USING HEC-RAS TO MODEL CANAL SYSTEMS

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ABSTRACT

The computer program HEC-RAS can be used to model irrigation canal systems to evaluate canal hydraulics for both steady and unsteady flow conditions. An example application is presented to illustrate how the program can be used to analyze canals with inline structures, inverted siphons, pumping plants, and turnouts. A very useful feature of the program is the ability to illustrate results graphically. For design applications alternative designs can be readily evaluated by using the program option for comparing various combinations of geometry and discharges. RAS permits the use of complex cross section shapes for both open and closed conduit sections. GIS-based maps and aerial photographs can be included with the channel geometry to present realistic depictions of canal alignments, and photographs of structures can be included with geometric data to provide visual references to these features. For unsteady flow analyses the program provides a straightforward and stable solution procedure to evaluate transient flow conditions in canals. An example of unsteady analysis described in this paper is the calculation of surge waves resulting from a pumping plant flow rejection.

INTRODUCTION

The HEC-RAS System

HEC-RAS is an integrated system of software for one-dimension water surface profile computations, and is designed for interactive use in multi-tasking, multi-user network environment. The system is comprised of a graphical user interface (GUI), separate hydraulic analysis components, data storage and management capabilities, graphic, and reporting facilities. HEC-RAS was developed by the Hydrologic Engineering Center, a research group for the U.S. Army Corp of Engineers. The program is freely distributable and can be obtained from the HEC web site: www.hec.uasce.army.mil.

The HEC-RAS system has the capability to perform one-dimensional surface profile hydraulic analysis in both steady state and unsteady conditions.

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The steady flow computational procedure is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning's equation) and contraction/expansion (coefficient multiplied by the change in velocity head). The momentum equation is utilized in situations where the water surface profile is rapidly varied. These situations include hydraulic structures. The effects of various obstructions such as culverts, weirs and other structures can be considered in the computations. Unsteady flow computation procedure is based on continuity and conservation of momentum. The HEC-RAS system is described in the Users Manual, Hydraulic Reference and Applications Guide HEC (2001).

Losses between cross sections in steady flow analysis are the sum of the friction losses and contraction expansion losses. In subcritical analysis, the computations start at the downstream boundary and proceed upstream. The water surface at the next cross section is computed such that the energy loss between the sections is the sum of the friction losses and the contraction and expansion losses. Friction losses are computed with a friction slope from Manning's equation and the contraction/expansion looses are computed a coefficient times the change in velocity head. The friction slope is a conveyance weighted average between the sections.

APPLICATIONS

Application of RAS to the East Branch of the California Aqueduct

The hydraulics of the East Branch of the California Aqueduct were analyzed using a HEC-RAS model (DeVries *et al.* 2004). Present flow conditions and the requirements for a proposed major capacity enlargement were modeled.

The purpose of preparing the HEC-RAS model was to 1) Evaluate the hydraulics of the East Branch canal using the data collected during the 1999 flow test, and 2) analyze the requirements to accommodate proposed enlargement flows (DWR 2002). The model results were used to review and comment on DWR design criteria, model and analysis, and evaluate existing factors influencing hydraulic conditions in the canals and associated structures, including the effect of debris and sediment. In addition, part of the model was used to undertake preliminary work on analyzing unsteady flow conditions and developing a scope of work and cost estimate for a more comprehensive unsteady flow analysis of the East Branch Canal.

Features of the East Branch Aqueduct. The East Branch Aqueduct from Alamo Powerplant to Mojave Siphon is comprised of a series of trapezoidal concrete-lined canals linked by check structures, siphons, and a pumping plant. The system is designed to convey water to State Water Project (SWP) contractors in scheduled amounts according to long-term water supply contracts.

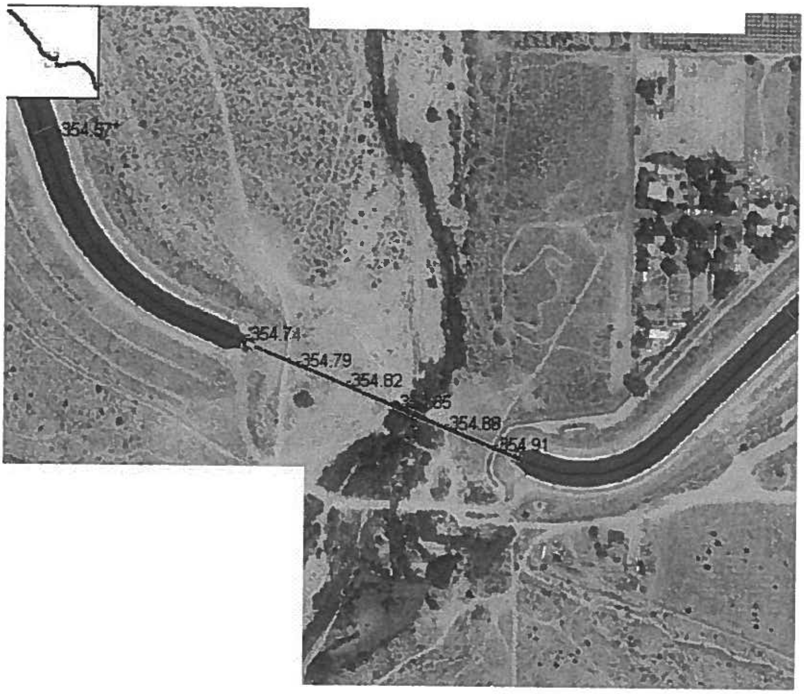

Figure 1. Typical Representation of Canal Alignment and Background Photo in the HEC-RAS Model

Aerial photos of the canal and the adjacent land were assembled into a collage to geo-reference the main physical features of the canal, as shown in Figure 1. A cascade of several hundred images that covered the East Branch Canal was downloaded from the Internet from (http://terraservice.net). For the aerial photos, these images are black and white, with 1-meter pixel resolution, and, for the quad sheets topography, the images are in color with 4-meter pixel resolution. These mosaics where combined into a single image using the MrSid compression technology. The view at every location can be zoomed in or out, depending on the amount of detail required. The sample photo in Figure 1 shows the Check 56 and Little Rock siphon structure.

Originally, DWR constructed the East Branch Canal with a capacity of 1,643 cfs (46.52 cms) at the Alamo Powerplant / Cottonwood Chute Bypass, and 1,376 cfs (38.96 cms) at Pearblossom Pumping Plant. The original facilities were designed to deliver approximately 1,000,000 acre-feet (1,233,000,000 m^3) of water annually, with provision for enlarging the system to accommodate an additional flow of 800 cfs (22.65 cms) without extensive modifications.

Development of the East Branch HEC-RAS model

The layout and geometry of the physical features of the East Branch were taken from several sources including DWR (1997), DWR (2002), a spreadsheet prepared by DWR for analysis of present conditions using data from a 1999 flow test, and engineering drawings of the structures provided by DWR. The HEC-RAS model was modified to accommodate some of the requirements of modeling the trapezoidal East Branch canal and the modifications have been included in HEC-RAS Version 3.1.

In conventional hydraulic analysis of river systems, the system is modeled starting from the downstream end and working upstream. In contrast, the longitudinal distances of the East Branch are referenced to the upstream end of the California Aqueduct as the 'mileage' refers to the distance from the start of the Aqueduct at the Clifton Court Forebay in the Delta. The difference in referencing systems for longitudinal distances was overcome by entering the East Branch mileage into the model as negative values. The approach does not affect the numerical computations of the model, but maintains the familiar reference mileposts.

The East Branch was modeled as two independent reaches, each with a specified flow distribution and downstream boundary conditions. There were 256 user entered cross sections to describe the physical system and 481 interpolated sections to get the computation distance below 1000 feet (305 m). The Check structures 43 to 66 were modeled with the "Inline Structures" option of RAS.

HEC-RAS is a generalized program that typically handles cross sections with hundreds of station elevation points and usually three Manning n values; one for the channel and one for each overbank. Canals have simpler geometry; the cross sections used to describe the East Branch pools were entered with four points and a single Manning n value. A typical HEC-RAS cross-section in Pool 43 is shown in Figure 2.

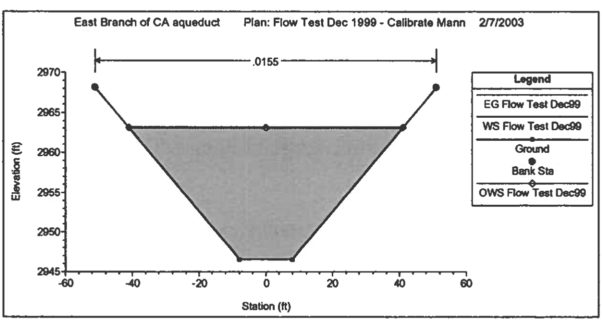

Figure 2. Typical HEC-RAS Cross Section

Two cross sections were entered for each pool, one at the upstream most point and another at the downstream end. Cross sections were then interpolated to get the computation distance less than 1000 feet (305m). Figure 3 shows the interpolation in Pool 43 from the upstream end to the first box siphon.

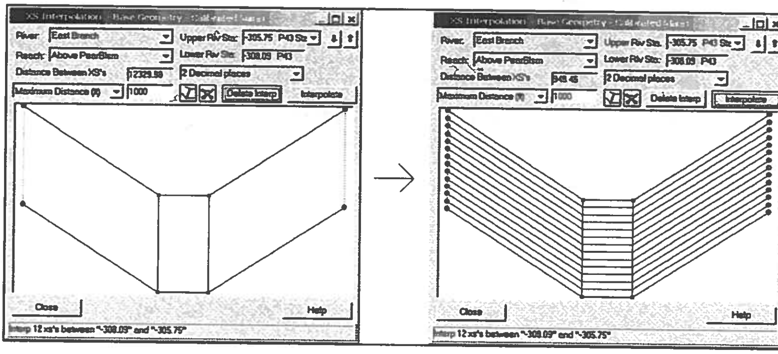

Figure 3. Interpolation of Cross Sections

The box siphons and inverted siphons were modeled as a series of cross sections with lids. This makes the computations energy based and roughly equivalent to standard culvert hydraulics without the inlet control check. The profile plot of the Myrick siphon with Check 46 is shown in Figure 4, and a cross section inside Myrick siphon is shown in Figure 5.

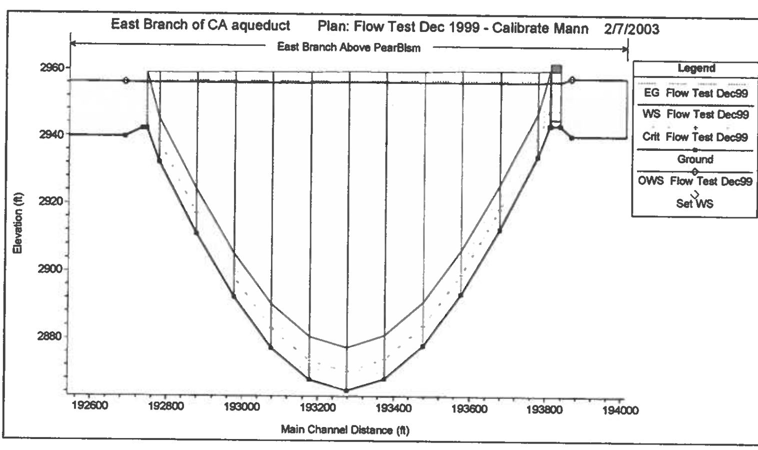

Figure 4. HEC-RAS Profile of Check 46 and Myrick Siphon

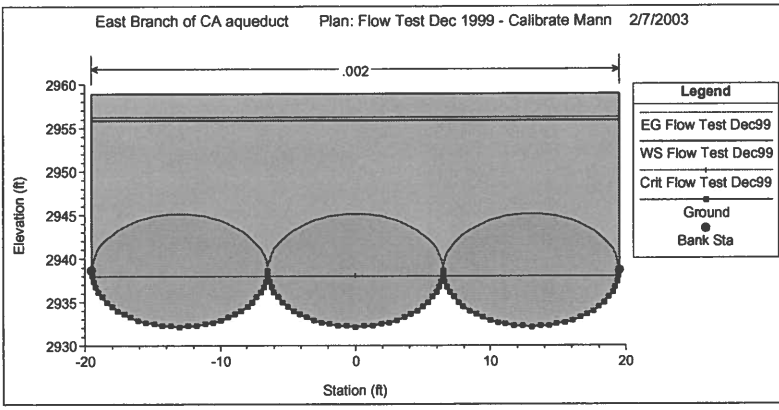

Figure 5. HEC-RAS Cross Section of Myrick Siphon

Check structures were modeled with the "Inline Structure" option. The inline structure allows for an overflow weir (not used in East Branch model) and a series of independently controlled gates. Figure 6 shows the "cross section" view of check 43 with the gates 10 feet open (3.05 m). The darker section below represents the sills and the very dark section above represents the partially closed gates.

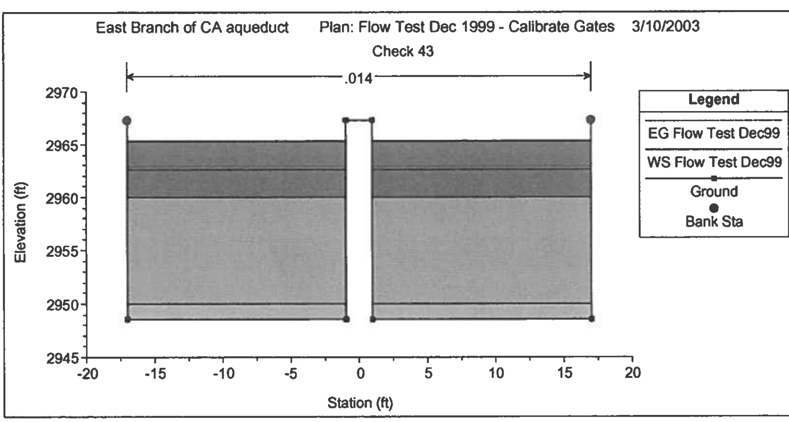

Figure 6. HEC-RAS Cross Section of Check 43

ANALYSIS

Analysis of Present Canal Performance

A flow test to determine aqueduct capacity was conducted by DWR in 1999. The flow test data were used as the water level and flow input data for the model. The water levels during the flow test were taken as the average of the water levels measured during the 2-hour data collection period while nearly steady state conditions existed. The discharge was 2010 cfs (56.92 cms) at Pearblossom Pumping Plant. The expansion and contraction loss coefficients used in analyzing the flow test data were 0.3 and 0.1 respectively. The friction loss coefficients are shown in Table 1.

Table 1. Friction Coefficients used in East Branch Model

Location	Manning n-value from calibration	Roughness height	
		k (ft)	Effective n-value
Pools	0.0154 - 0.0193		
Transition sections	0.014		
Box siphons		0.003	0.0136
Circular siphons		0.002	0.0130

The Manning n values for the transition structures and the siphons were not determined by calibration since head loss data for these structures was not available.

Calibrated Manning n-values are in the range of 0.0154 - 0.0193, these values are consistent with values observed by the USBR tests supported by Tilp (1965). A key issue with interpreting flow data is to recognize that flow and water level measurements may be imprecise, and it is necessary to carry out sensitivity analysis to determine the effects of data measurement errors on the calculations. Therefore, the HEC-RAS model was run to calculate Manning n-values for +/- 5% variation in the flow. The sensitivity analysis indicated that if flow was 5% lower, the estimated increase in Manning n-value would be about 6%. Similarly, a 5% increase in flow reduces the Manning n-value by about 4%. The Manning n values calculated by DWR are within the 5% error bands of the values calculated by HEC–RAS corroborating the different analysis method (DeVries *et al.* 2004).

Analysis of Transient Flows at Pearblossom Pumping Plant

An unsteady flow analysis was made to simulate the surges that would occur in the canal if electrical power supplied to Pearblossom Pumping Plant were suddenly cut off causing a complete flow rejection at the plant. This condition is simulated in the model by assuming the flow at the downstream end of the canal pool was decreased to zero in one minute. The initial flow in the canal is 3000 cfs

(84.95 cms) with a starting water surface elevation at the plant intake of 2941.00 feet (896.42 m).

Several conditions for reacting to this sudden canal shut down were simulated:

1. A **rapid reaction scenario** in which the gates in the check structure at the upstream end of the reach were closed starting four minutes after the power failure. The gates are closed in ten minutes assuming a linear change in flow with time. The maximum rise is to elevation 2943.55 feet (897.19 m) at about 19 minutes after plant shutdown. It is at this time that the effect of closing the check gates upstream is first experienced at the downstream end of the reach.

2. A **time delay scenario** in which the operators wait to begin the closure of the gates at the upstream check structure for 20 minutes after the power failure. The gates are closed in ten minutes.

The time delay scenario is discussed in detail below.

Time delay scenario. The scenario in which the operators wait to begin the closure of the gates at the upstream check structure may be more realistic than the rapid response case because there is often a possibility that the plant can be brought on line again without needing to shut down the system. In this case, it is assumed the canal shutdown begins 20 minutes after the power failure because it was not possible to re-start the plant. The gates are closed in ten minutes Figure 7 shows the water surface plot for this case.

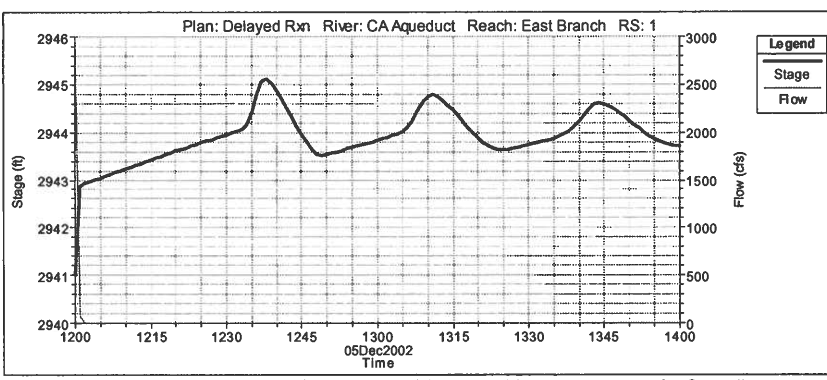

Figure 7. Transient conditions at Pearblossom (downstream end of pool)

The initial water surface is again at 2941.0 feet (896.41 m) and the initial part of the surge at the time of closure is to elevation 2942.85 feet (896.98 m). However, the flow from upstream continues at 3000 cfs (84.95) for about 20 minutes after the closure. This produces a constantly rising water surface until about 35 minutes after the positive surge was first generated. The maximum water surface rise is just over 4 feet (1.2 m) above the initial water surface.

During this time the positive wave has traveled to the upstream end of the pool. It was reflected as a positive wave at the gate and then traveled as a positive wave to the downstream end. This reflected wave causes a further rise in the water surface at Pearblossom to elevation 2945.12 feet (897.67 m) at 38 minutes after the flow rejection at the pumping plant. The water surface begins dropping at this time as a result of the negative wave produced at the upstream end of the pool.

Conditions at the upstream end are illustrated in Figure 8. The gate begins closing just about the time that the positive wave generated at the downstream end arrives at the upstream end of the reach. This can be noted by observing the simulation in profile. The negative wave produced as the gate is closing counteracts the positive wave from downstream, and this effect continues until the gate is fully closed. After this time the shape of the water surface plot is similar to that for the downstream location.

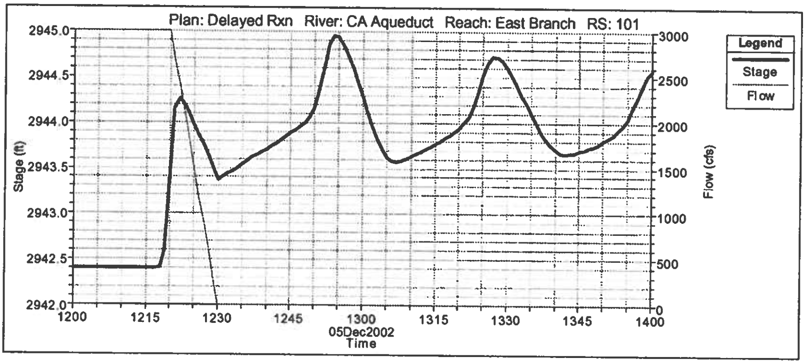

Figure 8. Transient conditions at Pearblossom (upstream end of pool) – Time delay

At the upstream end of the pool, the water surface reaches a maximum elevation of 2944.95 feet (897.62 m) about 54 minutes after plant shutdown. This is a rise of about 2.5 feet (.76 m) at the upstream end of the pool. It should be noted that a significant part of the water surface rise is due to the increase in water volume in the pool due to the delayed closure of the upstream gates.

Expansion Variations – Vertical Walls. The East Branch is being studied for a flow expansion and the RAS model can be used to evaluate various channel configurations. A good portion of the additional height needed for the expanded flows is to contain the transient waves caused by pumping plant load rejections. The expanded channel configurations discussed in this study were based on an enlargement of the trapezoidal channel. An alternative (and perhaps cheaper) expansion was evaluated that increased the channel capacity with the addition of vertical walls placed at the top of the existing channel. The vertical walls were added to the bounding cross-sections in the pool and the internal cross-sections were re-interpolated so that they would have the wall as well. The maximum difference between the trapezoidal expansion and the vertical wall expansion was 0.10 feet (.03 m) and average 0.05 feet (.015 m) for the pool.

These scenarios illustrate how the unsteady flow computation feature of the HEC-RAS model can be used to evaluate various emergency conditions in the East Branch Aqueduct. A wide variety of conditions can be modeled to assist in the formulation of emergency operation procedures and the evaluation of canal freeboard and other design consideration associated with the aqueduct enlargement.

ACKNOWLEDGEMENTS

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