Unsteady-Flow Modeling for Emergency Shutdown of the CAP Canal

Bert Clemmens
bclemmens@westconsultants.com

Brian Wahlin
West, bwahlin@westconsultants.com

Marcus Shapiro

Patrick Dent
pdent@cap-az.com

Follow this and additional works at: https://digitalcommons.usu.edu/ishs

Part of the Hydraulic Engineering Commons

Recommended Citation
Unsteady-Flow Modeling for Emergency Shutdown of the CAP Canal

Bert Clemmens¹, Brian Wahlin¹, Marcus Shapiro² and Patrick Dent²

¹WEST Consultants, Inc.
Tempe, AZ 85284
USA

²Central Arizona Water Conservancy District
Phoenix, AZ 85024
USA

E-mail: bwhalin@westconsultants.com

ABSTRACT

The Central Arizona Project (CAP) is designed to bring about 1.9 ML of Colorado River water per year to Maricopa, Pima, and Pinal counties in Arizona. CAP carries water from the Colorado River at Lake Havasu to Tucson. The CAP canal system is a 540 km long system of conveyance system aqueducts, tunnels, pumping plants, pipelines, and a large reservoir (just north of Phoenix, AZ). Water is pumped uphill from the Colorado River. This study was conducted for the Hayden-Rhodes Aqueduct of the Central Arizona Project (CAP), which starts at the afterbay of the Bouse Hill Pumping Plant and extends to the forebay of the Salt-Gila Pumping Plant, which is just south of the Salt River, east of Phoenix. Additional pumping plants downstream lift water uphill to Tucson. The canal between adjacent pumping plants defines a canal segment. A sloping lined canal with check structures carries water between power plants. The canal between adjacent check structures defines a canal pool. The Pumping Plants use electric motors to power the pumps. Power outages could cause the canal to overtop at the forebay of these power plants, since the flow in the canal would not stop with a power outage. The check structures in between the power plants can be closed with battery-operated electric motors. This was intended to contain water within each canal pool if power is lost. Stopping the pumping plants and closing the check gates causes surge waves in the canal. Some additional freeboard at the downstream end of each canal pool was provided to contain these surge waves. Increases in frictional resistance in the canal have reduced the available canal freeboard from design conditions. This study evaluated the available freeboard in the canal during steady conditions and during unsteady conditions associated with a power outage. HEC-RAS was used to determine water surface elevations and available freeboard under both steady and unsteady conditions.

Keywords: Irrigation systems, canal and diversion structures, canal management, operator training

1. INTRODUCTION

1.1. The Central Arizona Project

The Central Arizona Project (CAP) is Arizona’s largest renewable water supply and was constructed to help the state conserve its groundwater by importing surface water from the Colorado River. CAP was designed to deliver an average of 1.9 ML of water per year to residents of Maricopa, Pima, and Pinal counties (see Figure 1), making it a critical economic lynchpin for the region. CAP delivers untreated water to three major types of customers: municipal and industrial, agricultural, and Native American users. The customers are then responsible for their own water treatment. The CAP canal travels 540 km across the state of Arizona. The canal begins at Lake Havasu, continues through the Phoenix metropolitan area, and ends in Tucson. CAP consists of 14 pumping plants, 1 pump/generating plant, 10 siphons, 3 tunnels, and more than 45 turnouts for customer deliveries. During its travels across the state of Arizona, water is pumped more than 850 vertical meters and flows through the canal via gravity following the natural contours of the land. Construction of the system began in 1973 and was substantially complete in 1993. The CAP Canal System is operated by the Central Arizona Water Conservancy District (CAWCD).
1.2. The Hayden-Rhodes Aqueduct

The Hayden-Rhodes Aqueduct portion of the CAP starts at the afterbay of the Bouse Hill Pumping Plant and extends to the forebay of the Salt-Gila Pumping Plant, which is just south of the Salt River, east of Phoenix. The aqueduct has two additional pumping plants, the Little Harquahala Pumping Plant and the Hassayampa Pumping Plant. These two pumping plants lift the water roughly 60 m vertically. Additional pumping plants downstream lift water uphill to Tucson. The canal between adjacent pumping plants defines a canal segment. A sloping lined canal with check structures carries water between power plants. The aqueduct has 7 inverted siphons under river beds and 2 tunnels. The profile of the canal invert is shown in Figure 2, along with labels for the pumping plants and major siphons.

- Segment 1 is the canal between the Bouse Hill Pumping Plant and the Little Harquahala Pumping Plant.
- Segment 2 is the canal between the Little Harquahala Pumping Plant and the Hassayampa Pumping Plant.
- Segment 3 is the canal between the Hassayampa Pumping Plant and the Salt-Gila Pumping Plant.

Segment 1 includes four check structures, CS-01 to CS-04. The canal between adjacent check structures or between a check structure and a pumping plant defines a canal pool. Thus Segment 1 has 5 pools. Segment 2 has 9 check structures, CS-05 to CS-13, and 10 pools. Segment 3 has 12 check structures, CS-14 to CS-25, and 13 pools. The pumping plants use electric motors to power the pumps. Power outages could cause the canal to overtop at the forebay of these power plants since the flow in the canal would not stop with a power outage. The check structures in between the power plants can be closed with battery-operated electric motors. This was intended to contain water.
within each canal pool. Stopping the pumping plants and closing the check gates causes surge waves in the canal. Some additional freeboard at the downstream end of each canal pool was provided to contain these surge waves. Increases in frictional resistance in the canal have reduced the available canal freeboard from design conditions and might cause canal and/or check gate overtopping during power outages. These check gates were designed to overtop in an emergency.

The Hayden-Rhodes Aqueduct also includes the Waddell Canal, which connects Waddell Dam to Segment 3 between the Agua Fria Siphon and the New River siphon. The canal from Waddell Dam to the Salt-Gila Pumping Plant was called Segment 3M. Unsteady hydraulic simulation results for Segment 3M and the Waddell Canal are not discussed in this paper. The section of Segment 3 upstream of the Waddell Canal is called Segment 3 upper, and the section of Segment 3 downstream of the Waddell Canal is called Segment 3 lower.

Separate HEC-RAS (2010) models were developed for each Segment. In some cases, additional cross sections were added for the unsteady portion of the model to increase model stability. This effectively resulted in separate models for steady and unsteady flow.

![Profile of the Hayden-Rhodes Aqueduct](image)

**Figure 2. Profile of the Hayden-Rhodes Aqueduct.**

2. **STEADY-STATE WATER SURFACE PROFILES**

The design capacity of the Hayden-Rhodes Aqueduct was 935 m$^3$/s. The canal was designed so that the water was at normal depth at all locations when flow was at capacity; there is no backwater from the check gates. In fact, at high flows, operators often take the check gates out of the water. The design water depth was 5.0 m, for a design Manning $n$ value of 0.016. The design freeboard was approximately 0.67 m, for a lining height of roughly 5.67 m. This lining height was increased to 5.94 m just upstream from check gates to contain surge waves, which are discussed in the section on emergency shutdown. (Note that these dimensions varied for a few canal pools.) As-built construction drawings were available, from which the cross sections and structure details were developed for HEC-RAS. In addition to this information, development of steady state profiles with HEC-RAS requires knowledge of Manning $n$ values for the canal sections, Manning $n$ values for the inverted siphons, and calibration of the check structure gates. Steady state profiles were developed for 935 m$^3$/s, 966 m$^3$/s, 997 m$^3$/s, and 1028 m$^3$/s.
2.1. Canal Manning n Values

The discharges at the pumping plants are measured in the outlet pipes with multipath ultrasonic meters. Under steady conditions, it is possible to approximate the discharge in each canal pool, from which estimates of Manning n can be determined. CAWCD staff had compiled many years’ worth of data from which they estimated Manning n values for each canal pool. These estimates were provided as input to this study. For pools 1 through 6, the Manning n value was set to 0.018. Manning n values for pools 7 through 18 and the Waddell Canal were 0.0175. Manning n values for pool 19 through the Salt-Gila Pumping Plant were set to 0.017. Note that these are above the design value of 0.016.

2.2. Siphon Manning n Values

Siphons and tunnels were modeled in HEC-RAS as channels with lidded cross sections. Preissman slots were added to the top of these cross sections to account for water pressure above the top of the siphon or tunnel. These models were calibrated at 935 m$^3$/s as follows: Manning n values for siphons and tunnels were determined through trial and error with both the steady and unsteady HEC-RAS models to match the head losses provided by CAWCD at 935 m$^3$/s. The resulting Manning n values are given in Table 1. Minor differences in Manning n values were found for the steady and unsteady models.

<table>
<thead>
<tr>
<th>Site</th>
<th>Measured Head Loss (m)</th>
<th>Manning n values from CAWCD</th>
<th>Manning n values from calibrated steady flow</th>
<th>Manning n values from calibrated unsteady flow</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cunningham Wash</td>
<td>1.07</td>
<td>0.0159</td>
<td>0.0159</td>
<td>0.0159</td>
</tr>
<tr>
<td>Centennial Wash</td>
<td>1.29</td>
<td>0.0145</td>
<td>0.0137</td>
<td>0.0141</td>
</tr>
<tr>
<td>Burnt Mt Tunnel</td>
<td>0.63</td>
<td>0.0130</td>
<td>0.0124</td>
<td>0.0122</td>
</tr>
<tr>
<td>Jackrabbit Wash</td>
<td>0.47</td>
<td>0.0179</td>
<td>0.0169</td>
<td>0.0162</td>
</tr>
<tr>
<td>Hassayampa River</td>
<td>0.84</td>
<td>0.0149</td>
<td>0.0143</td>
<td>0.0141</td>
</tr>
<tr>
<td>Agua Fria Tunnel</td>
<td>0.85</td>
<td>0.0130</td>
<td>0.0119</td>
<td>0.0117</td>
</tr>
<tr>
<td>Agua Fria River</td>
<td>1.98</td>
<td>0.0141</td>
<td>0.0135</td>
<td>0.0134</td>
</tr>
<tr>
<td>New River</td>
<td>1.15</td>
<td>0.0139</td>
<td>0.0135</td>
<td>0.0127</td>
</tr>
<tr>
<td>Salt River</td>
<td>1.74</td>
<td>0.0134</td>
<td>0.0135</td>
<td>0.0134</td>
</tr>
<tr>
<td>Waddell Siphon</td>
<td>0.60</td>
<td>0.0161</td>
<td>0.0149</td>
<td>0.0150</td>
</tr>
</tbody>
</table>

2.3. Check Structure Calibrations

CAWCD staff had developed relationships for each check structure gate with discharge over gate openings plotted against the difference in the upstream and downstream depths. This function sometimes had positive discharge even when the depth difference was negative. The upstream and downstream depths had not been reconciled to express elevation difference, so the functions developed did not match physical conditions. These relationships were useful for gate operations, but attempts to use them in HEC-RAS were not successful. WEST and CAWCD agreed on the following method to determine gate openings and gate calibrations:

- The standard HEC-RAS equations for gates were used to compute discharge in the model. The submerged discharge coefficient was set to 1.0.
- For gates without siphons or tunnels, the gate opening was adjusted to provide a drop in water surface elevation of roughly 0.046 m. (In most cases, this was approximately the same as the drop in energy head).
- For gates with siphons or tunnels, the gate opening was adjusted to provide a minimum drop in energy head of roughly 0.037 m. For lower flows, meeting the target water level resulted in much higher energy differences. At high flow, the change in energy head was often the governing criteria.
2.4. Steady-State Profile Results

With the above Manning $n$ values and gate calibrations, gate openings were determined by trial and error. The depth upstream from the checks with a siphon was the maximum of normal depth or 5.33 m, with a head loss greater than 0.037 m. The depth upstream from other checks was based on the elevation drop of roughly 0.046 m. These criteria were sufficient to determine water surface profiles. Water surface profiles were generated for 935 m$^3$/s, 966 m$^3$/s, 997 m$^3$/s, and 1,028 m$^3$/s. Figure 3. shows the steady-state profile for segment 3 at the design discharge of 935 m$^3$/s. The increase in lining height just upstream from check structures can be seen as a stair-step in the lining. Additional lining increases can be seen between CS-22 and CS-24, where the lining was increased due to subsidence.

Figure 3. Steady-state profile for Segment 3 at design discharge of 935 m$^3$/s.

Figure 4 shows the freeboard for Segment 3 at design discharge. One location below the allowable freeboard of 0.304 m can be seen just upstream from CS-24. This is likely an area of subsidence.
2.5. Steady State Freeboard Results

Operational freeboard was defined by CAWCD as 0.305 m. The freeboard under steady flow was used to determine capacity. For some segments, the minimum freeboard was essentially equal to the required freeboard at one of the modelled discharges. In one case, the capacity could be determined from the Manning n value. In other cases, it was below 935 m$^3$/s, and capacity could only be determined by trial and error. This was not deemed necessary for this study. The distance over which the canal did not meet operational freeboard for the specified discharge was also determined for each canal segment. These results are given in Table 2. The lining deficiency often went from zero to a few centimeters.

Table 2. Length of canal (km) where lining needs to be increased to meet operational freeboard.

<table>
<thead>
<tr>
<th>Canal</th>
<th>@ 935 m$^3$/s (km)</th>
<th>@ 966 m$^3$/s (km)</th>
<th>@ 997 m$^3$/s (km)</th>
<th>@ 1028 m$^3$/s (km)</th>
<th>Capacity (m$^3$/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Segment 1</td>
<td>6.1</td>
<td>24.0</td>
<td>40.1</td>
<td>43.8</td>
<td>916</td>
</tr>
<tr>
<td>Segment 2</td>
<td>0.0</td>
<td>17.2</td>
<td>50.9</td>
<td>75.3</td>
<td>935</td>
</tr>
<tr>
<td>Segment 3 upper</td>
<td>0.0</td>
<td>0.0</td>
<td>22.0</td>
<td>31.5</td>
<td>966</td>
</tr>
<tr>
<td>Segment 3 lower</td>
<td>0.5</td>
<td>0.6</td>
<td>1.3</td>
<td>48.0</td>
<td>&lt; 935</td>
</tr>
<tr>
<td>Waddell Canal</td>
<td>0.8</td>
<td>1.6</td>
<td>4.0</td>
<td>6.3</td>
<td>&lt; 935</td>
</tr>
<tr>
<td>Total</td>
<td>7.4</td>
<td>43.5</td>
<td>118.3</td>
<td>204.9</td>
<td></td>
</tr>
</tbody>
</table>

At 1028 m$^3$/s, the water surface is above the canal lining at CS-01 in Segment 1 and CS-05 and CS-06 in Segment 2 (negative freeboard).
3. EMERGENCY SHUTDOWN

With the unsteady-flow HEC-RAS models, pump discharge at the upstream and downstream end of these canal segments was stopped for one minute, and gates were closed over a 10-minute period to simulate an emergency shutdown. The reduction of inflow and outflow in each canal pool caused a surge in the water levels of the pool and reflection waves that traveled both upstream and downstream. As discussed earlier, the lining height at the check structures was increased (to 5.94 m at most pools to accommodate the maximum water height from this surge wave. With higher resistance (Manning n and larger flow, it was expected that the water would exceed the lining height during these surges. The design elevation of the top of the road was 0.67 m above the top of the lining. In many pools, this was the top of the embankment that contained water in the canal. The hope was that the water surface elevation during surges would be contained within the canal and not spill. The current elevations of these roads/embankments were not surveyed, so these results rely on the design intent.

Each check structure contains two check gates. The height of the check gates is 5.94 m, the nominal height of the lining for most canal pools at the check gate. Thus, if the surge wave exceeded the lining height, it was usually higher than the check gate height, resulting in flow over the check gates. This could be a concern since water might accumulate in the most downstream pool of a segment. Fortunately, the lining height at the forebay to pump stations was 6.55 m. With the gates closed, the lined canal section is supposed to contain the water within each pool. The water depths for each pool were examined 16 hrs after shutdown to see if this occurred.

3.1. Surge Wave Heights

In general, the maximum water surface elevation was at the downstream end of each canal pool, at the gate forebay. By design, the lining at the downstream end of each pool was increased to accommodate this surge. These increases in lining height generally matched the surge height fairly well. However, the highest water surface elevation relative to the top of lining elevation was generally upstream at one of the transitions in lining height. Usually the highest water surface elevation relative to the top of lining elevation was where the increase in lining height first started. In a few cases, it was where the second stair-step in lining height occurred.

At the design discharge of 935 m$^3$/s, the maximum water surface elevation exceeded the lining height at the downstream end of all pools even though additional lining height had been provided to accommodate these surges during original construction. This is caused by operational water depths that are greater than the design values, mostly resulting from Manning n values that are greater than the design value. These short-duration surge waves that exceed the top of the lining are not a concern, since the velocity is mostly vertical and does not appear to cause scour. The maximum water surface elevations from emergency shut down at 935 m$^3$/s in Segment 1 are shown in Figure 5.
Figure 5. Maximum water surface elevation resulting from emergency shut down in Segment 1 at 935 m$^3$/s.

Up to 997 m$^3$/s, the maximum water surface elevation did not exceed the top of the road in any cases. At 1,028 m$^3$/s, the top of the road was exceeded at Segment 2, CS-06. The depth was within 0.02 m of the top of road at CS-01, CS-05, and CS-11. Thus, increasing the discharge above the design discharge has the potential for aqueduct overtopping during an emergency shut down. The maximum water surface elevations from emergency shut down in Segment 1 are shown in Figure 6. At the scale of these figures, it is hard to determine how close the water level is to the top of the road.

3.2. Ability to Contain Pool Volume

For design conditions with an initial flow of 935 m$^3$/s, some flow did pass over the gates during the surge waves from emergency shutdown, but the water after 16 hrs was below the top of all the gates in all the segments and below the top of the lining. The water surface profile 16 hours after shut down for Segment 2 is shown in Figure 7. Note that the water surfaces are below the top of lining, and thus below the top of the check gates. In this case, no significant oscillations remain in the water surface.
At 1,028 m$^3$/s, the depth at 16 hrs exceeded the top of the radial gates at three checks in Segment 1, 3 checks in Segment 2, and 0 checks in Segment 3. The hydrograph for flow over the gates at 1,028 m$^3$/s shows water cycling over about a 12 hr period as the water depth in the pool continues to cycle, as shown in Figure 8 for Segment 2. For most gates, the flow dropped to zero as the waves were on the low side at the gate. Two check gates showed flow over the gates continuously. Some pools had lining heights that were lower than the top of the gates. The water depth was above the lining height but below the top of the gates at 16 hrs in 1 pool each for Segments 1 and 2 (these were just upstream from the pumping plants) and for 3 pools in Segment 3. Water depths exceeding the lining height for extended time periods are a concern since this could lead to embankment failure and spill.
4. SUMMARY

The HEC-RAS models were able to determine steady state profiles, available capacity, and available freeboard for the Hayden-Rhodes Aqueduct. The discharge capacity of the aqueduct was sufficiently close to the design discharge, such that the design discharge could be provided with only modest reductions in freeboard. These estimates are highly influenced by frictional resistance and highlight the need for CAWCD to be diligent in keeping frictional resistance on the canal walls within acceptable values. Additional lining is needed in a few key locations, but increasing the lining height by a decimeter or less over many kilometres is impractical.

With flow at the design discharge of 935 m$^3$/s, the Hayden-Rhodes Aqueduct has slightly less than 0.305 m (1 ft) of freeboard. At 110% of the design capacity (1028 m$^3$/s), water will overtop the canal lining in Segments 1 and 2. This highlights the importance of maintaining the canal walls to reduce friction.

During emergency shutdown from the design discharge of 935 m$^3$/s, the maximum water surface elevation exceeds the canal lining within all canal pools. At 110% of the design discharge, the maximum water surface elevation overtop the canal roadway at several locations during emergency shutdown. Further analysis would have to be done to determine if these water surface elevations would cause canal embankment overtopping and spill.

During emergency shutdown from the design discharge of 935 m$^3$/s, the canal water volume is contained within each canal pool even though some water spills temporarily over some check gates. At 110% of the design discharge, the canal water volume continues to flow downstream over check gates and possibly over the canal embankment. It is unclear whether or not the canal volume is contained within the canal if an emergency shut down occurs at this discharge.

5. REFERENCES