment Exclusion Test 19 – Sluice removed, 90 degree diversion; long, curved vanes

Wet weight of diverted sediments	3.6 g
Left diversion Q	65 ft^3/s
Right diversion Q	65 ft^3/s

Noted observations: Despite the fact that the vanes become partially covered (these vanes became similarly covered during Test 13) they are highly effective.

Sediment Exclusion Test 20 – Sluice removed, 90 degree diversion, Iowa Vanes



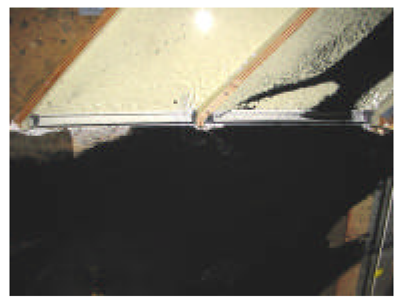
Wet weight of diverted sediments 0.6 g

Left diversion Q 65 ft^3/s

Right diversion Q 65 ft^3/s

Noted observations: The Iowa vanes are again superior to other systems tested at moving bed load transport away from the edge of the channel.

Sediment Exclusion Test 21 – Sluice removed, 45 degree diversion, no sediment exclusion system



Wet weight of diverted sediments 1253.7 g

Left diversion Q 65 ft^3/s

Right diversion Q 65 ft^3/s

Noted observations: The angled diversion with the associated wider diversion opening provides enhanced diversion capability and diminished recirculation in the diversion bays. Sediment diversion is increased significantly.

Sediment Exclusion Test 22 – Sluice removed, 45 degree diversion, 1.0 ft invert vanes

Wet weight of diverted sediments 113.7 g

Left diversion Q 65 ft^3/s

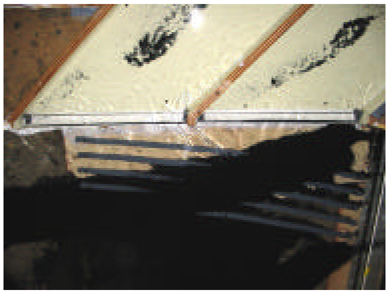
Right diversion Q 65 ft^3/s

Noted observations: The 1.0 ft invert vanes functioned effectively to exclude over 90% of the sediments diverted in the Test 21 with no sediment exclusion system.

Sediment Exclusion Test 23 – Sluice removed, 45 degree diversion; long, curved vanes

Wet weight of diverted sediments	12.2 g
Left diversion Q	65 ft^3/s
Right diversion Q	65 ft^3/s

Noted observations: Again, despite becoming partially covered by sediment deposits the curved vanes are highly effective.



Sediment Exclusion Test 24 – Sluice removed, 45 degree diversion, Iowa Vanes

Wet weight of diverted sediments	5.9 g
Left diversion Q	65 ft^3/s
Right diversion Q	65 ft^3/s

Noted observations: Iowa Vanes look very promising as a comparatively simple means of limiting sediment diversion.

Summary of Modified Sediment Exclusion Tests

Figure 26 is a comparison plot of measured sediment diversions for the for conditions of focus (no exclusion system, 1.0 ft invert vanes, long curved vanes and Iowa vanes) for each of the three diversion layouts (with sluiceway and 90^o diversion, without sluiceway and 90^o diversion and without sluiceway and 45^o diversion).

Figure 26

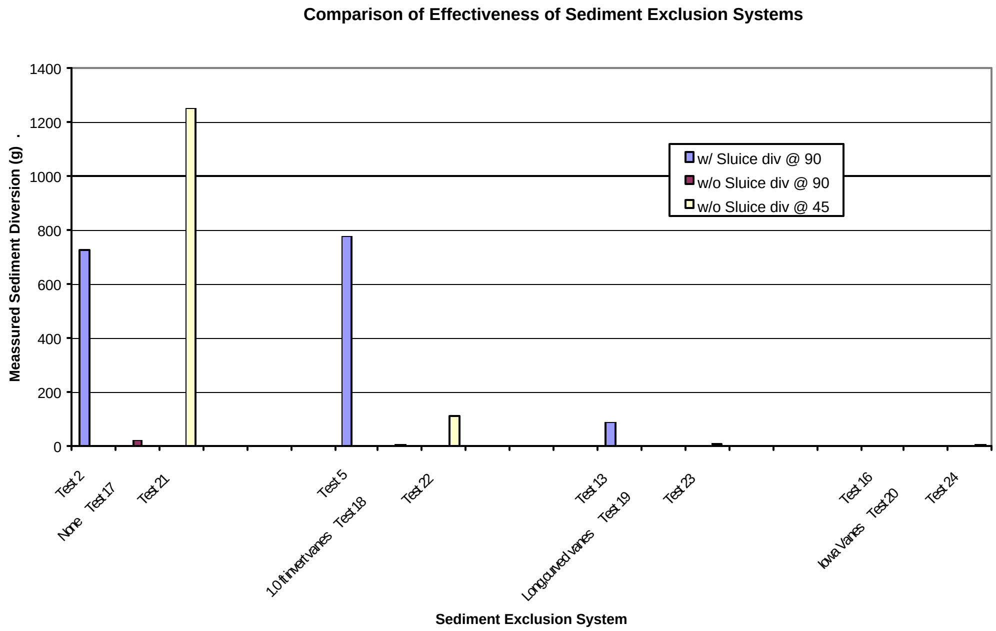


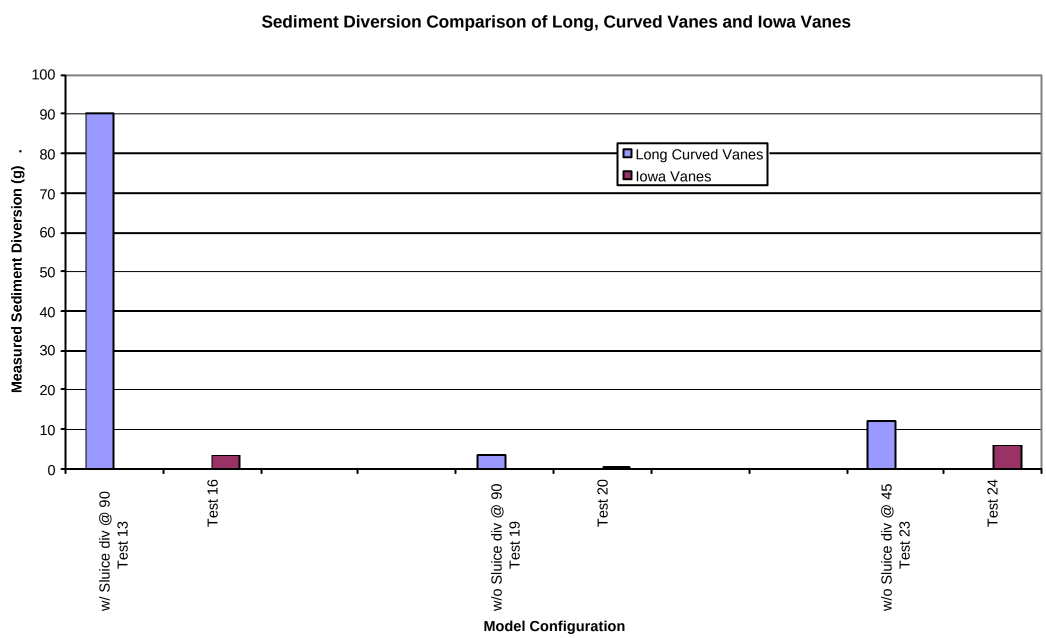
Chart summarizing comparative amounts of bed load sediments diverted for the four selected sediment exclusion systems under three diversion structure configurations.

The sediment exclusion tests involved numerous difficult to manage variables, not the least of which were factors contributing to the dynamic characteristics in the behavior of the model observed and noted above. The poor degree of correlation in sediment diversion measurements from repeat tests performed for two conditions (Tests 1 & 11 and Tests 13 & 14) highlights the limitations of information obtained. As an example, Figure 23 seems to indicate that in the tests with the sluiceway in place, no exclusion system (Test 2 conditions) was more effective than the 1.0 ft invert vanes (Test 5). But as results from the repeat conditions of Tests 1 & 11 show, model performance changed over time for the same conditions.

A means of improving reliability of model output would be to perform multiple tests for each condition of interest. Unfortunately, this would dramatically increase the cost of the study – a cost that would need to be justifiable based on project objectives. For the scope of this project where it was recognized at the outset that only data of a qualitative nature could be produced, a more intensive testing matrix would be justifiable only for focal areas of design interest. The questionable relationship shown in Figure 23 between Tests 2 & 5 is a moot point unless both alternatives (no exclusion system and the 1.0 ft invert vanes) are to be under continued scrutiny as design alternatives.

What can be readily summarized from Figure 26 is that both the long curved vanes and the Iowa vanes demonstrated performance that was clearly better than either no system or the 1.0 ft invert vanes. Figure 27 is a similar chart with a smaller vertical scale that enables a better comparative look at the long curved and the Iowa vanes.

Figure 27



A side-by-side comparison of the two sediment exclusion systems shown to be the most effective out of four alternatives selected for follow up testing

The perspective provided in Figure 27 suggests the Iowa vanes offered a significant degree of performance improvement compared with the long, curved vanes under all model configurations tested. The paired tests performed with the sluiceway removed

(Tests 19 & 20 and Tests 23 & 24) were performed back-to-back which should minimize the impacts of channel dynamics.

Low Discharge Flow Field and Maximum Diversion Tests

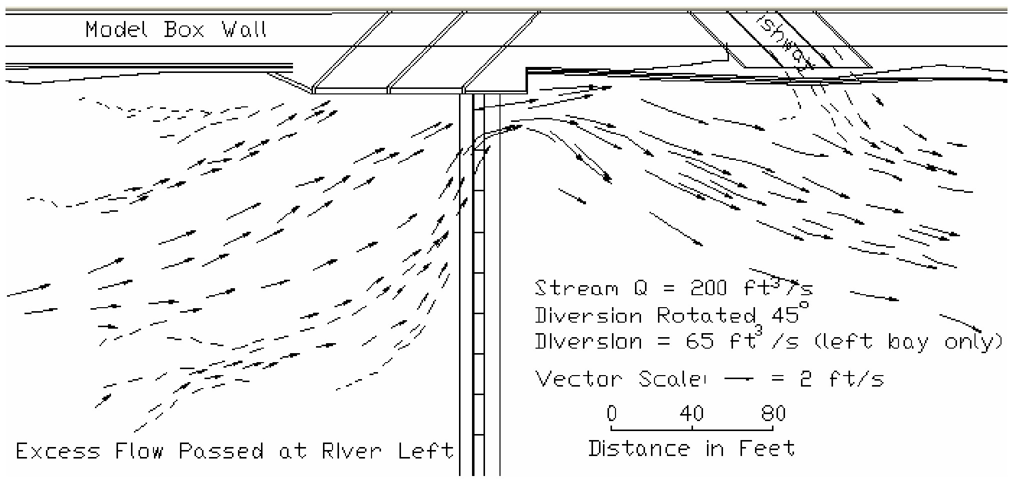
With completion of the sediment diversion testing discussed above, URS – Albuquerque personnel requested that additional testing be performed to:

- Examine flow conditions in the proximity of the diversion and the fishway entrance for the minimum stream discharge at which diversions would be made, (identified as 200 ft^3/s in a communication dated 4/24/04 from Mark Holstad) for the model configuration of *Sediment Exclusion Test 24* (sluiceway removed, diversion rotated 45^o Iowa Vanes in place excess flow passing at river left)
- Determine the maximum discharge that can be diverted with the final sediment test model configuration with 1000 ft^3/s stream flow. [The initial request in the 4/24/04 message from Mark Holstad was to determine first whether 142 ft^3/s and then whether 185 ft^3/s could be diverted. In follow-up discussions it was agreed that a single test identifying the maximum diversion rate would answer questions regarding both flow rates.]

Minimum Diversion Flow Conditions

Digital video was taken from approximately 20 ft above the model for this test using the methodology discussed above for the flow field tests with a 1000 ft^3/s stream discharge. Both Styrofoam objects floating on the surface and dye injected into the stream were utilized to track current patterns and magnitudes. Figure 28 is a sketch produced in Autocad showing surface velocity vectors from this test.

Figure 28



Surface velocity vector map for the velocity field along the left bank of the Rio Grande near the proposed diversion off-take and the fishway entrance. River Discharge = 200 ft^3/s, diversion discharge = 65 ft^3/s (entering only the left diversion bay – diversion bays are rotated 45^o), excess flow passed at river left. Shown in the drawing is approximately 190 ft stream width nearest the left bank (of a total structure width of approximately 590 ft).

Maximum Diversion @ 1000 ft^3/s River Discharge

Maximum Diversion tests were performed for river discharge of 1000 ft^3/s with upstream pool elevation of 4995 ft. Measured diversions in the respective diversion bays were: Left diversion bay, 80.1 ft^3/s; Right diversion bay, 57.3 ft^3/s.

The results of the *Maximum Diversion* test are at odds with observations of previous tests. First, in previous tests with the sluice removed and the diversion at 45^o, no problems had been encountered in diverting at least 65 ft^3/s in each bay. Further, in testing with a 90^o diversion orientation flow entered the right (downstream) bay more readily than the left bay. That behavior is consistently seen in situations where flow is forced to make a sharp direction change entering a diversion. Apparently as a function of both the longer diversion openings and the reduction in amount of direction change, the hydraulics no longer favor flow into the downstream bay. It appears slope of the water surface affected by drawdown effects of excess flow accelerating to pass downstream immediately in front of the diversion mouth may be a large part of the explanation for the fact that now flow now is more readily entering the left bay.

In the time interval between completion of the sediment exclusion tests and reception of the request for the maximum diversion test, Jungseok Ho (University of New Mexico Doctoral candidate who assisted in the model study) independently looked at a series of flow conditions as part of his Dissertation research including a discharge of 3000 ft^3/s for which he fully lowered all the overshot gates across the river. The 3000 ft^3/s test produced a significant degree of channel re-shaping as sediments that had accumulated upstream from gates raised during the extended period of 1000 ft^3/s testing were mobilized.

The inability to divert 65 ft^3/s into the right bay which had been readily achieved in earlier tests suggests that diversion capacity is highly sensitive to existing bed forms and associated current patterns in the flow field near the diversion mouth. It also reinforces a concern as to whether sufficient head will be available for the diversion system to maintain targeted capabilities with a 4995 water surface elevation.

Model Study Summary

The sediment transport scaling criteria employed in structuring this physical scale model study appears to have been a valid design methodology. For the limited scope of verification conducted, sediment transport in the model fell within the range of field observations. Data generated from this effort has made possible a clear focus on appropriate technologies for limiting diversion of bed load sediments, and brings forward critical concerns regarding energy (head) availability. The model study was not able to effectively investigate all aspects of sediment management.

The slope distortion factor of 6.5 used to meet transport scaling criteria significantly diminished the ability to examine aspects of in-channel sediment management in the region upstream from the proposed diversion structure. Because of this slope distortion, the pooled reach upstream from the diversion structure only extended upstream approximately 250 ft. This is a fraction of the stream reach expected to be impacted in prototype thus deposition and/or mobilization of sediments in much of the pooled reach could not be simulated.

In developments highly characteristic of past model studies, the actual performed testing matrix deviated and expanded significantly from the estimated series of tests outlined in the project proposal. As information produced by the model was analyzed, issues surfaced that pointed the client toward new informational objectives. Where a series of 13 tests was initially proposed, approximately three times that number have been completed and documented.

A number of uncertainties persist – particularly regarding the hydraulics of the diversion bays and fish screens. At the 1:24 scale, this study is too small to effectively investigate function of the diversion bays in a quantitative manner . As this report is being prepared, it appears the client is leaning toward addressing these uncertainties using a conservative design approach.

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