PAP 927

Computational Flow Analysis of the Existing Hydraulic Condition of the Spillway and Ejectors at Imperial Diversion Dam

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REVIEW REQUIREMENT

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REVIEW CERTIFICATION

Peer Reviewer - I have reviewed the assigned Items/Section(s) noted for the above document and believe them to be in accordance with the project requirements, standards of the profession, and Reclamation policy.

Reviewer: ____________________ Review Date: 6/2/2005
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Preparer: I have discussed the above document and review requirements with the Peer Reviewer and believe that this review is completed, and that the document will meet the requirements of the project.

Team Member: ____________________ Date: 5-26-04.
Signature

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Computational Flow Analysis of the Existing Hydraulic Condition of the Spillway and Ejectors at Imperial Diversion Dam

INTRODUCTION

Background

The facility

Imperial Diversion Dam is primarily a concrete slab-and-buttress structure on the Colorado River located about 14 miles northeast of Yuma, Arizona[1]. The right (west) side of the dam is in California; the left (east) side of the dam is in Arizona. The dam was constructed between 1936 and 1938 by the Bureau of Reclamation to impound water for irrigation. The reservoir also provides recreational opportunities for boating, camping, and fishing. The original capacity of Imperial Reservoir was 83,000 acre-feet at reservoir water surface elevation 181.0. As anticipated during design, siltation regularly fills most of the reservoir and so periodic dredging is required to maintain sufficient reservoir volume to allow for irrigation releases.

The overall length of Imperial Diversion Dam is about 3,479 feet. The sections of the dam, starting at the right end, are the California abutment, the All American Canal headworks, the sluiceway, the overflow weir section, the Gila Canal headworks, the Arizona abutment, and the Arizona dike, as displayed in Figure 1.

The overflow weir section is a concrete slab-and-buttress structure with an ogee-shaped concrete flow surface on its downstream face and acts as an uncontrolled spillway as displayed in Figure 2. This section of the dam is 1,197.5 feet long, has a crest elevation of 181.00 feet, a bottom elevation of 150.00 feet at the foundation contact, a base width of about 79 feet, and is founded on native material or compacted fill. A concrete apron with top elevation 154.00 feet extends about 170 feet into Imperial Reservoir upstream from the overflow section. The interior of the overflow section is filled with gravel ballast to elevation 161.0 feet to provide stability and an archway is provided through each of the buttresses for personnel access. The discharge capacity of the overflow weir section is approximately 142,000 ft^3/s at reservoir water surface elevation 191.0 according to hydraulic model studies[2].

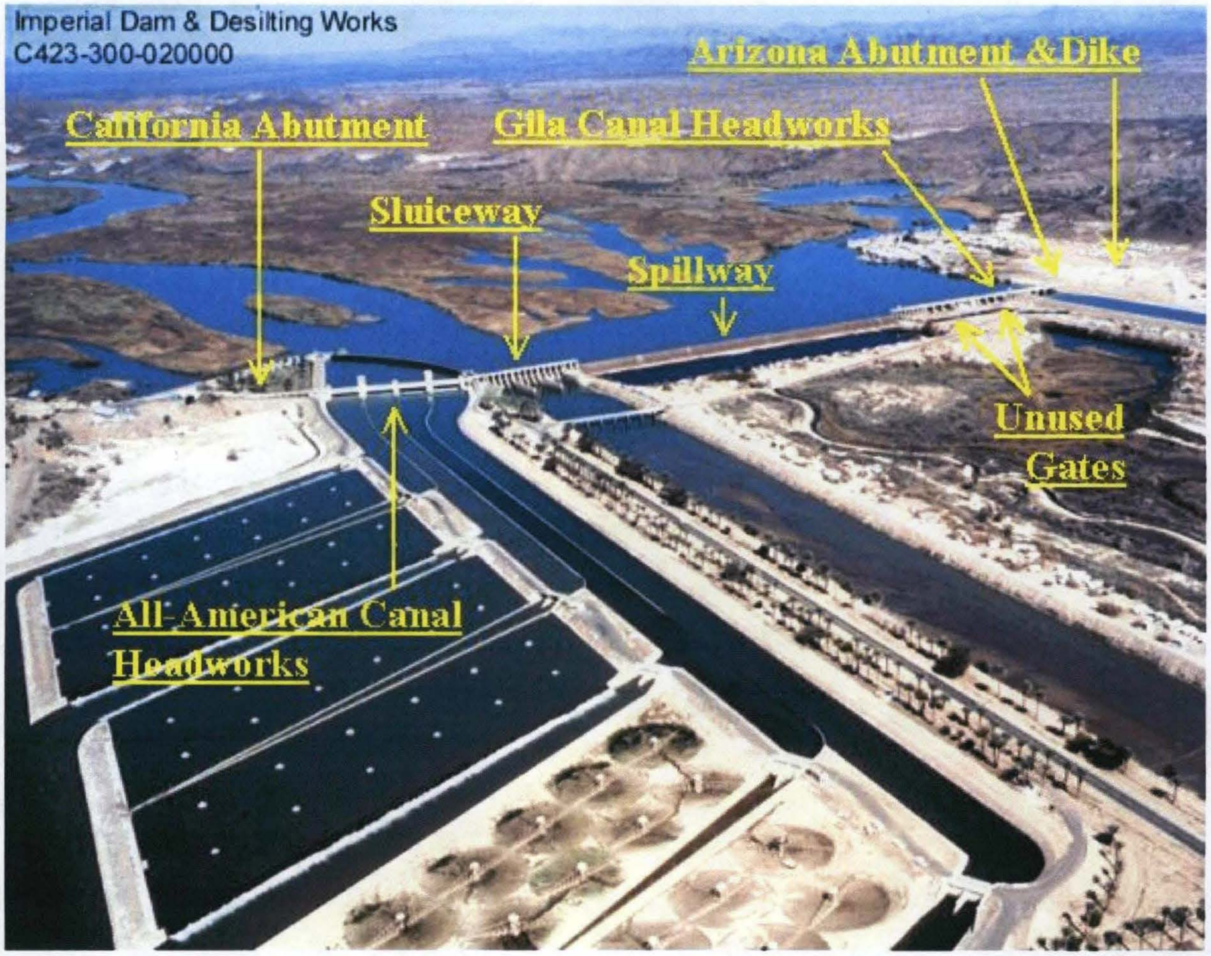

Figure 1. Imperial Diversion Dam and Appurtenances. In part, this study investigates the hydraulic conditions of the spillway and stilling basin.

The overflow weir was constructed in 15 individual units, 14 of which are 78.5 feet long, and one, the center unit, is 98.5 feet long. Each unit abuts the cantilevered ends of the upstream and downstream slab surfaces of the adjacent units. The clear space between buttresses is 17.5 feet, except at the cantilevered ends where the clear space between the end-buttresses of adjacent units is 16.0 feet. Each buttress is 2.5 feet thick. Inside the spillway structure, each 78.5-foot-long unit has 11 vertical cast-iron-pipe foundation drains near its downstream edge; the 98.5-foot-long middle unit has 14 of these drains. Flow ejectors at elevation 159.6 feet drain standing water from inside of the spillway structures as shown in Figure 3.

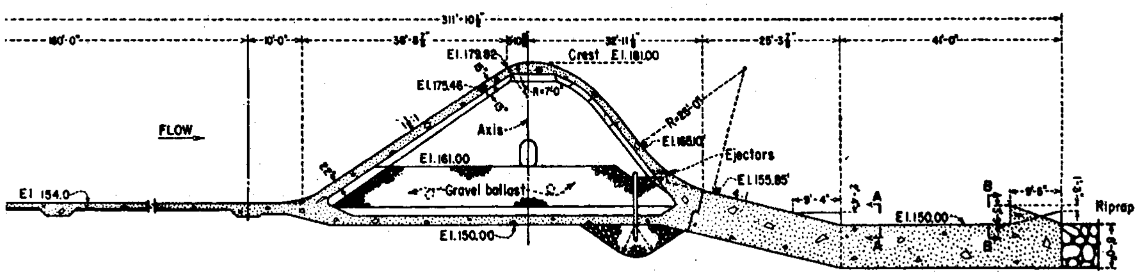

Figure 2. Typical section. The ejectors at elevation 159.6 feet are not displayed on many of the images in this report.

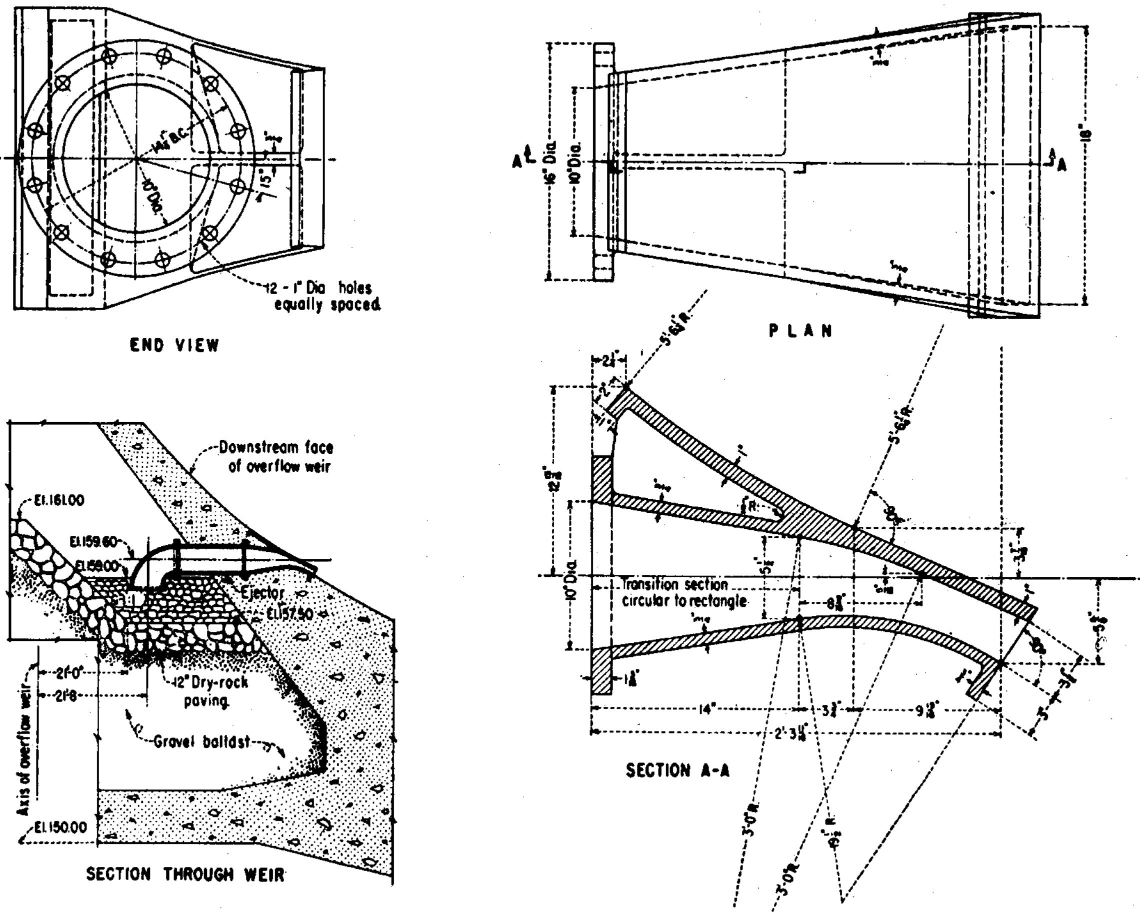

Figure 3. Ejector configuration.

Subsequent to the original model studies and construction of Imperial Diversion Dam, a service roadway was placed from 270 feet to 540 feet downstream from the toe of the spillway. The lowest elevation along the north edge of roadway is approximately 161.18 feet. A 6-foot culvert with an approximate invert elevation of 155 feet drains water under the roadway that is displayed in Figure 4.

Figure 4. Looking downstream at the service road and culvert.

Hydrology

Imperial Irrigation District personnel provided the following historic maximum reservoir water surface elevation and discharges[1] that occurred on August 20, 1983.

Maximum historic reservoir water surface elevation	181.51 ft
Maximum historic spillway (overflow weir) discharge	2,380 ft^3/s
Maximum historic sluiceway discharge	30,000 ft^3/s

This event was caused by the temporary emergency closure of the California Sluice Gates to rescue two passengers in a canoe. It is reported that high water marks inside of overflow weir section part of the dam were created during this event[3]. During the overflow event the roadway was overtopped. It was also reported that there was significant backwater due to the sluiceway discharge into the old Colorado River channel. It was speculated that the high water mark was caused by reverse flow though the flow ejectors caused by stilling basin water surface elevation that was higher than originally studied. The measured high watermarks inside of overflow weir section part of the dam vary between 164.29 feet and 164.5 feet. The event has lead to the consideration that reverse flow prevention devices on the ejectors might benefit the stability of the dam.

This event and others have lead to the recommendation 2001-SOD-B to assess the failure probability for hydrologic loads.

Scope

This study analyzed various conditions at Imperial Dam using a Computational Fluid Dynamics (CFD) program simulating 3-dimensional (3D) and 2-dimenstional (2D) flow fields. The investigation included the following:

- Revaluate the reservoir water surface elevation to discharge relationship for the spillway
- For the stability analysis, simulate the pressure field with the service roadway (with higher stilling basin water surface than originally studied) and the original conditions for the following cases
    - Simulate the Design Storm
    - Simulate the 10,000-year flood
    - Simulate the 1,000-year flood
- Simulate the maximum historic event to examine flows through the ejectors
- Determine the maximum spillway discharge without overtopping the service roadway
- Investigate the maximum spillway discharge without reverse flow though the ejectors

Conclusions

The capacity of the overflow wear is not reduced due to the service roadway for discharges up to the design storm since the weir is controlling the flow and is not submerged.

For the dam stability analyses, 6 cases were simulated and the pressure profile that resulted from water on the concrete surfaces were conveyed to the Structural Analysis Group. These cases were

- The design storm, 180,000 ft^3/s, with the service roadway in place
- The design storm, 180,000 ft^3/s, without the service roadway (for the assumption it washes away)
- The 10,000-year storm, 143,000 ft^3/s, with the service roadway in place
- The 10,000-year storm, 143,000 ft^3/s, without the service roadway (for the assumption it washes away)
- The 1,000-year storm, 82,000 ft^3/s, with the service roadway in place
- The 1,000-year storm, 82,000 ft^3/s, without the service roadway (for the assumption it washes away)

The high water mark inside of overflow weir section of the dam was likely created by the record-setting overflow weir event of 1983. The high watermark could have reached the average stilling basin surface water elevation of 165.87 feet if the maximum discharge was allowed to spill for long durations. This does not preclude the possibility of backwater from the sluiceway discharge of 30,000 ft^3/s creating a similar effect.

The service roadway will be overtopped for overflow weir discharges greater than to be 169 ft^3/s, but the effects of variations in siltation and vegetation was not evaluated.

The culvert under the service roadway will be inadequate for overflow weir discharges greater than 118 ft^3/s, and reverse flow through the ejectors into the overflow weir section part of the dam will occur. The variations in siltation and vegetation effects on this discharge were not evaluated.

If the service roadway erodes away during a storm and the discharge-tailwater reverts back to that shown on drawing 212-D-843 (1936), the ejectors will function as originally designed. This study did not investigate channel alterations after 1936 and assumed that 212-D-843 would represent an extreme low-water condition.

Recommendations

This study did not evaluate the stability of the overflow weir section of the dam. These recommendations should be considered in conjunction with the results stability analysis.

Reverse flow prevention devices on the ejectors are recommended for consideration. Some of the issues for consideration include

- The lack of drainage from inside of the dam during discharge events.
- Possibility of leakage from the stilling basin through the reverse flow prevention devices.
- Mechanical reverse flow prevention devices typically decrease discharges, which would cause a higher water level inside of the dam.

Alternatives to the reverse flow prevention devices would be to modify the service roadway or pipe water from inside the dam to a point downstream of the service roadway.

COMPUTATIONAL DEVELOPMENT

There are many steps required to develop an appropriate CFD model. These include development, refinement, and testing of the grid, boundary conditions, model extents, and obstacles (structures).

CFD Program Description

The CFD program FLOW-3D® by Flow Science Inc.[4], was used to model the various configurations. FLOW-3D® is a physically-based finite difference/volume, free surface, transient flow modeling system that was developed to solve the Navier-Stokes equations, in three spatial dimensions.

The finite difference equations are based on an Eulerian mesh of non-uniform hexahedral control volumes using the Fractional Area/Volume (FAVOR) method[5]. Free surfaces and material interfaces are defined by a fractional volume-of-fluid (VOF) function. FLOW-3D® uses an orthogonal coordinate system as opposed to a body-fitted system.

Model Descriptions

The models used the Renormalized Group (RNG) option for viscosity, which is an advanced turbulence simulation technique. The RNG algorithm uses equations similar to the more familiar k-*e* turbulence model. However, equation constants are found empirically in the k-*e* model, but are derived explicitly in the RNG model. Generally, the RNG model has wider applicability than the k-*e* turbulence model. In particular, the RNG model is known to describe more accurately low intensity turbulence flows and flows having strong shear zones[5].

For the momentum equation, the first-order advection approximation was used. For pressure iterations, the Line Alternating-Direction-Implicit successive over-relaxation option was chosen for 3D models, and the line-Implicit successive over-relaxation option was used for 2D models.

The CFD approach to these simulations used various cell configurations and spatial extents to optimize computation time. While smaller cell sizes develop more precise definition of obstacles and flows, they also increase the size of the computational domain, and decrease the time step (when the explicit option is used) of the simulation. Both of these increase computational time required for obtaining a quasi-steady state solution. Balancing the accuracy of the solution with the time and computational resources available is always a challenge.

For the Imperial study, several models were used. In all cases, general flow patterns were resolved quickly using a low number of large cells, and then the simulation was continued with a large number of smaller cells so the flow would be well defined and to achieve a simulation that was independent of the grid. Appendix A through E has describes the various model setups. Many of the storm events modeled were taken from the report, "Flood Frequency Study, Imperial Diversion Dam, Arizona[6]."

To create the geometry (stereolithography), the surveyed coordinates were offset by 1704818.379 feet to the left in the X-direction, and 7319037.208 feet to the south in the Y-direction such that the minimum X valued modeled was zero, and the minimum Y value modeled was zero.

RESULTS

Spillway Crest Calibration

This study evaluated the discharge - reservoir water surface elevation relationship of the spillway crest. The study included reservoir water surface elevations between 182 feet to 198 feet, at 1-foot increments. These were simulated to provide flow conditions for the ejectors and for comparison of the submerged flow tests. A description of the models used and tabular results are in Appendix A – Spillway Crest Calibration Models. Results

(Figure 5) matched very closely to the 1935 1:30 physical model study performed at the Old Customhouse Laboratory circa 1935. The average difference was less than 1.2% too low for the CFD simulations. The CFD study assumed the silt elevation to be 172 feet, while the silt elevation would likely change during a storm.

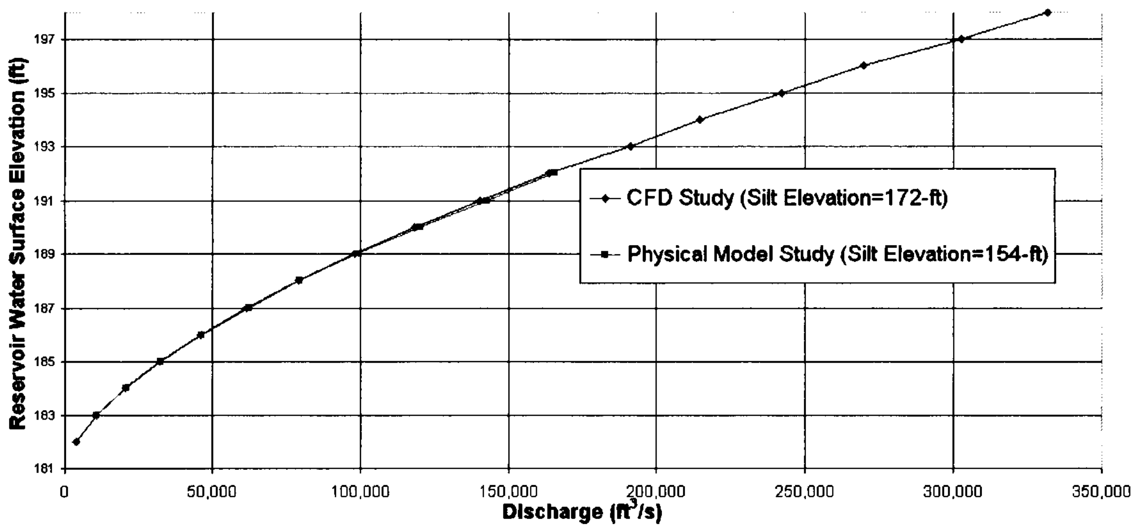

Figure 5. Discharge capacity curve for the overflow weir at Imperial dam.

Design Storm Modeling

Since the original model studies did not include the existing service roadway, the stability of the dam was examined with the new conditions. An additional complicating factor is if the roadway quickly erodes during a storm and relieves pressure around the dam. This part of the study investigated hydraulic pressure on and around overflow weir section part of the dam with and without a service road. The Design Storm has been estimated to at 180,000 $ft^3/.s.$[6] For purposes of this study, the discharge - tailwater elevation relationship was taken from drawing 212-D-843 (Figure 6), which is estimated to give lower water elevations than might be the case due to subsequent downstream construction, and should provide conservative information for the stability analysis from before to after the erosion of the service roadway.

To achieve the pressures two steps were needed. First a 3D model of the stilling basin and service roadway determined the average height of the surface for the average water surface elevation at the end of the stilling basin. Second a 2D spillway model was used to simulate the average water surface elevation in the stilling basin and to determine pressures on and around overflow weir section part of the dam with the roadway in place and without the roadway.

From the 3D model, the average water surface height downstream of the stilling basin was determined to be 176.81 feet elevation. Critical flow as defined by $V = \sqrt{gD}$ (velocity is equal to the square root of gravity times the hydraulic depth) occurs near the roadway with an approximate elevation of 174 feet, which indicates that the downstream water level would need to be much higher than what can be extrapolated from the discharge-tailwater curve on drawing 212-D-843. The 3D model details can be found in Appendix B – Tailwater models.

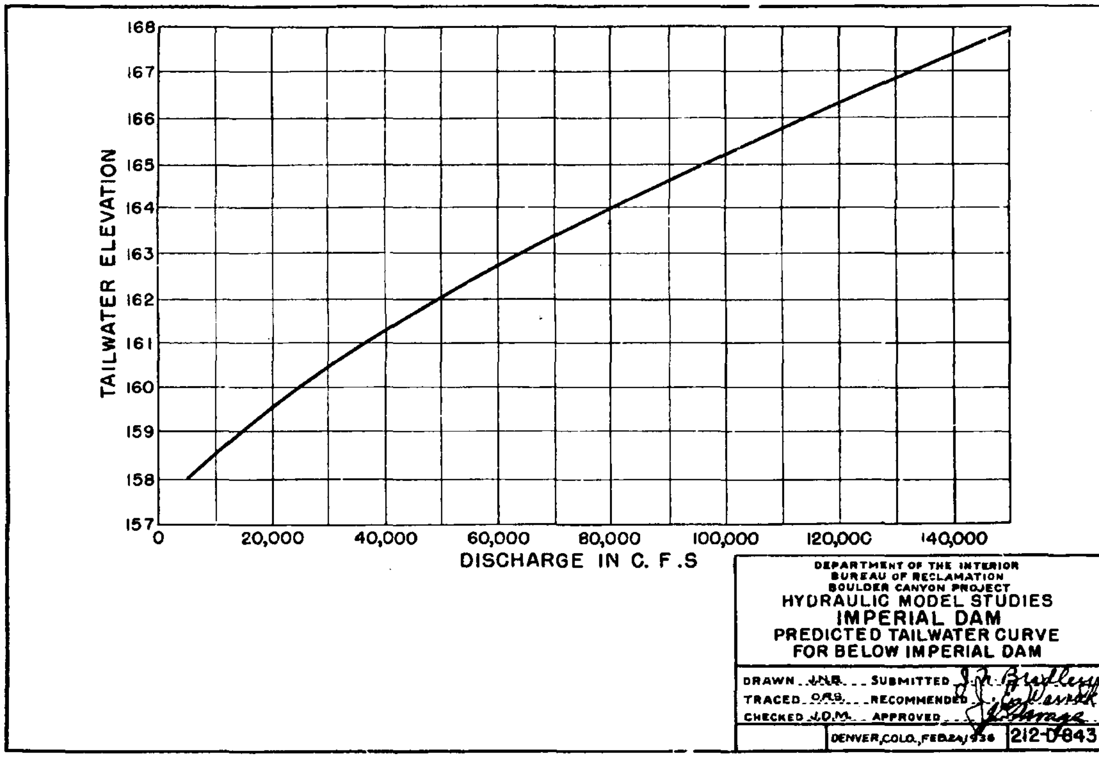

Figure 6. . Drawing 212-D-845, Imperial Dam predicted tailwater curve for below imperial Dam.

With a service road, the 2D spillway model shows that the tailwater is not high enough to decrease discharge over the crest (Figure 8). Average pressures on and around overflow weir section part of the dam from the 2D model with service road and without service road (Figure 9) were transmitted to the Structural Analysis Group for the stability analysis. Appendix C – Spillway Models details the 2D spillway models.

To insure the assumption that critical flow would hydraulically separate downstream phenomena from the stilling basin, the 3D model was modified to include downstream water surface elevation of 170 feet that was extrapolated from drawing 212-D-843. The simulation displayed critical flow across the length of the service roadway and a hydraulic jump downstream of the service roadway, indicating that tailwater surface elevation up to those indicated by drawing 212-D-843 will not effect water elevations in the stilling basin.

For this case, reverse flow occurs through the ejectors (see the section Maximum Spillway Discharge Without Reverse Flow Through Flow Ejectors). Given sufficient duration of overtopping at this discharge, the water level would become equal to the average water level in the stilling basin, 176.81 feet.

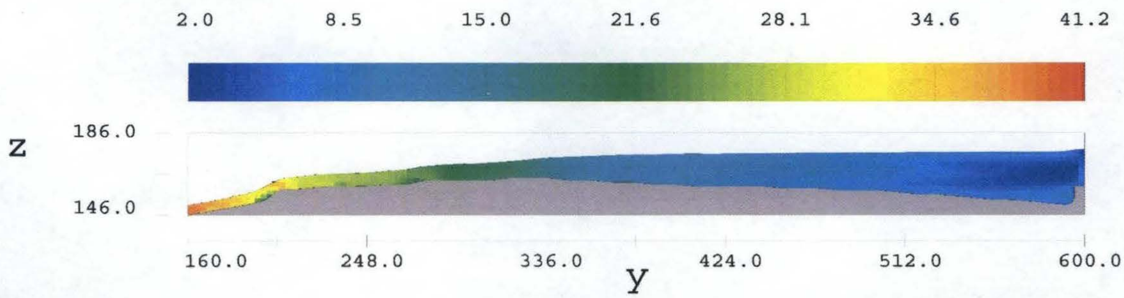

Figure 7. Total velocity shown in colored contours in ft/s for a section of the 180,000 ft^3/s model. The flow is from the right to the left. On the right is a water-source object, which simulates flow from the stilling basin. Note that super-critical flow occurs near the highest elevation, which is the service roadway, around y=300 feet.

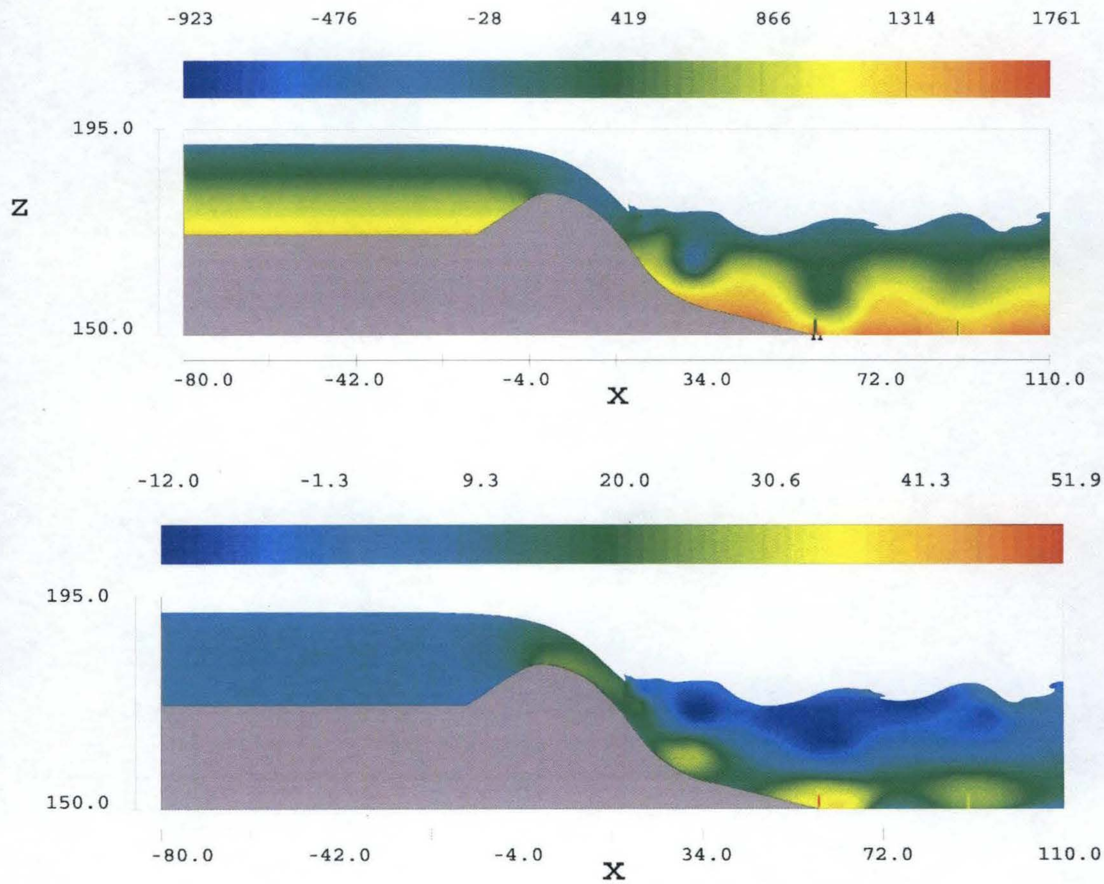

Figure 8. Color contours of static pressures shown in lbs/ft^2 (top) and X-Velocities in ft/s (bottom) for the 180,000 ft^3/s 2D model with a service roadway downstream. The downstream water level is not high enough to decease the flow over the crest. The ejector at elevation 159.6 feet is not shown nor was simulated in this model.

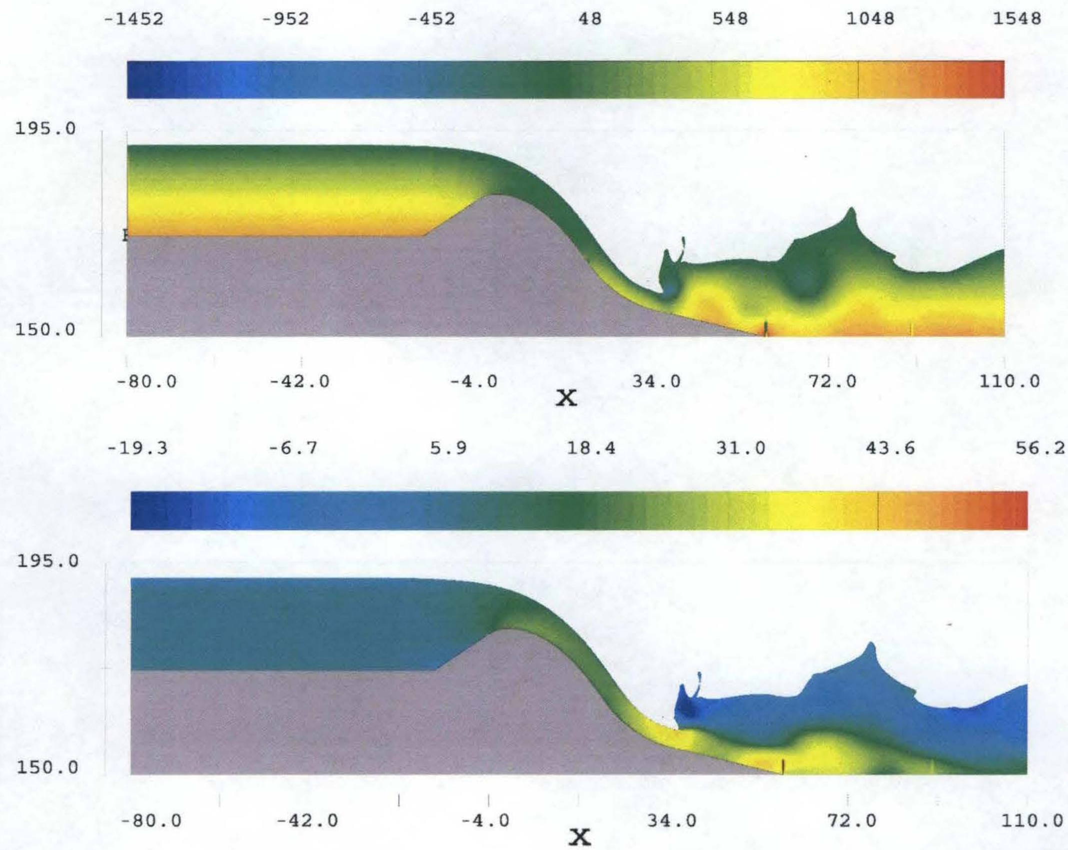

Figure 9. Color contours of pressures shown in lbs/ft² (top) and X-Velocities in ft/s (bottom) for the 180,000 ft3/s 2D model without a service roadway downstream. The ejector at elevation 159.6 feet is not shown nor was simulated in this model.

The 10,000-Year Flood Modeling

The investigation of the 10,000-year flood event used the same procedures described under the above section labeled Design Storm Modeling. The 10,000-year flood has been estimated at 143,000 ft^3/s.[6]

From the 3D model (Figure 10), the average water surface height downstream of the stilling basin was determined to be 175.25 feet elevation.

With a service road, the 2D model shows that the stilling basin water surface is not high enough not decrease discharge over the crest (Figure 11). Average pressures on and around overflow weir section part of the dam with service road and without service road (Figure 12) were transmitted to the Structural Analysis Group for the stability analysis.

For this case, reverse flow occurs through the ejectors (see the section Maximum Spillway Discharge Without Reverse Flow Through Flow Ejectors). Given sufficient duration of overtopping at this discharge, the water level would become equal to the average water level in the stilling basin, 175.25 feet.

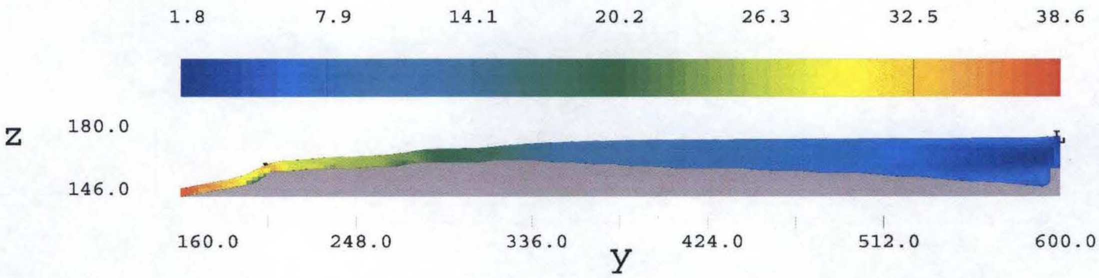

Figure 10. Total velocity shown in colored contours in ft/s for a section of the 143,000 ft^3/s model. The flow is from the right to the left. On the right is a water-source object, which simulates flow from the stilling basin. Note that super-critical flow occurs near the highest elevation, which is the service roadway, around y=300 feet.

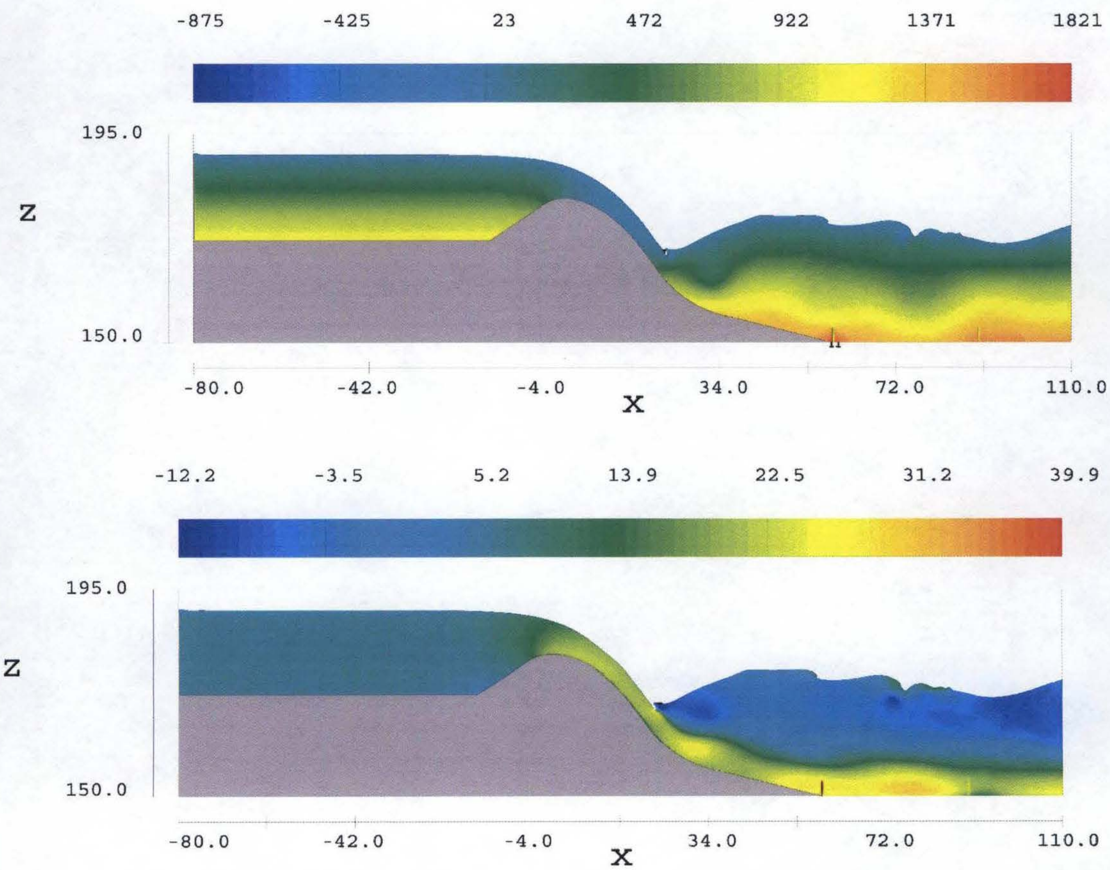

Figure 11. Color contours of pressures shown in lbs/ft^2 (top) and X-Velocities in ft/s (bottom) for the 143,000 ft^3/s 2D model with a service roadway downstream. The ejector at elevation 159.6 feet is not shown nor was simulated in this model.

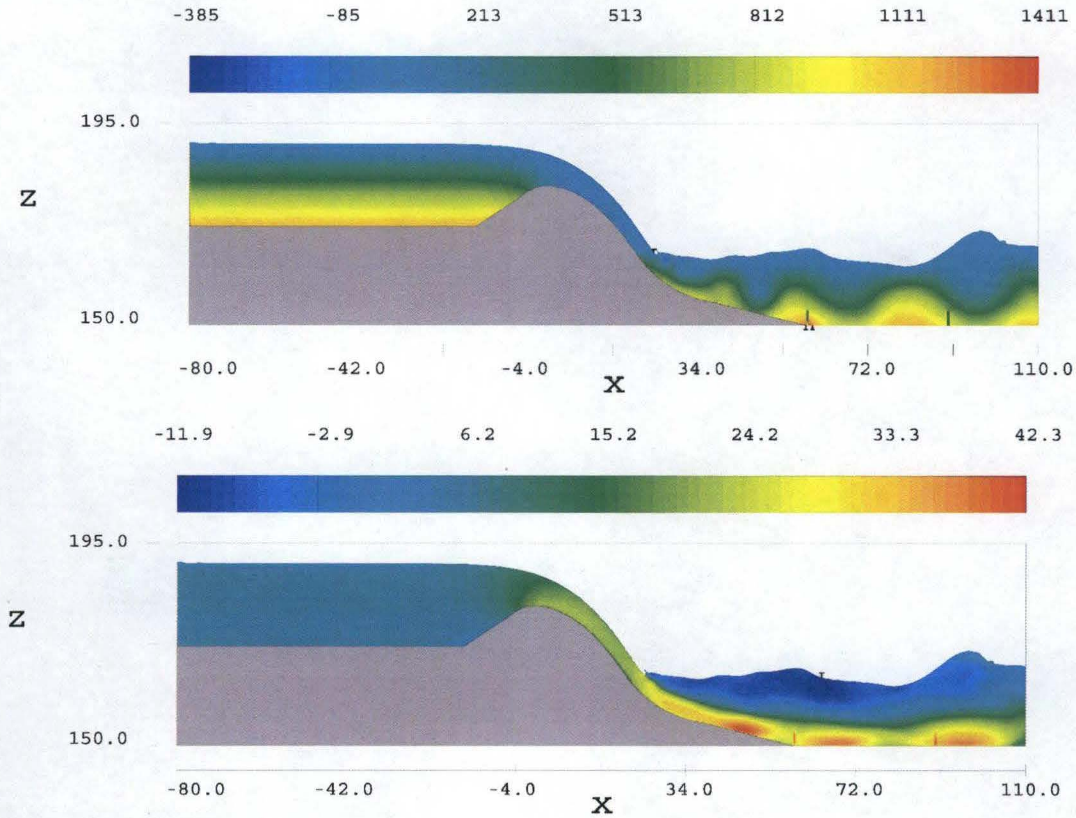

Figure 12. Color contours of pressures shown in lbs/ft² (top) and X-Velocities in ft/s (bottom) for the 143,000 ft3/s 2D model without a service roadway downstream. The ejector at elevation 159.6 feet is not shown nor was simulated in this model.

The 1,000-Year Flood Modeling

The investigation of the 1,000-year flood event also used the same procedures described under the above section labeled Design Storm Modeling. The 1,000-year flood has been estimated to pass 82,000 ft3/s.[6]

From the 3D model (Figure 13), the average water surface height downstream of the stilling basin was determined to be 172.15 feet elevation.

With a service road, the 2D model shows that the tailwater is not high enough not decrease discharge over the crest (Figure 14). Average pressures on and around overflow weir section part of the dam with service road and without service road (Figure 15) were transmitted to the Structural Analysis Group for the stability analysis.

For this case, reverse flow occurs through the ejectors (see the section Maximum Spillway Discharge Without Reverse Flow Through Flow Ejectors). Given sufficient duration of overtopping at this discharge, the water level would become equal to the average water level in the stilling basin, 172.15 feet.

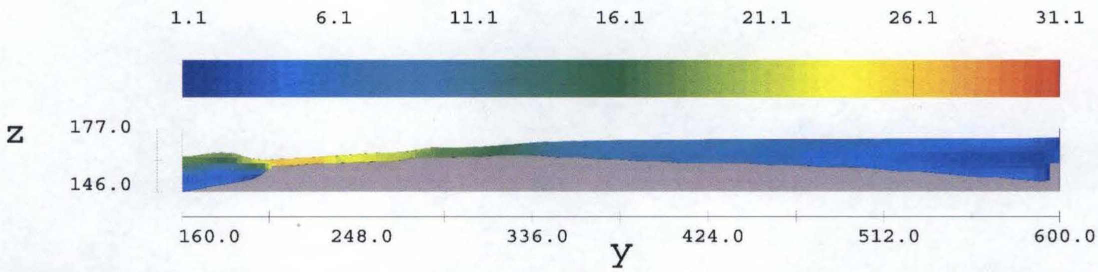

Figure 13. Total velocity shown in colored contours in ft/s for a section of the 82,000 ft^3/s model. The flow is from the right to the left. On the right is a water-source object, which simulates flow from the stilling basin. Note that super-critical flow occurs near the highest elevation, which is the service roadway, around y=300 feet.

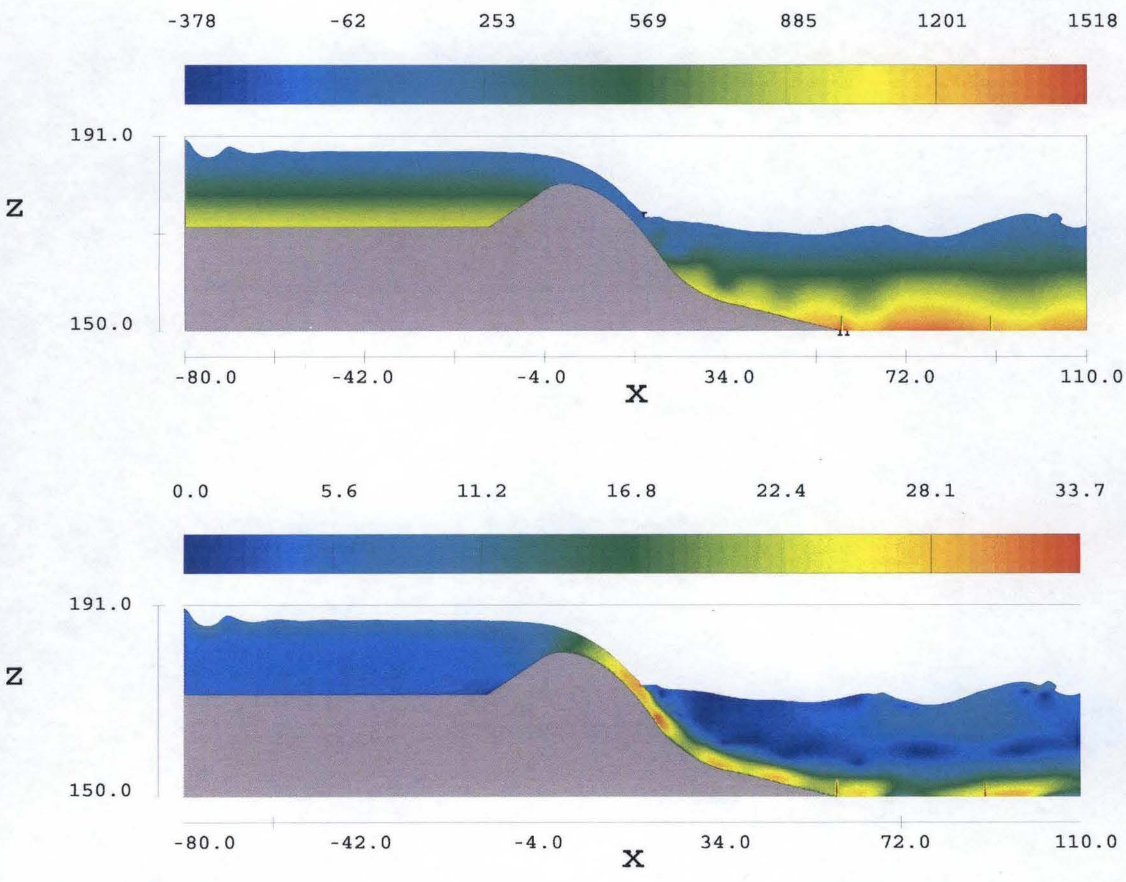

Figure 14. Color contours of pressures shown in lbs/ft^2 (top) and X-Velocities in ft/s (bottom) for the 82,000 ft^3/s 2D model with a service roadway downstream. The ejector at elevation 159.6 feet is not shown nor was simulated in this model.

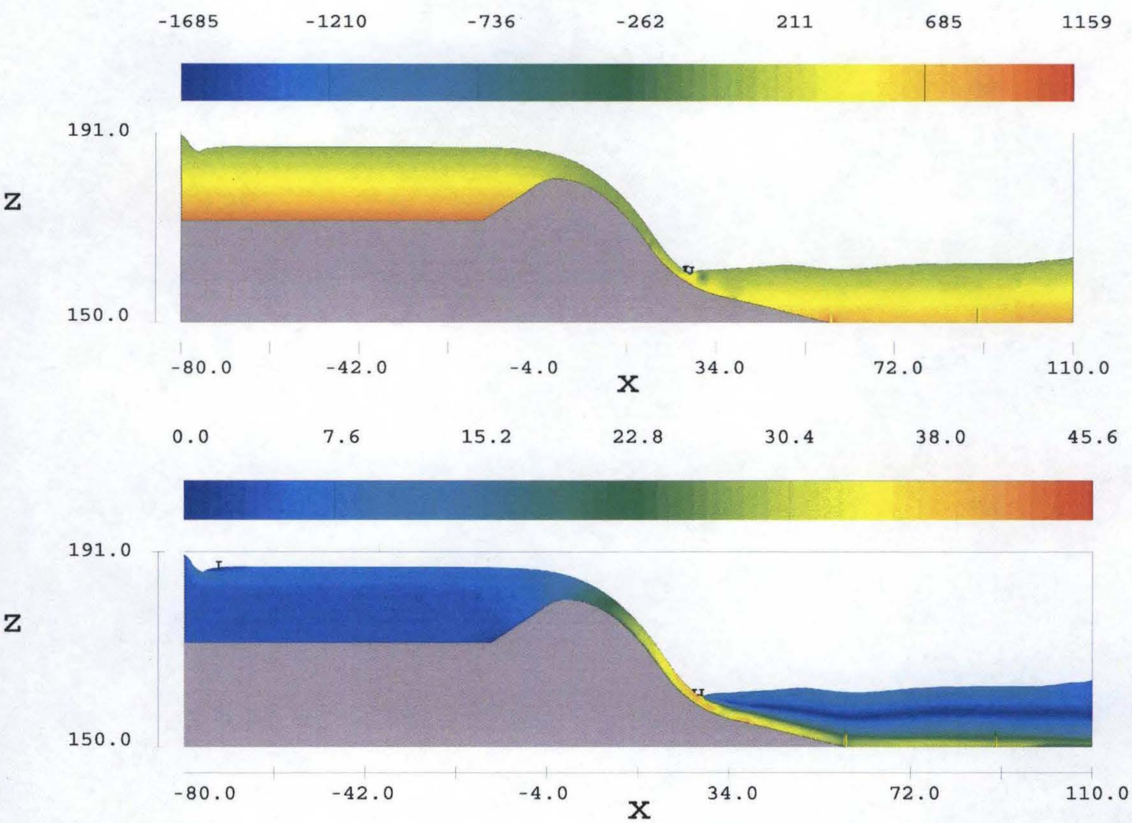

Figure 15. Color contours of pressures shown in lbs/ft^2 (top) and X-Velocities in ft/s (bottom) for the 82,000 ft^3/s 2D model without a service roadway downstream. The ejector at elevation 159.6 feet is not shown nor was simulated in this model.

Maximum Historic Spillway Discharge – 1983

The overtopping event that occurred on August 20, 1983 was modeled to determine if that overflow condition could cause reverse flow through the ejectors. During the event a maximum overflow weir discharge of 2,380 ft^3/s at reservoir water surface elevation 181.51 ft was measured. The measured high watermarks inside of overflow weir section varied between 164.29 feet and 164.5 feet. This part of the investigation considered if reverse flow through the ejectors could have occurred during the event.

First, a 3D model of the stilling basin and service roadway determined the average water surface elevation at the end of the stilling basin. The spatial extents of this 3D model only included part of the roadway that was overtopped and included the culvert. The 3D model details can be found in Appendix B – Tailwater models. The average water surface elevation downstream of the stilling basin ranged between 165.42 feet and 165.87 feet (Figure 16).

Second, a 2D spillway model (Figure 17) was used to determine flow conditions approaching the ejectors. Appendix C – Spillway Models details the 2D spillway model. This simulation determined the surface elevation of the flow at the entrance of the 2D ejector model (x=19 ft) is 167.95 feet for this condition with an average velocity of 19.5 ft/s.

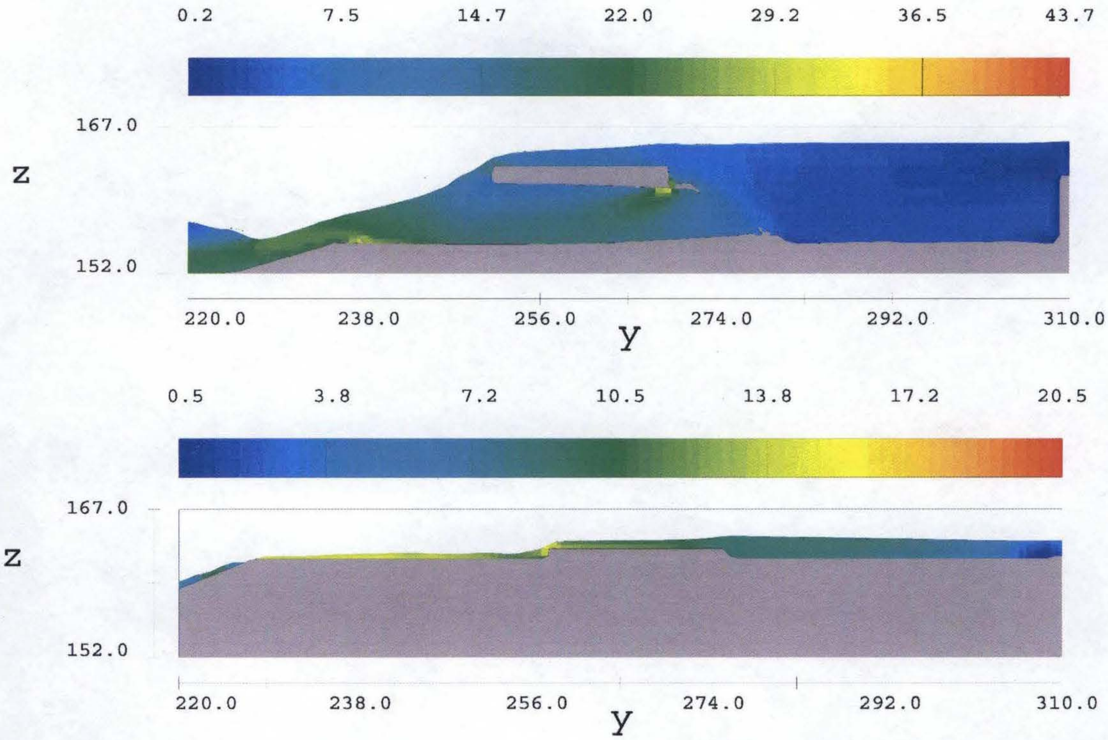

Figure 16. Total velocity contours in ft/s (from right to left) for the maximum historic event through the culvert (top) and 58 feet to the east of the culvert (bottom).

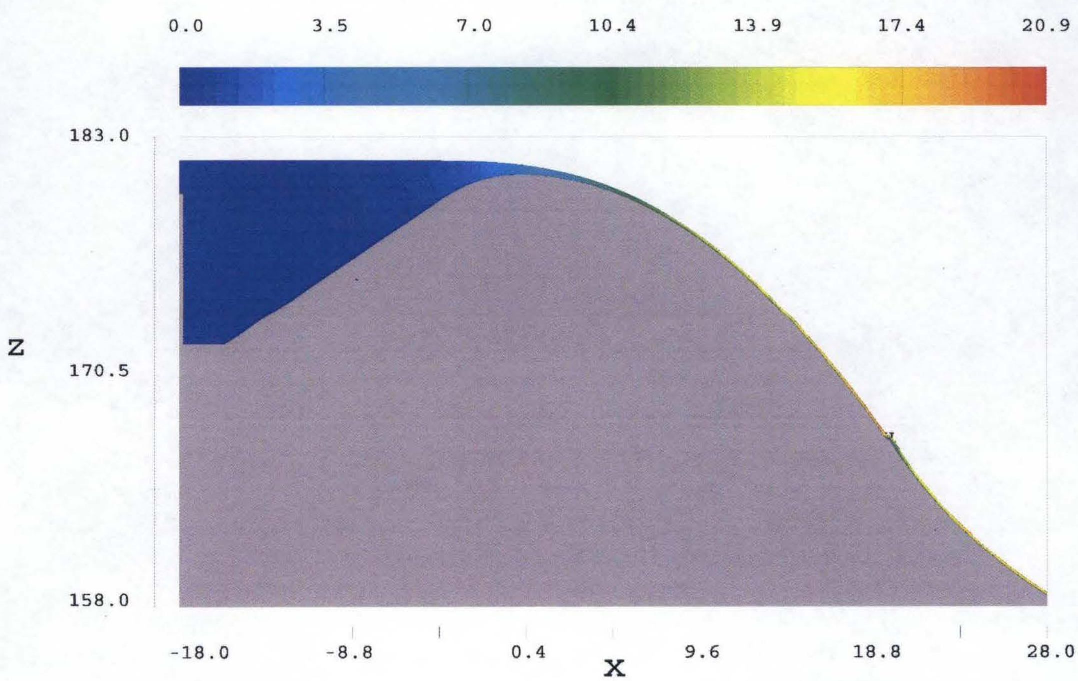

Figure 17. Velocity contours in ft/s for the historic maximum discharge overflow event of 2,380 ft^3/s. The surface elevation simulated at the elevation of the flow at the entrance of the 2D ejector model (x=19 ft) is 167.95 feet.

Third, a 2D ejector model was used to determine if reverse flow would take place. The description of the model is in Appendix D – Ejector 2D models. Results indicated that there would be reverse flow, and the end of the ejector would be well submerged. If the condition were allowed to continue, the simulation indicated the water level inside of the overflow weir could nearly reach the water surface elevation in the stilling basin of 165.87 feet.

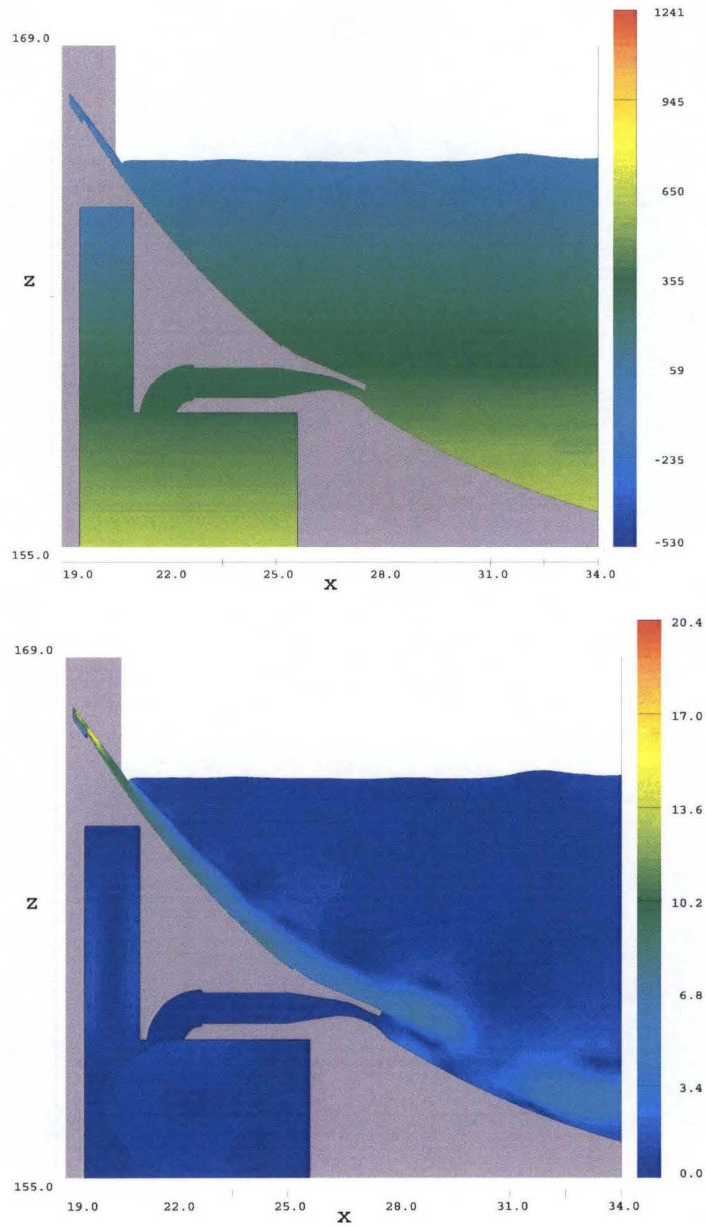

Figure 18. Contours of pressure in lbs/ft^2 (top) and velocity in ft/s (bottom) for the historic maximum overflow wier event of 2,380 ft^3/s. The 2D spillway simulation determined that the water surface elevation at the entrance of this model should be 167.95 feet. An object source and separte object to fix the depth can be seen on the top-left of each image. Note that at the pressures distribution inside of the overflow weir and outside are nearly equal and the velocity field at the downstream portion of the ejectors appears unstable.

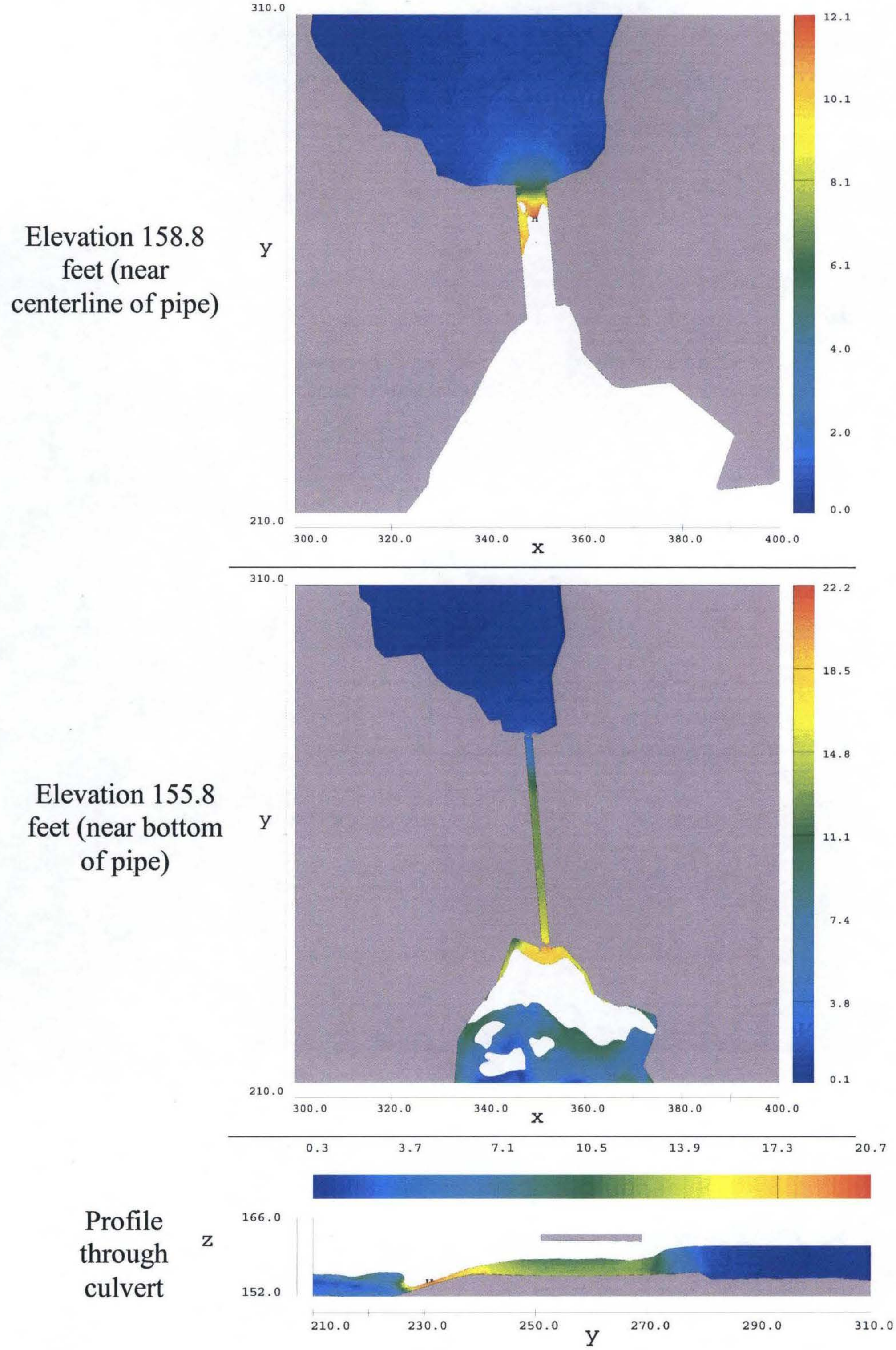

Figure 19. Velocity contours in ft/s from the Maximum Spillway Discharge Without Overtopping The Service Roadway model. On the top is a horizontal slice through elevation 158.8 feet with the flow from top to bottom. In the middle is a horiontal view through elevation 155.8 feet with the flow from top to bottom. On the bottom is a profile view through the culvert at X=350 feet with flow from right to left.

The 2D model of an ejector used an entrance velocity higher than what resulted from the 2D model to be conservative since a higher velocity head would tend to push the hydraulic jump away from the ejector. Unlike other models, this model also used a surface roughness of 0.007 feet for the concrete in addition to the wall shear option. As can be seen from Figure 22 the pressure distribution inside the ejector and the stilling basin are virtually the same. Accordingly, for long overtopping periods with steady flows of 118 ft^3/s or greater, the water surface inside of the overflow weir would be near the water surface elevation in the stilling basin.

From this part of the investigation, it is concluded that with sufficient duration, all flows above 118 ft^3/s through the culvert will cause the water surface elevation of 160 feet in the stilling basin, submerging the ejectors and causing reverse flow.

Maximum Spillway Discharge Without Overtopping the Service Roadway

The maximum flow condition that would not overtop the roadway was investigated to inform operators of that limit. The lowest measured part on the north side of the service roadway is 161.18 feet, and is to the west of the culvert. This elevation is below the top of the culvert, which is at elevation 161.6 feet. The 3D model used to simulate the flow through the culvert is described in Appendix B – Tailwater models. The spatial extents (Figure 19) were smaller that the previously described 3D models, and only contained the region immediately around the culvert. An upstream boundary condition was set to 161.18 feet, which would not account for waves and some energy losses of the flow moving towards the culvert, such that the results would slightly over predict the allowable flow without any overtopping. This flow was found to be 169 ft^3/s. Changes of siltation and vegetation would affect the flow.

Maximum Spillway Discharge Without Reverse Flow Through Flow Ejectors

Due to the nature of the flow restriction of the service roadway and culvert, an assumption was made and tested that reverse flow though the ejectors would take place for all flows that cause an average water surface elevation in the stilling basin above elevation 160 feet. This is 4.8-inches above the centerline of the ejectors. The approach used 3 steps. First, 3D modeling of the culvert and immediate area using a water surface elevation of 160 feet, to determined the discharge through the culvert. Second, a 2D spillway model was used to simulate the approach depth and velocity for the model of the flow ejector. Third, a 2D model of a flow ejector was used to simulate the water surface elevation inside of the overflow weir. The last 2 steps could have been combined into one model if the resulting computational domain was not too large for available equipment and time.

The 3D model (Figure 20) was very similar to that presented in the previous section labeled Maximum Spillway Discharge Without Overtopping the Service Roadway and is detailed in Appendix B – Tailwater models. It used an upstream boundary condition with a water surface elevation 160 feet and downstream water surface elevation was 156 feet (drawing 212-D-843). The resulting simulated discharge was 118 ft^3/s. Changes of vegetation growth and siltation would affect the maximum discharge.

For the 2D spillway model, the depth of 118 ft^3/s going over the 1,197.5-foot-wide overflow weir is very small compared to the total height of the structure. This required many small cells and long run times and a model with smaller extents. This model used the "wall shear" option (law-of-the-wall). The flow is highly friction controlled and velocities are much lower than what would otherwise be expected (when compared to the energy head) as can be seen from Figure 21, where peak velocities can be seen around 6.95 ft/s.

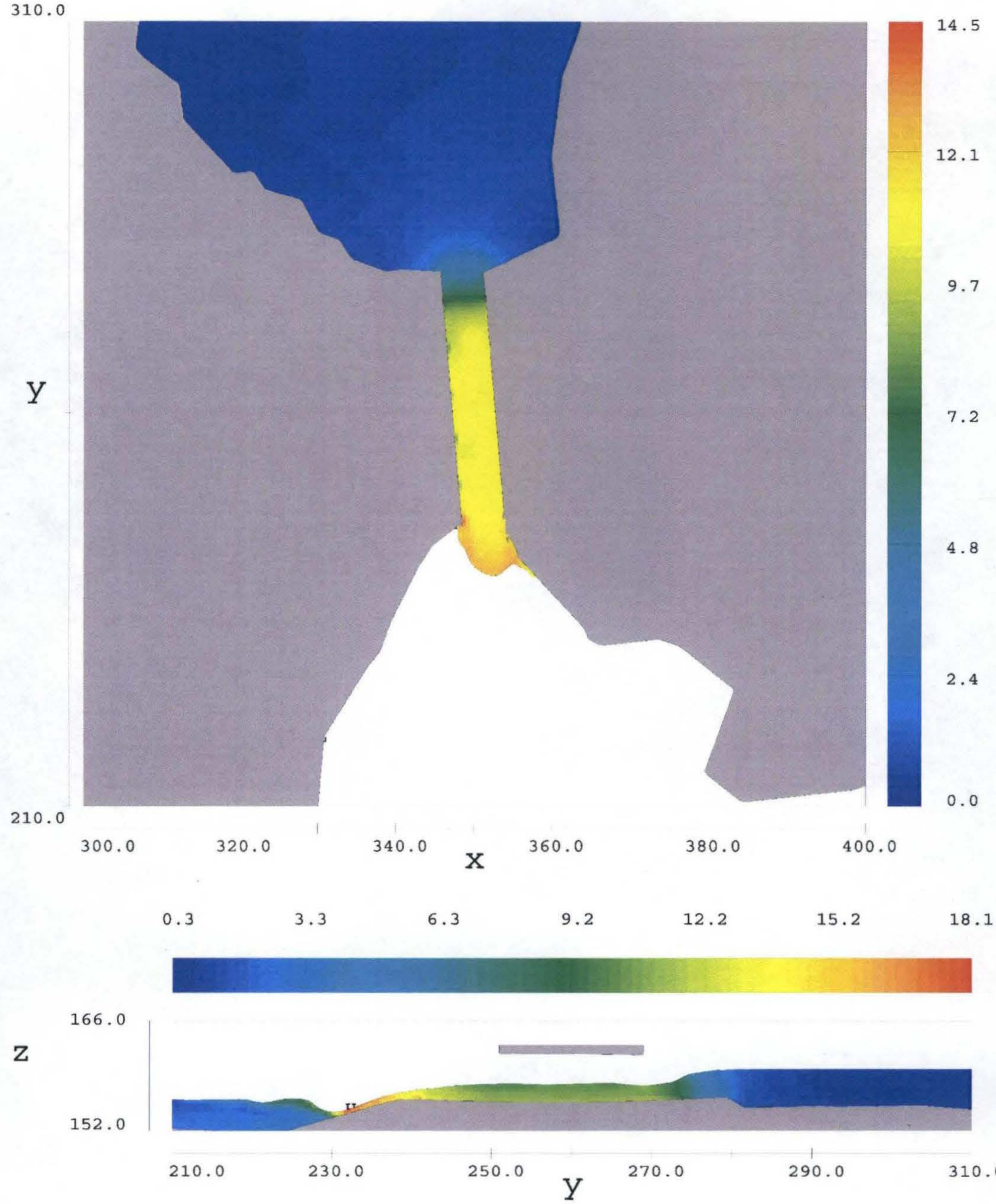

Figure 20. Velocity contours in ft/s from the maximum spillway discharge without reverse flow through the ejectors with the Service Roadway model. On the top is a horizontal view through elevation 152.7 feet with flow from top to bottom. On the middle is a profile view through elevation 155.8 feet with flow from top to bottom. On the bottom is a profile view through the culvert at X=350 feet with flow from right to left.

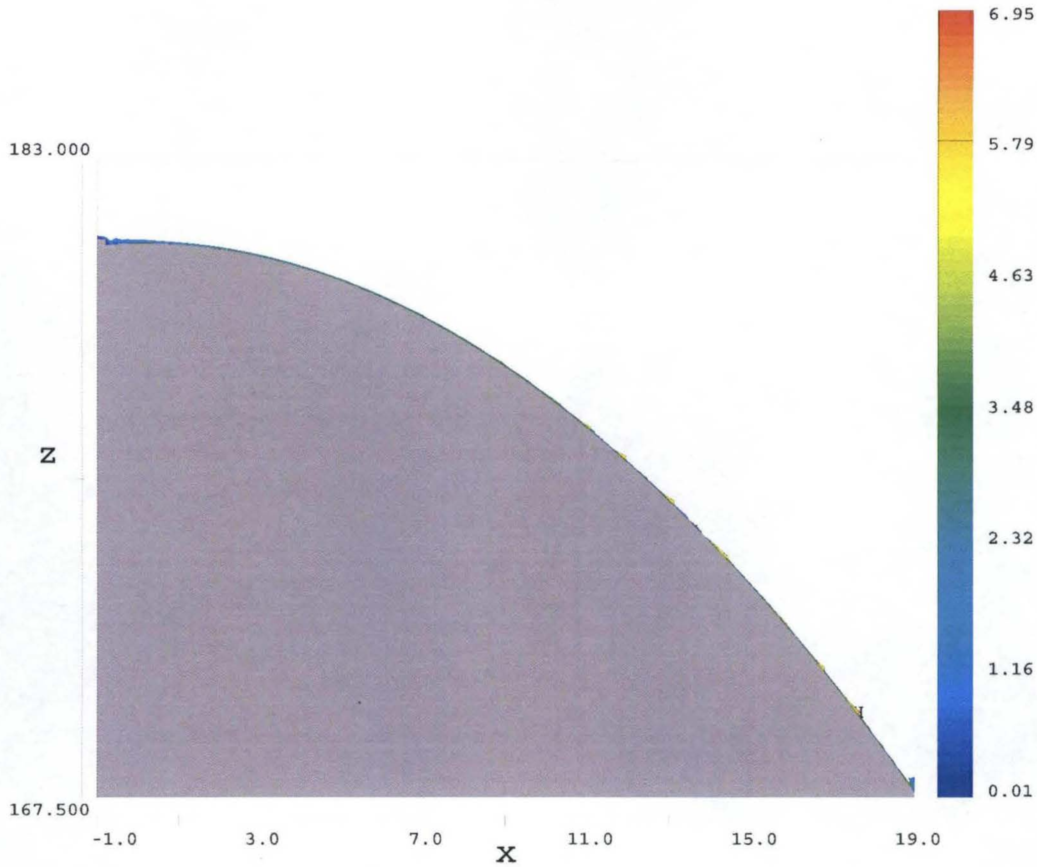

Figure 21. Overflow weir simulated discharge of 118 ft^3/s. The source object on the top-left simulates an entrance flow of 0.0985 ft^2/s per foot of crest width. The surface waves are typical of thin flows with high Froude numbers.

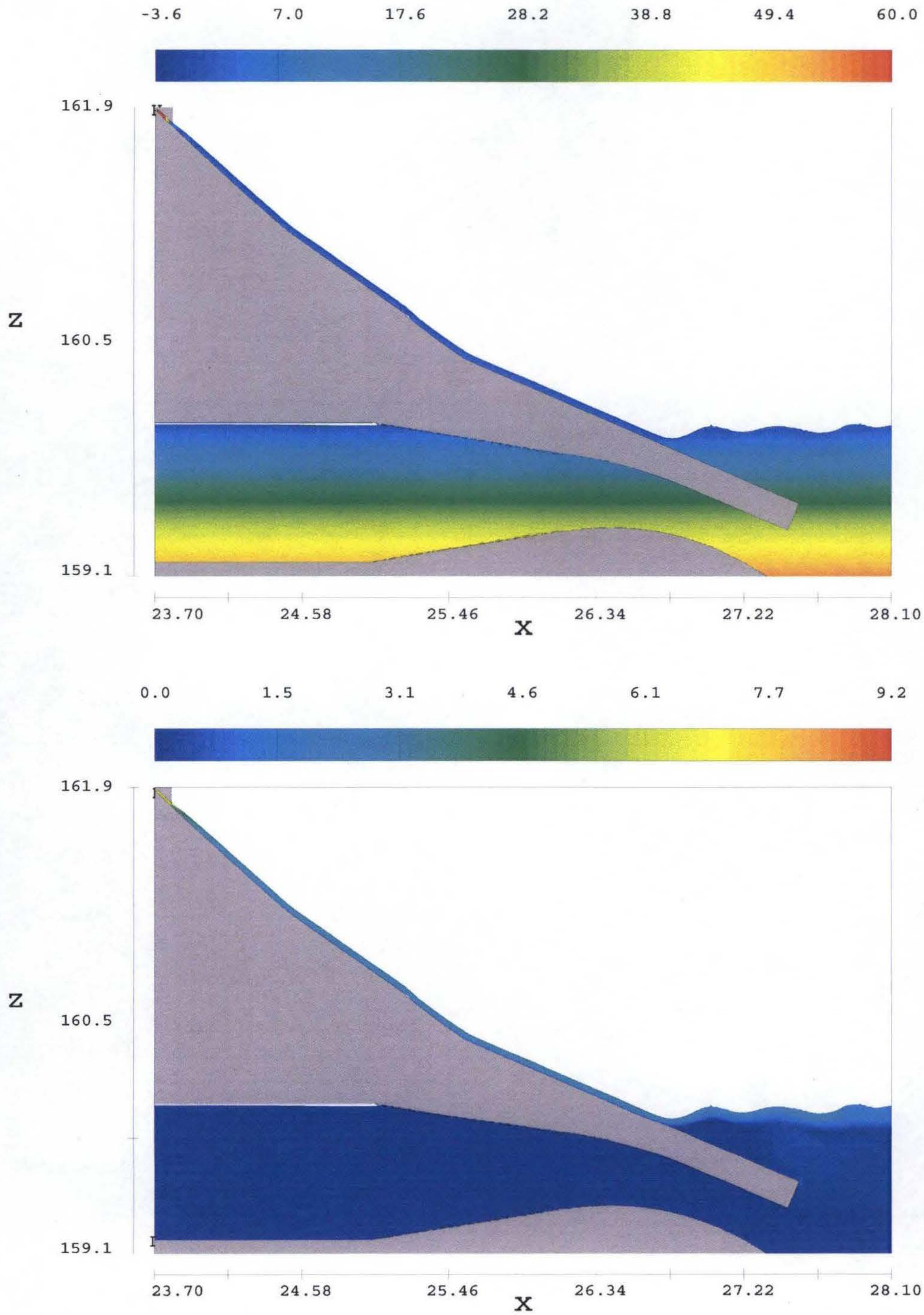

Figure 22. Overflow weir simulated discharge of 118 ft3/s over ejectors. Note that pressure distribution shown on the top in lbs/ft2 is nearly the same inside as outside of the ejector. On the bottom are color contours in ft/s.

Sectional Models of the Ejectors

The 3D ejector sectional model was used to simulate outflow of the ejectors (Figure 3) while there was no overflow. Initially these models were intended to be also used in conjunction with an overflow condition, but due to instabilities caused by cells that were either too large to capture the small components of the ejectors, or too many cells that caused excessively long run times that were not feasible. The models are described in Appendix E – Ejector Sectional models.

The results of the simulation are displayed in Table 1.

Table 1. Results of CFD Simulation Compared with the 1934 Model Studies. During the CFD run, the simulation proved to be very unstable and compairs poorly with the physical model study.

Water Surface Elevation Inside Overflow Weir (ft)	Sectional CFD Model Discharge (ft3/s)	Physical Model Discharge (1934) (ft3/s)
160	1.9	2.55
161	3.6	
162	4.9	5.75
163	5.9	
164	6.5	
165	7.4	

APPENDIX A – SPILLWAY CREST CALIBRATION MODELS

This appendix describes the 2D models that simulated the spillway crest overflow for the reservoir water surface elevations between 182 feet to 198 feet, at 1-foot increments. This was to provide intermediate flow conditions to the ejectors models.

The 2D models were developed using drawing 212-D-374 for the shape of the ogee crest, spillway and upstream geometry. The silt level was estimated from the AutoCAD drawing labeled GilaGravityCanalHeadWorks062603.dwg to be at elevation 172 feet. It should be noted that dredging changes the silt elevation. Also it should be noted that during a major flood event, the silt elevations could change dramatically.

For all cases, the simulation was started using a single mesh with cell size of 1-foot on each side. The simulation was continued using cell size of 0.5 feet, with a nested mesh (Figure 23) of 0.125 feet. Velocity plots for the overtopping case with reservoir water surface elevation at 188 feet are displayed in Figure 24 and the nested mesh is displayed in Figure 25. Comparative results between this study and the 1:30 physical model study is shown in Table 2.

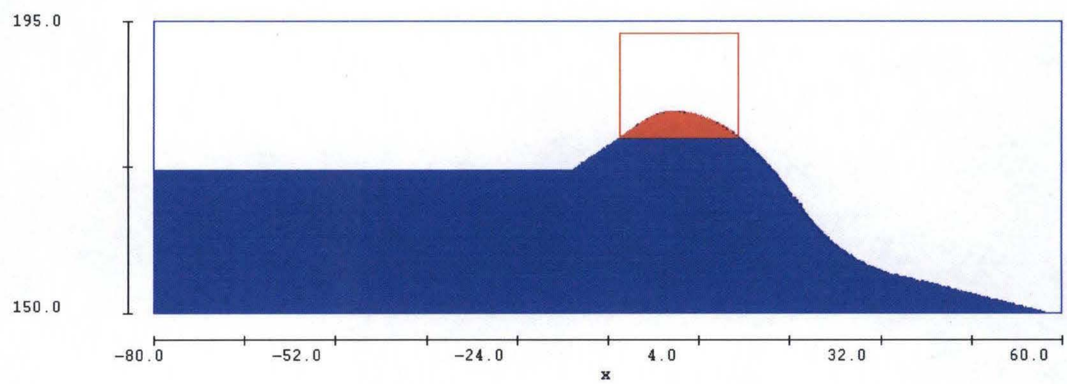

Figure 23. Composite view of outer- and inner-mesh limits used for the overtopping investigations. The red rectangle corresponds the nested mesh, while the blue rectangle corresponds to the limit of the outer mesh. The grid lines are not shown in this plot.

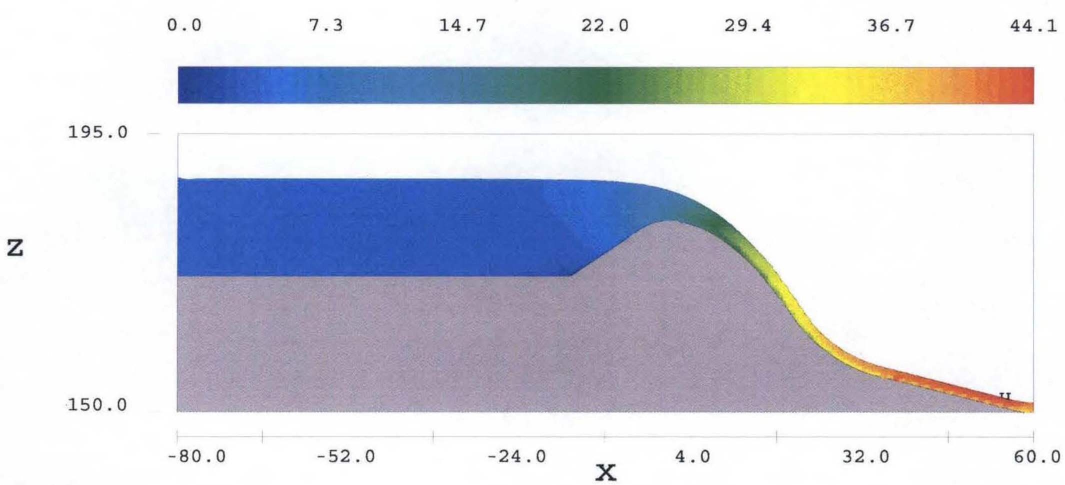

Figure 24. Final configuration of the outer mesh of the overtopping case with reservoir water surface elevation at 188 feet. The cells are 0.5 feet on each side. Grid lines are not shown.

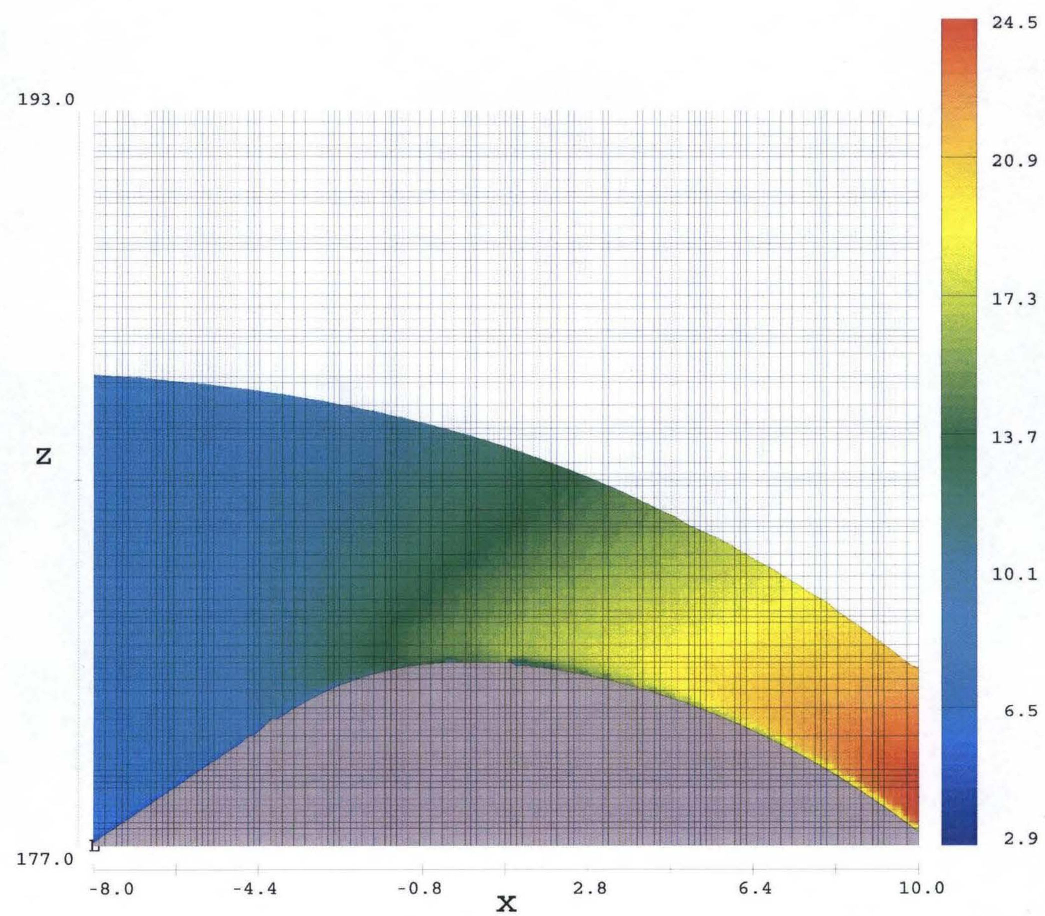

Figure 25. Final nested mesh for the overtopping case with reservoir water surface elevation at 188 feet. The cells are equally spaced 0.125 feet on each side. (Not all grid lines may appear in this plot)

Table 2. Comparison of the water surface elevation verse discharge results of the CFD study verses the 1:30 physical model study.

Water surface elevation (ft)	CFD Study Results (ft^3/s)	Physical model Study (ft^3/s)
182	3,740	
183	10,886	11,042
184	20,522	20,845
185	32,347	32,764
186	46,163	46,458
187	61,600	62,303
188	79,090	79,619
189	98,085	98,902
190	118,540	120,277
191	140,570	142,763
192	163,897	165,578
193	191,166	
194	214,547	
195	241,997	
196	269,505	
197	302,592	
198	331,919	

APPENDIX B – TAILWATER MODELS

This appendix describes the 3D models that simulated tailwater conditions due to the service roadway for the following conditions:

- The design storm (180,000 ft^3/s)
- The 10,000-year flood (143,000 ft^3/s)
- The 1,000-year flood (82,000 ft^3/s)
- The Maximum spillway discharge without reverse flow through the flow ejectors
- The Maximum spillway discharge without overtopping the service roadway
- The Maximum historic spillway discharge (1983)

The resulting average stilling basin water surface elevations were used as the downstream boundary condition for 2D spillway models, which determined pressures on and around overflow weir section part of the dam while the service roadway is intact. The resulting average stilling basin water surface elevations were also used to determine if the overflow weir section would be submerged causing a change in the reservoir water surface elevation to discharge relationship.

Bathymetry

In general, the spatial extent of these 3D models were from the base of the spillway on the north to tens of feet south of the service roadway, and from the east and west banks of the Old Colorado River Channel as surveyed on June 30, 2003 by Rick Hick, Brandon VanHorn, and Mario Mendez and shown in Figure 26. Geometric data[7] was entered in to an in-house FORTRAN program that generated a stereolithography file that was part of the input to Flow-3D, the Computational Fluids Modeling package that was used for the simulations as shown in Figure 27. To create the stereolithography, the surveyed coordinates were offset by 1704818.379 feet to the left in the X-direction, and 7319037.208 feet to the south in the Y-direction. Horizontal slices through the Flow-3D model are shown in Figure 28. This improves the available number of significant figures used in the Flow-3D code, which is compiled with FORTRAN.

The culvert under the service roadway was trimmed out of the stereolithography file and extended using a Flow-3D baffle with an invert elevation of 154.982 feet. The field survey indicated the invert was at elevation 155.150 on the north side and 154.982 on the south side.

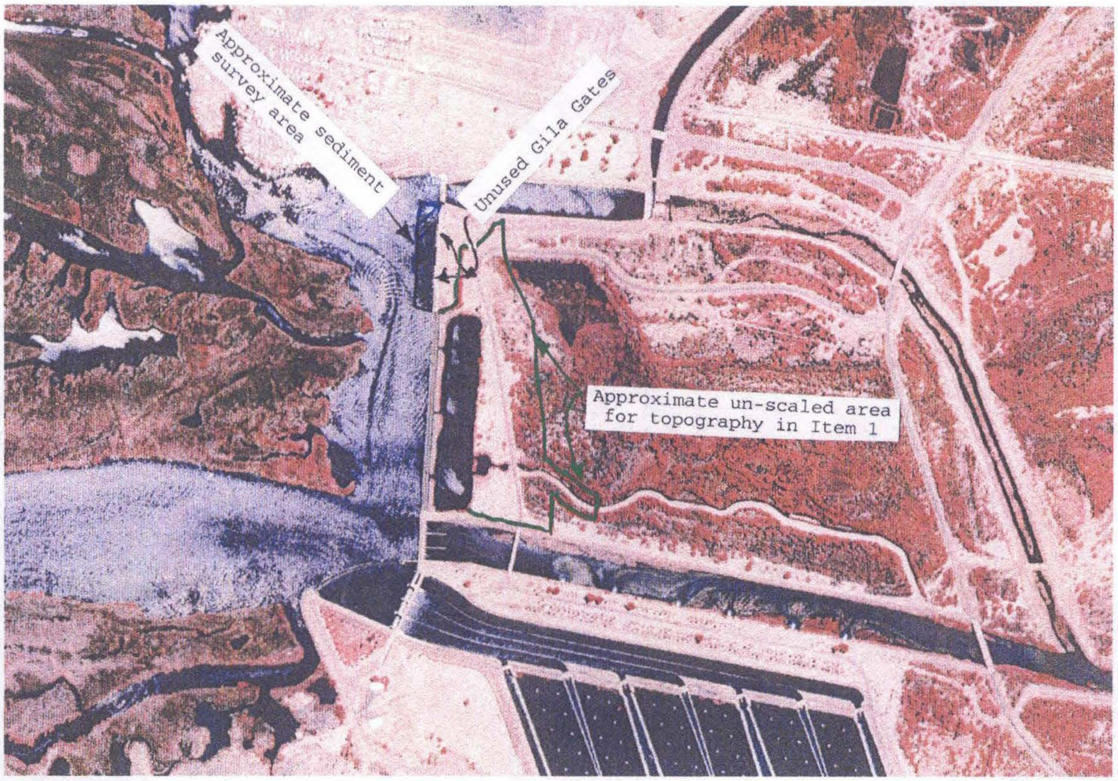

Figure 26. Limits of Old Colorado River Channel surveyed on June 30, 2003 by Rick Hick, Brandon VanHorn, and Mario Mendez. A subset of the region marked in green was used in the 3D modeling.

Cell Sizes

The initial cell size of each model ranged from approximately 2 feet to 10 feet to quickly resolve general flow patterns. The models were restarted and successively refined at least twice using smaller cells for more accuracy. The final extent and mesh size of the models are shown in Table 3.

Boundary Conditions

Four "no-flow" boundaries (wall condition) were used for the top, bottom, left (east), and right (west) boundaries of the model.

For high flows, it was assumed that the flow over the service roadway was critical such that conditions downstream of the roadway would not have effect the upstream flow conditions. An adequately low value of the downstream water height (south side) was used. For other cases the downstream water elevation was taken from drawing 212-D-843.

The upstream boundary varied by case. When the amount of discharge was known, the boundary was set to "no-flow" (wall) and a water source-object was placed at the boundary supplying a uniform flow to the model. This more accurately simulates the desired discharge. Where a water surface was known, the boundary was set to inflow water height so that the discharge could be calculated.

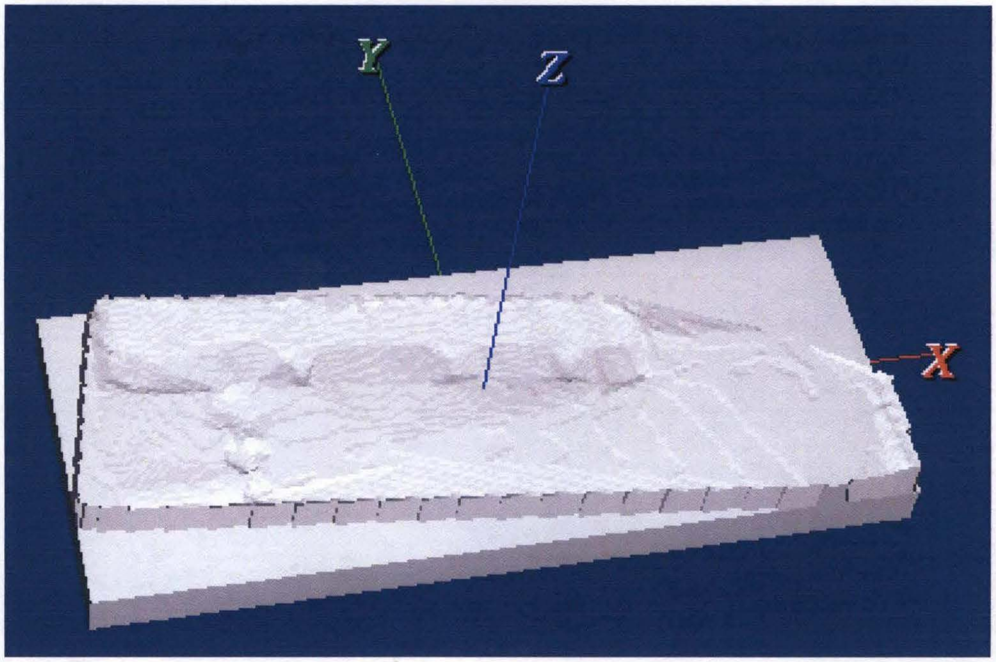

Figure 27. Stereolithography of the stilling basin and service roadway used in the 3-D modeling. The lower rectangle is the extent of the geometry, and outside of the range used in the models. The toe of the spillway is on the positive Y side. In this image it is difficult to make out the roadway that runs from the lower-left (south-west) of the image and extends well to the right (east) of the spillway then turns south. The surveyed coordinates were offset by 1704818.379 feet to the left in the X-direction, and 7319037.208 feet to the south in the Y-direction to create the stereolithography.

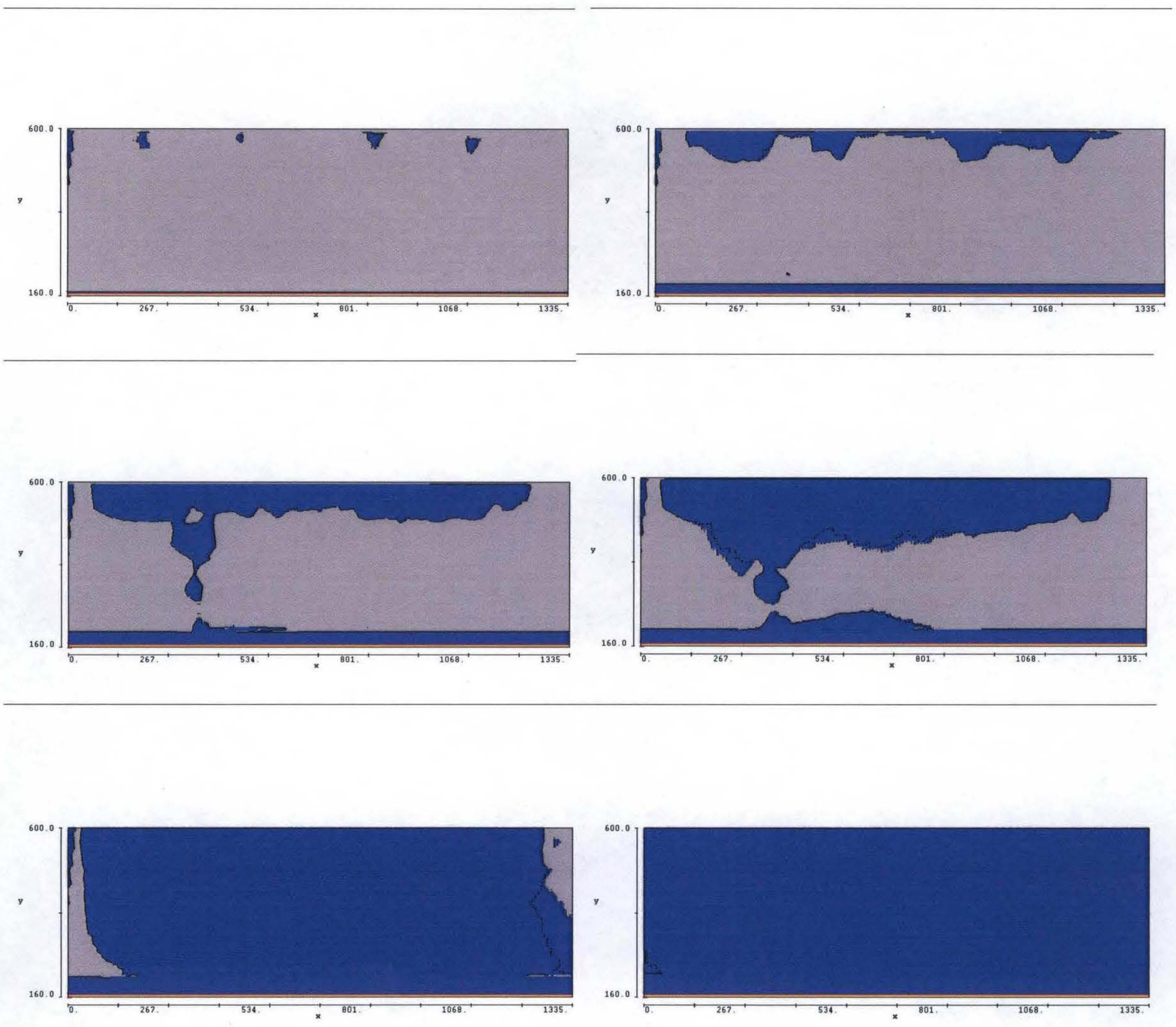

Figure 28. Typical slices of the 3D model in the X-Y plane. Elevations shown are (from left to right, top to bottom) 148-, 152-, 156-, 160-, 164-, and 168 feet. Flow is from top to bottom, with the toe of the spillway near the top and the service roadway near the bottom. The banks of the river can be seen on the sides. Some of the 3D models used smaller extents than shown in these images.

Table 3. Extent and number of cells used in the final simulations for the 3D models.

Case	X Min (ft)	X Max (ft)	No. of cells along X	Y Min (ft)	Y Max (ft)	No. of cells along Y	Z Min (ft)	Z Max (ft)	Number of cells along Z
Design storm (180,000 ft^3/s)	0	1135	445	160	600	147	146	168	20
10,000-year flood (143,000 ft^3/s)	0	1335	445	160	600	147	146	180	17
1,000-year flood (82,000 ft^3/s),	0	1335	399	160	600	132	146	170	10
Maximum historic spillway discharge (1983)	200	745	681	220	310	112	152	167	19
Maximum spillway discharge without overtopping the service roadway	300	400	200	210	310	200	152	166	28
Maximum spillway discharge without reverse flow through flow ejectors	300	400	200	210	310	200	152	166	28

APPENDIX C – SPILLWAY MODELS

This appendix describes the 2D spillway models that were mainly used to simulate pressures on the crest, upstream and stilling basin for the stability analysis to be performed by the Structural Analysis Group. These models were also used to determine if discharge over the overflow weir section is reduced due to higher downstream water levels caused by the service roadway. These models were also used to simulate flow conditions approaching the ejectors.

The 2D models were developed using drawing 212-D-374 for the shape of the ogee crest, spillway and upstream geometry. The silt level was estimated from the AutoCAD drawing labeled GilaGravityCanalHeadWorks062603.dwg to be at elevation 172 feet. It should be noted that periodic dredging changes the silt elevation. Also it should be noted that during a major flood event, the silt elevations could change dramatically. The 1:36 model study concluded that the head required to pass the maximum discharge for that study increased from 9.70 to 9.81 feet as the approach was raised from elevation 154.0 to elevation 175.0.

The chute blocks (Figure 29) were simulated using a Flow-3D baffle with an open porosity of 50%, located 58.25 feet downstream of the crest with a top at elevation 153.33 feet. The dentated sill was simulated using a Flow-3D baffle with an open porosity of 70%, located 89.75 feet downstream of the crest with a top elevation of 153.25 feet.

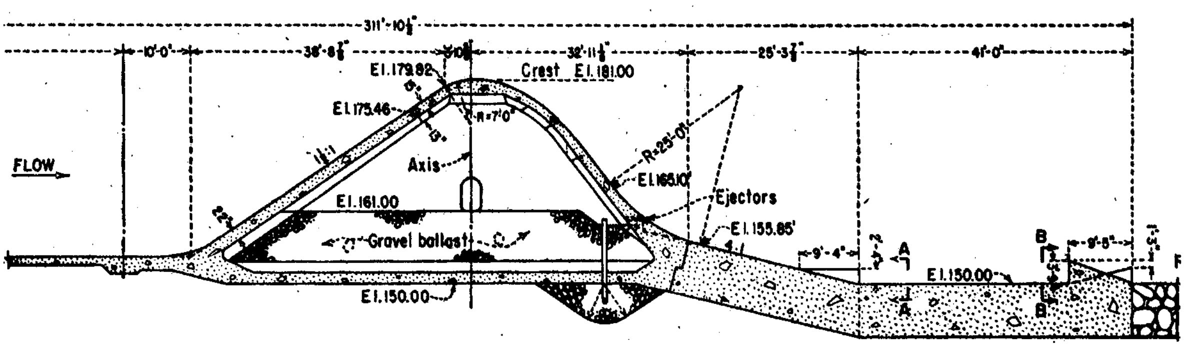

Figure 29. Overflow weir and stilling basin of Imperial Dam. The chute blocks are shown at the left of the apron that has an elevation of 150 feet. The chute blocks are 2.4895 feet wide with 2.4167 feet between them. The dentated sill is shown at the right of the apron. The dentates are spaced equally.

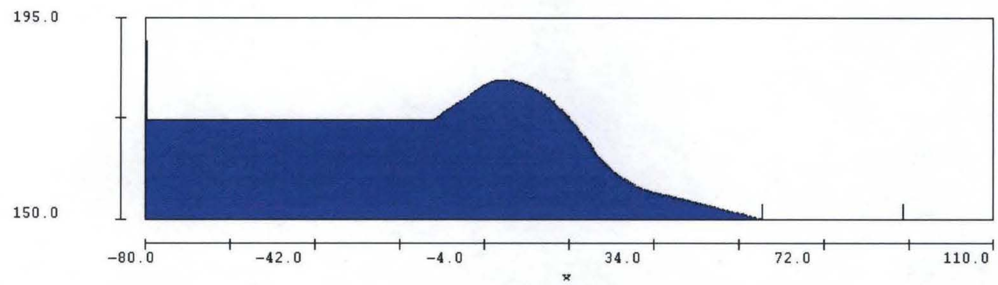

Figure 30. Flow-3D plot of the 2D spillway models. Note the baffles at 58.25 feet that simulated the chute blocks, and the baffle at 89.75 feet that simulated the dentated sill. The object to the left (X=-80) and above the silt) is the water source that controls the amount of flow going over the overflow weir section. The upstream silt is modeled at elevation 172.

Each model used an obstacle source to accurately control the amount of fluid entering the simulation. The object was placed near the left boundary (minimum X). Initial simulations used a cell size of 1.0 foot, while final runs used a cell size of 0.25 feet.

APPENDIX D – EJECTOR 2D MODELS

This appendix describes the 2D models of the ejectors used to simulate the 1983 event and to model the maximum spillway discharge without reverse flow through flow ejectors.

These models used results from the models as described in Appendix A – Spillway Crest Calibration Models and Appendix C – Spillway Models, and included an ejector detailed on drawing 212-D-3020 and displayed in Figure 31.

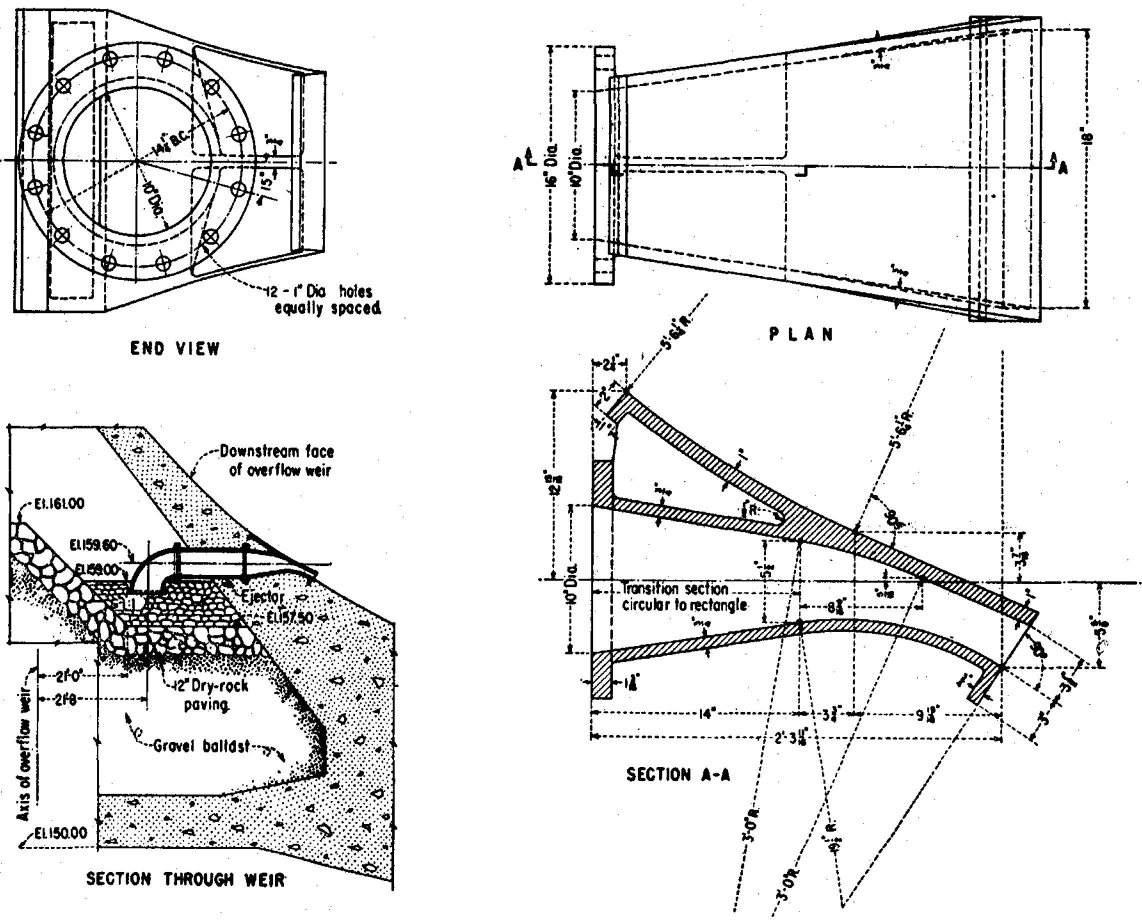

Figure 31. Ejector configuration.

These sectional models simulated half the total width of an ejector, and used

- A symmetrical boundary at the ejector centerline (minimum Y) that is delineated by section A-A of the plan view sown in Figure 31.
- A symmetrical boundary opposite to the above boundary (maximum Y) 0.10 feet away from the minimum Y boundary.
- A no flow boundary at the left boundary (minimum X).
- A pressure boundary with water surface elevation 157 at the right boundary (maximum X).

- A pressure boundary at the bottom (minimum Z) that maintained the desired water level in the overflow weir section.
- A wall boundary at the top (maximum Z)

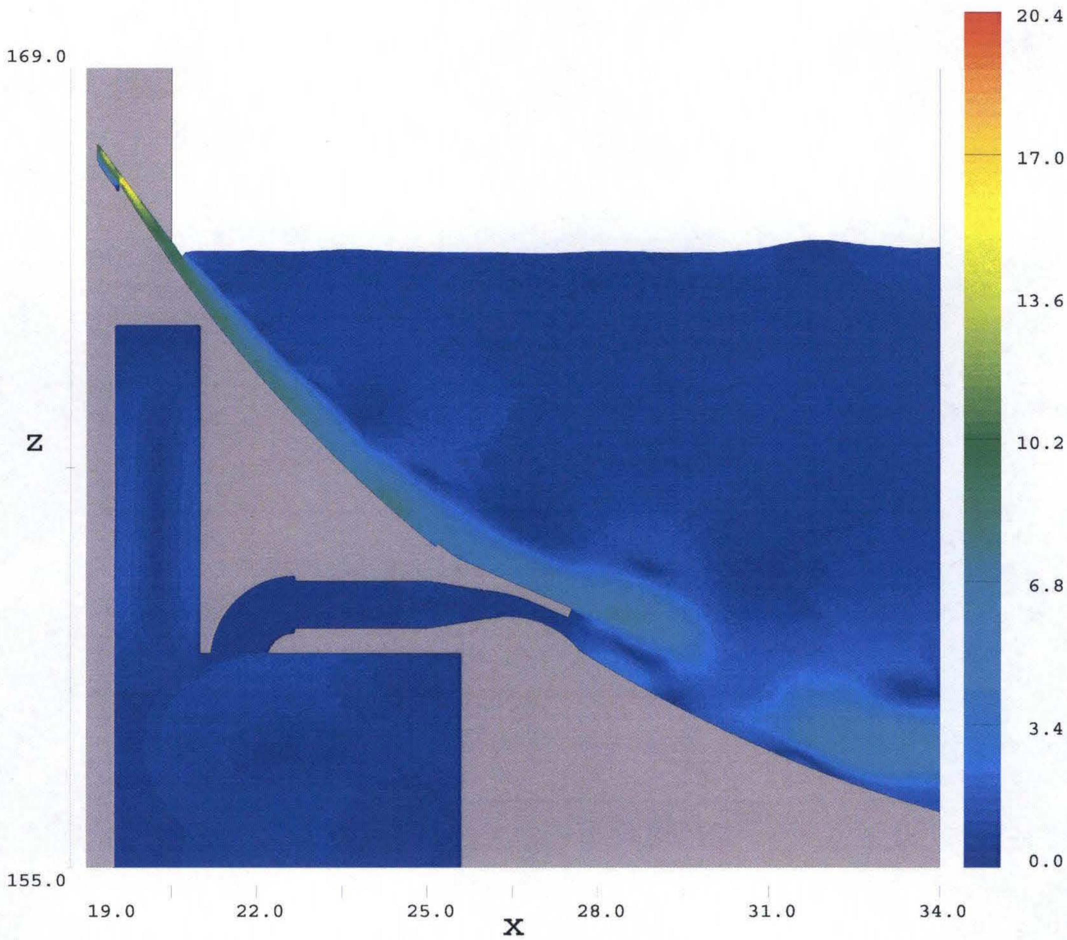

Figure 32. Profile view at the centerline symmetrical Y boundary for the 2D ejector models. A source-object supplies a set flow on the upper left and is confined by a block to achieve the velocity indicated by the spillway model.

APPENDIX E – EJECTOR SECTIONAL MODELS

The 3D ejector sectional models were used to simulate outflow of the ejectors (Figure 33) while there was no overflow. Initially these models were intended to be also used in conjunction with an overflow condition, but due to instabilities caused by cells that were either too large to capture the small components of the ejectors, or numerous cells that excessively long run times that were not feasible.

These models used the spillways as described in Appendix A – Spillway Crest Calibration Models, and included an ejector detailed on drawing 212-D-3020 and displayed in Figure 33.

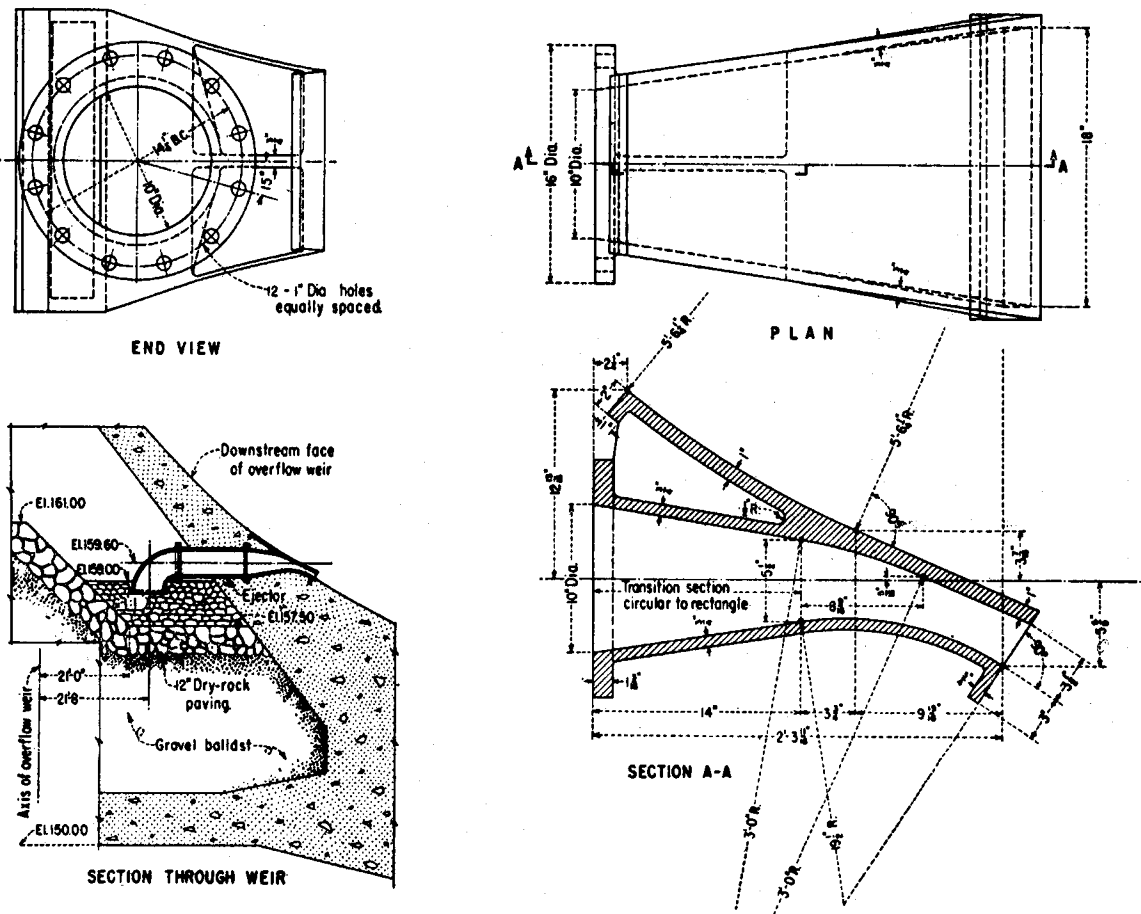

Figure 33. Ejector configuration.

These sectional models simulated half the total width of an ejector, and used

- A symmetrical boundary at the ejector centerline (minimum Y) that is delineated by section A-A of the plan view sown in Figure 33.
- A symmetrical boundary opposite to the above boundary (maximum Y), 1.5 feet away from the minimum Y boundary.
- A no flow boundary at the left boundary (minimum X).

- A pressure boundary with water surface elevation 157 at the right boundary (maximum X).
- A pressure boundary at the bottom (minimum Z) that maintained the desired water level in the overflow weir section.
- A wall boundary at the top (maximum Z).

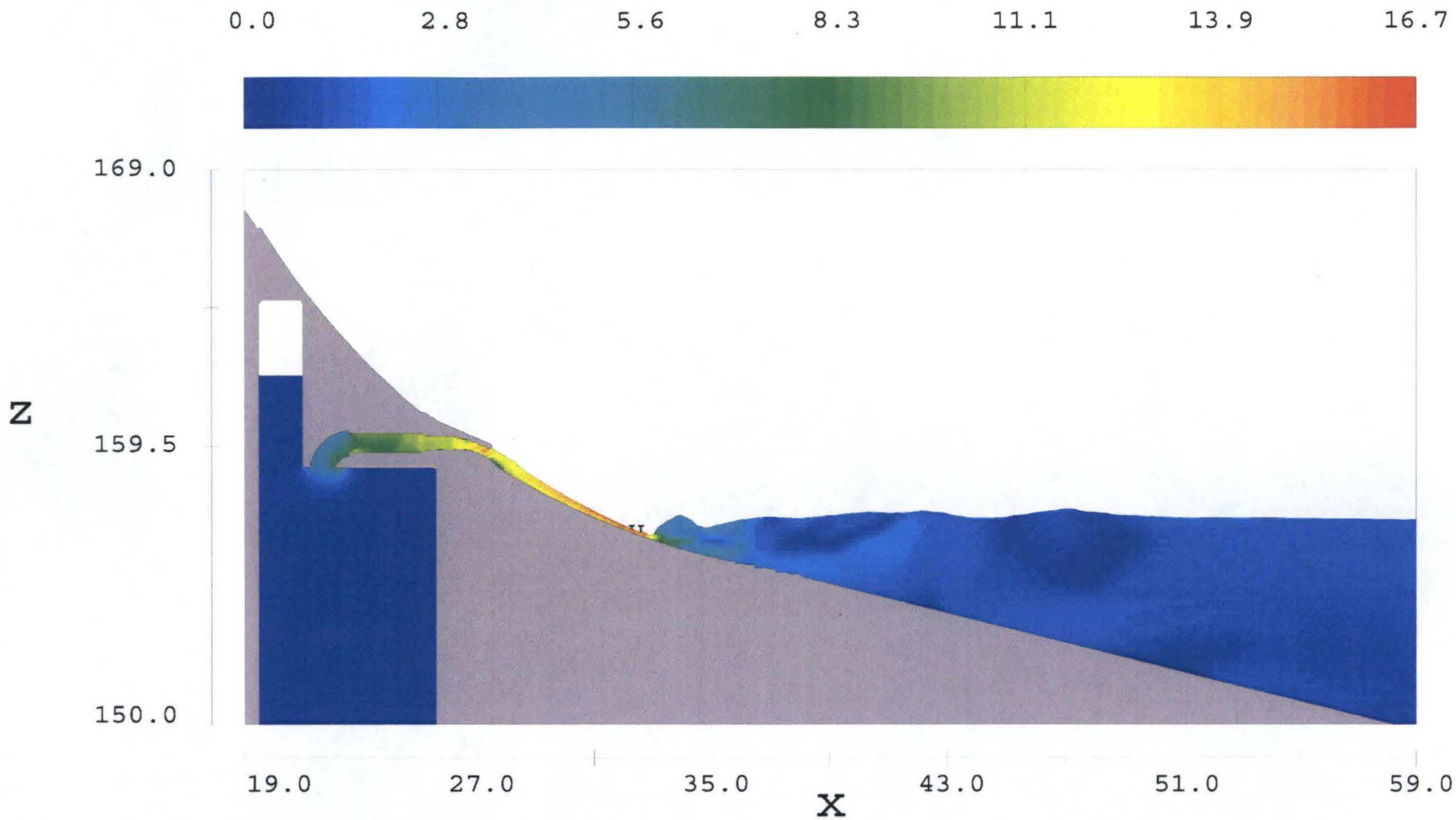

Figure 34. Profile view at the centerline symmetrical Y boundary for the 3D ejector sectional models.

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[6] Flood Frequency Study, Imperial Diversion Dam, Arizona, prepared by Flood Hydrology Group, Water Resources services, Technical Service Center, Denver Colorado, Bureau of Reclamation, U.S. Department of Interior, October 2003
[7] Email, Subject: Imperial Diversion Dam, From: Chuck Sullivan, To: Bob McGovern; James Higgs; Terry Payne, Sent: Thursday, February 26, 2004 8:38 AM