Nevada Irrigation District

Staff Report

for the Board of Directors Meeting of June 8, 2016

TO: Board of Directors

FROM: Remleh Scherzinger, General Manager
Gary King, Engineering Manager

DATE: June 1, 2016

SUBJECT: **Hemphill Diversion Structure Alternatives Analysis**

ENGINEERING

RECOMMENDATION:
Recommend Alternatives 4 and 5 from the Analysis for the existing Hemphill Diversion facility, as recommended by the Engineering Committee.

BACKGROUND:
After the completion of the Gaging Station fish passage project, environmental groups and regulatory organizations involved with the fish in Auburn Ravine have indicated that a significant issue existed with the passage of fish above the Hemphill facility. Stakeholder groups and regulatory agencies have indicated a desire for the District to resolve this issue. Despite concerns, no grant funding to assist with this issue has been obtained from regulatory agencies even though applications have been made for financial assistance at their request.

Recently, the District and its consultant have completed an engineering alternatives study to analyze viable alternatives that would allow the continued operation of Hemphill canal. The study is attached to this Staff Report and has been developed to be attached to a future Environmental Impact Report (EIR) if it is determined to move forward with a selected project.

The report has a recommended Alternative 4 River Bank Infiltration with a Pumps System with the removal of the existing diversion. The project alternatives were presented to the Engineering Committee, and the Committee requested that staff move forward with the recommended Alternative 4 and to further evaluate Alternative 5. If the Board concurs with Alternatives 4 and 5, the District will move to the next phase of design and permitting.

As part of the next phase, the District will pursue concurrence with regulatory agencies and begin preparation of a California Environmental Quality Act (CEQA) Environmental Impact Report (EIR) and property rights for the selected project.

Once the EIR is complete, we will complete a complex list of permitting with the appropriate regulatory agencies.

Once all permits are secured, we will proceed with construction of a replacement facility, and the existing facility would be removed from service. This out of service facility has been identified as a partial fish passage barrier and would need to be removed. This engineering study did not review the full impacts of removal of the existing diversion facility which will be addressed as part of the EIR.

This project will be a complex process, and as stated in numerous meetings, the District is committed to resolving this issue. However, to achieve this goal we will need support from numerous organizations to complete a project of this scale in Auburn Ravine. Numerous organizations were in attendance at the May Engineering Committee.

After this Board meeting, staff will post the Alternatives report, and power point from the Engineering Committee meeting placed on the Projects Section of the Districts website.

In addition in October of 2015, Placer County District Attorney, Jane Crue contacted the District for a meeting regarding the diversion facility. As a result of numerous meetings, it was agreed that this report will be provided to Placer County by the end of June 2016.

BUDGETARY IMPACT:
The District issued work order #7032 for this project and to date, we have spent $92,333.64 on this project.

GDK

HEMPHILL DIVERSION STRUCTURE ALTERNATIVES ANALYSIS

Prepared for:

Nevada Irrigation District
Grass Valley, California

Prepared by:

Kleinschmidt

Pittsfield, Maine
www.KleinschmidtGroup.com

April 2016

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HEMPHILL DIVERSION STRUCTURE
ALTERNATIVES ANALYSIS

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1.0 INTRODUCTION

Nevada Irrigation Districts (District) Hemphill diversion has been utilized by the District since its purchase of the facility in 1933. The structure as it exists today is an approximately 8-foot-tall concrete structure located in Auburn Ravine near the community of Lincoln in Placer County, California. Historically, the structure has been fitted with 3-foot-tall flashboards during the irrigation season (April to October) to increase the water surface elevation upstream and direct flows into the Hemphill canal.

The canal intake is located 40 feet upstream of the structure on river-left (looking downstream). Historic District flow data from the Hemphill canal gauge (BR 220) indicated that the average daily canal flow during irrigation season ranged from 6 to 16 cubic feet per second (CFS). Flows in Auburn Ravine at the Highway 65 gauging station downstream of the diversion structure can range from 10 CFS to 180 CFS during irrigation season. The peak flow data noted in the Raw Water Master Plan (RWMP) indicated that the Hemphill canal would have a peak demand of approximately 18 CFS by the year 2032.

This report identifies and assesses conceptual alternatives for continuing to provide water to the Hemphill canal with or without the Hemphill diversion structure in order to meet customer demand. Options examined ranged from a “do nothing” or status quo alternative to complete removal of the Hemphill diversion structure and included both mechanical (pumping) and non-mechanical options. This assessment includes a review of technical viability and potential costs. No detailed consideration was given to environmental issues, such as sediment transport, which could affect any design and/or operation. Once this study is complete, the District will have their environmental consultant review potential solutions to assess the environmental impacts. The available site data and options reviewed are summarized in the following sections.

2.0 HYDROLOGY DATA

Daily data was obtained from the District for both gages for the irrigation season from 2005 until 2015. The data from the two gages was combined to provide the total daily stream flow at the structure.

The gage data shows that historically flows in Auburn Ravine near the Hemphill structure range from near 5.0 CFS to a 180 CFS. Figure 2-1 shows a flow duration curve for the irrigation season (May to October) for the period of record (2005 - 2015) for flows upstream of the structure (i.e., canal flow plus flow over the structure). Figure 2-2 shows the typical distribution of daily flows in Auburn Ravine near the structure during irrigation season. The figure indicates that sufficient flow comprised of both natural and imported water is typically present in Auburn Ravine during the irrigation season to meet the demands from the Hemphill canal. The forecast peak flow in 2032 was noted to be approximately 18 CFS for the Hemphill canal, up from 2007 peak flow demand of 12 CFS.

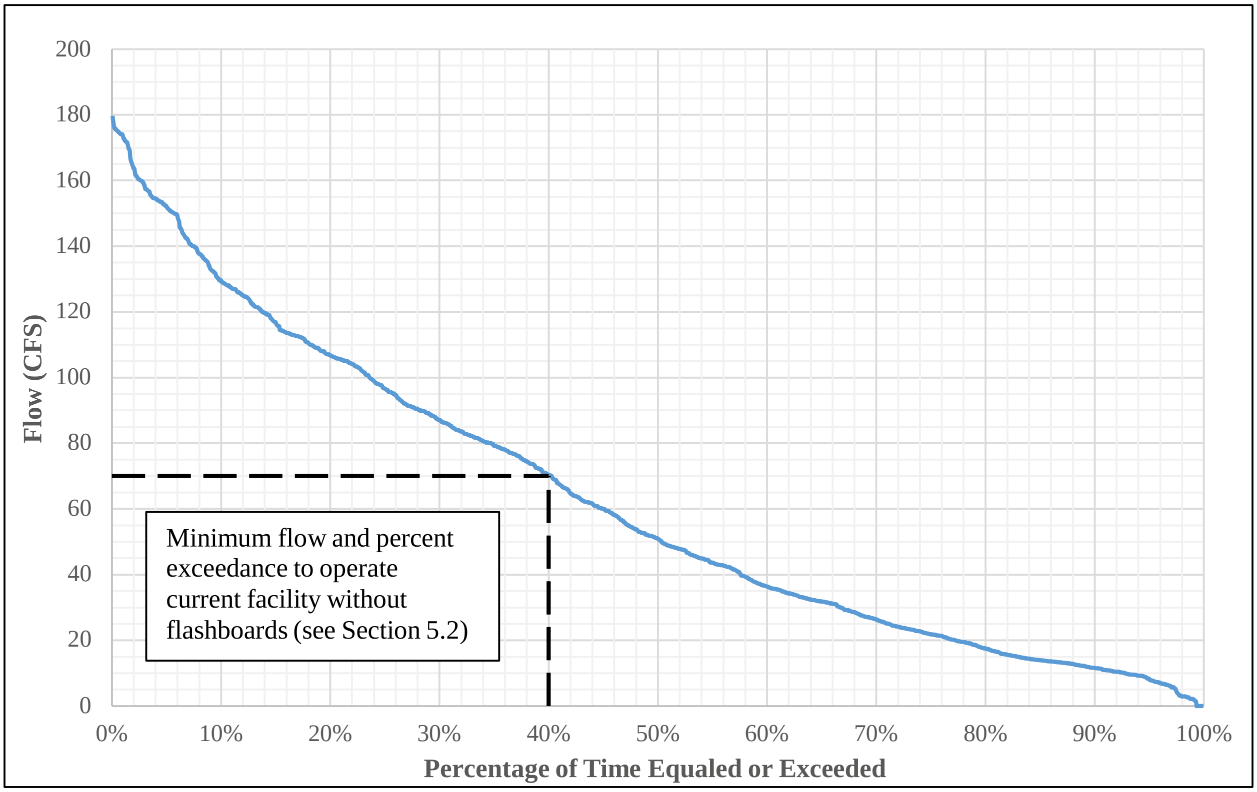

FIGURE 2-1 AUBURN RAVINE IRRIGATION SEASON (MAY - OCTOBER) FLOW DURATION CURVE

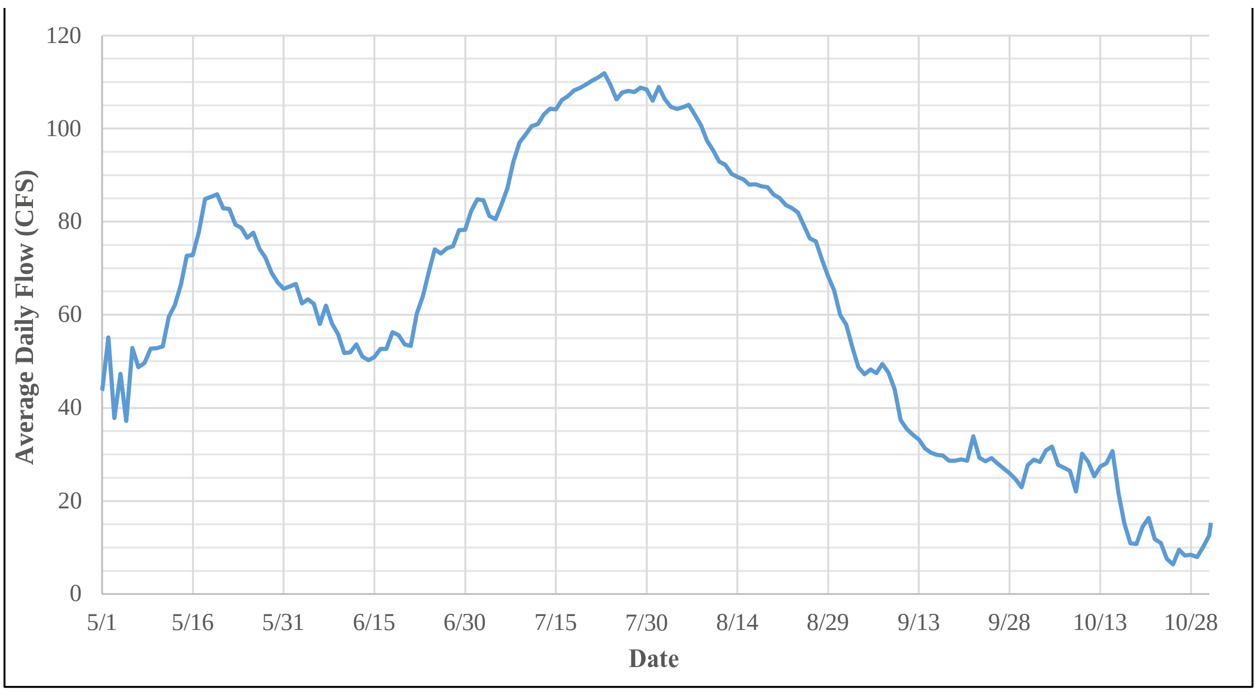

FIGURE 2-2 AVERAGE DAILY FLOW IN AUBURN RAVINE

3.0 SURVEYED ELEVATION DATA

R.E.Y. Engineers, Inc. performed an elevation survey of the site in October 2015. The surveyed area extended from 270 feet downstream of the structure to about 530 feet to the approximate upstream limit of the impoundment created by the structure without the flashboards, 150 feet of the canal, and the structure.

The permanent concrete crest of the diversion structure varies from elevation 197.20 feet at the north end to 197.44 feet at the south end. The Auburn Ravine channel bottom upstream of the structure is relatively flat. At the upstream extent of the survey, the channel bottom is at elevation 196.55 feet, while 40 feet upstream of the structure the channel bottom is at elevation 196.06 feet. This represents an average slope of approximately 0.094 percent. At the base of the structure, the channel elevation is 189.37 feet. The channel bottom elevation 75 feet downstream from the structure base is 191.10 feet. This is most likely a result of scour from flow spilling over the structure. The channel bottom elevation at the downstream boundary of the surveyed area (270 feet downstream of the structure) was found to be at elevation 190.26 feet.

The upstream invert elevation of the culvert at the entrance to the Hemphill canal is 197.51 feet. The downstream invert elevation of the culvert is 197.59 feet. The surveyed invert elevations were taken from the pipe bottoms and indicated the culvert has a slightly adverse slope (the downstream end is higher than the upstream end). Photo 3-1 is the Hemphill diversion structure, looking downstream. Photo 3-2 shows the canal intake structure. Photo 3-3 depicts the canal culvert outlet, which shows that sediment has accumulated in the bottom of the culvert, reducing its effective diameter. It is important to note that the crest of the diversion structure is lower than the invert of the culvert, meaning that there will be no flow into the canal until the water surface elevation in the stream exceeds the diversion structure crest, hence the need for the flashboards. The channel elevation downstream of the Parshall flume gage, which is located 200 feet from the canal intake, is 196.12 feet. This results in an average canal slope of 0.7 percent from the canal intake to the downstream side of the Parshall flume. Photo 3-4 shows the Parshall flume and the flat appearance of the landscape and canal. Photo 3-5 shows the extent of the survey data collected.

PHOTO 3-1 HEMPHILL DIVERSION STRUCTURE LOOKING DOWNSTREAM

PHOTO 3-2 CANAL INTAKE STRUCTURE

PHOTO 3-3 CANAL CULVERT OUTLET WITH ACCUMULATED SEDIMENT

PHOTO 3-4 HEMPHILL CANAL AT THE PARSHALL FLUME

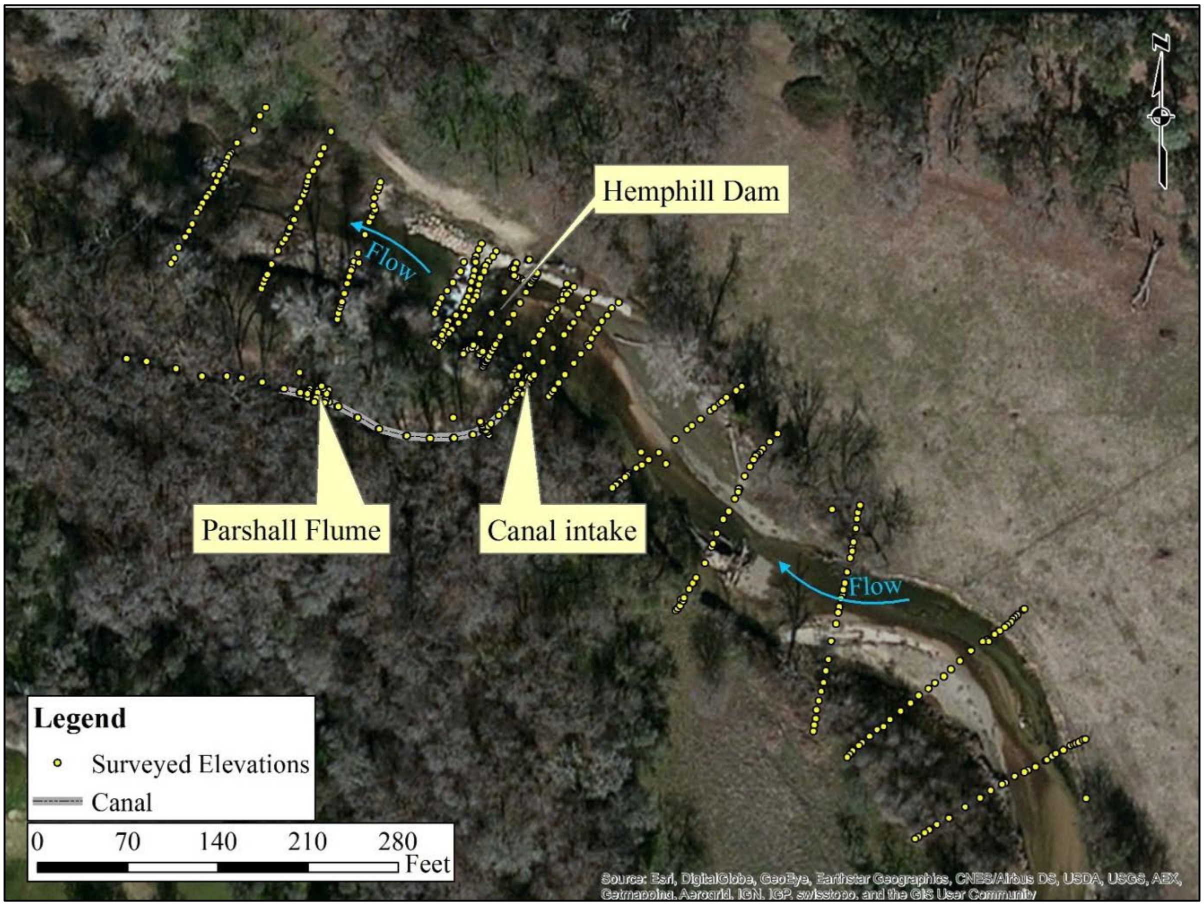

PHOTO 3-5 HEMPHILL DIVERSION STRUCTURE SURVEY DATA

3.1 HEMPHILL DIVERSION STRUCTURE SURVEY DATA

The survey, and as shown in the aerial view of the Hemphill diversion structure in Photo 3-5, shows evidence of significant sediment accumulation in the impoundment. Photo 3-6 shows the unstable river overbanks located upstream of the project. Removal of the diversion structure will increase the instability of the streambed and banks and result in increased sediment load until a stable channel is established. Photo 3-7 shows an area of extensive accumulated sediment on river-right upstream of the structure, and Photo 3-8 shows the sediment accumulated downstream of the structure. Structure removal would impact the current sediment deposition and result in changes to the channel bottom elevations in the river, which in turn would impact design conditions.

Removal of the Hemphill diversion structure could potentially require further study to determine the actual volume of accumulated sediment, sources of future sediment, chemical, and physical make-up of the sediment, and its potential for downstream migration. Further, studies regarding methods of stabilizing the overbank areas and/or controlling future sediment accumulation in Auburn Ravine would also likely be required. Sources of future sediment may not be confined to the impoundment area.

Sediment studies would include obtaining sediment samples (corings) in the river to determine the depth of sediment in order to estimate the total volume of sediment and to conduct chemical analyses to determine if the sediment contains any hazardous materials, such as heavy metals (e.g., lead, mercury, cadmium, etc.) or industrial organic compounds (e.g., polycyclic aromatic hydrocarbons or polychlorinated biphenyls). The presence of hazardous materials in the sediment would complicate removal of the diversion structure since it is unlikely that regulatory agencies would permit large quantities of sediment to be transported downstream. This may necessitate removal of accumulated sediment that would need to be disposed of off-site; greatly increasing construction costs. For this concept level analysis, the District assumed that the sediment is clean and will migrate naturally downstream.

PHOTO 3-6 VIEW LOOKING UPSTREAM FROM DIVERSION STRUCTURE SHOWING UNSTABLE OVERBANK

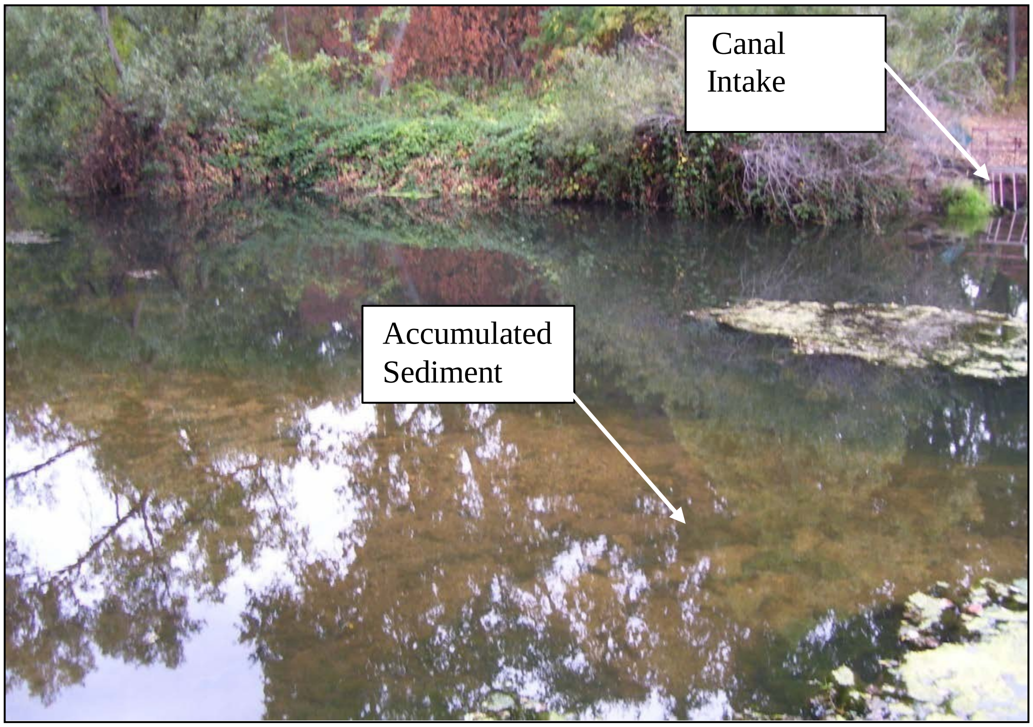

PHOTO 3-7 VIEW OF ACCUMULATED SEDIMENT ON RIVER RIGHT UPSTREAM OF DIVERSION STRUCTURE

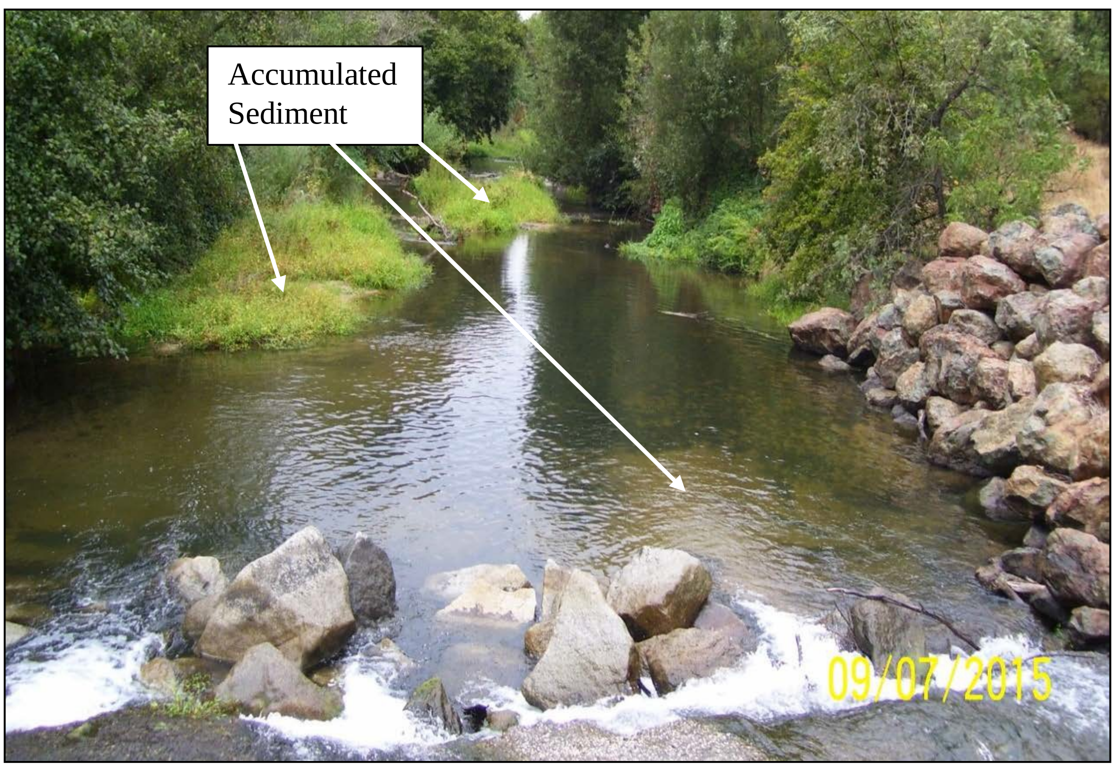

PHOTO 3-8 DOWNSTREAM VIEW OF DIVERSION STRUCTURE WITH SCOUR POOL, SEDIMENT ACCUMULATION, AND DEBRIS

4.0 HYDRAULIC ANALYSIS

The historical flow data and the survey data were used to develop a hydraulic model of the Hemphill diversion structure. The U.S. Army Corps of Engineers' (USACE) HEC-RAS v5.0 hydraulic modeling software was used to develop a 2-dimensional (2D) model of the Hemphill diversion structure and stream. The model geometry included the area surveyed.

4.1 MODEL GEOMETRY

The model domain extends approximately 530 feet upstream and 270 feet downstream of the structure. The surveyed elevation data points were converted to a raster elevation grid file using AutoCAD Civil3D. The elevation file from Civil3D was then converted into a usable file format for HEC-RAS v5.0 using Esri's ArcGIS ArcMap software. Three-foot by 3- foot grids were created to define the upstream river channel, the channel downstream from the structure, and the canal. Figure 4-1 shows the 2D model grids. The checkered, green patterns are the individual cells in the 2D grid (note: the squares that appear to be much larger than 3 foot by 3 foot are a result of zooming out to view the entire geometry).

The stream grid was assigned a Manning's roughness coefficient (n) equal to 0.04, which is appropriate for an earthen stream channel, winding and sluggish, with cobble bottom and clean banks. The canal grid was assigned a Manning's n equal to 0.03, which is typical for an earthen channel, winding and sluggish, with some weeds (Chow, 1959).

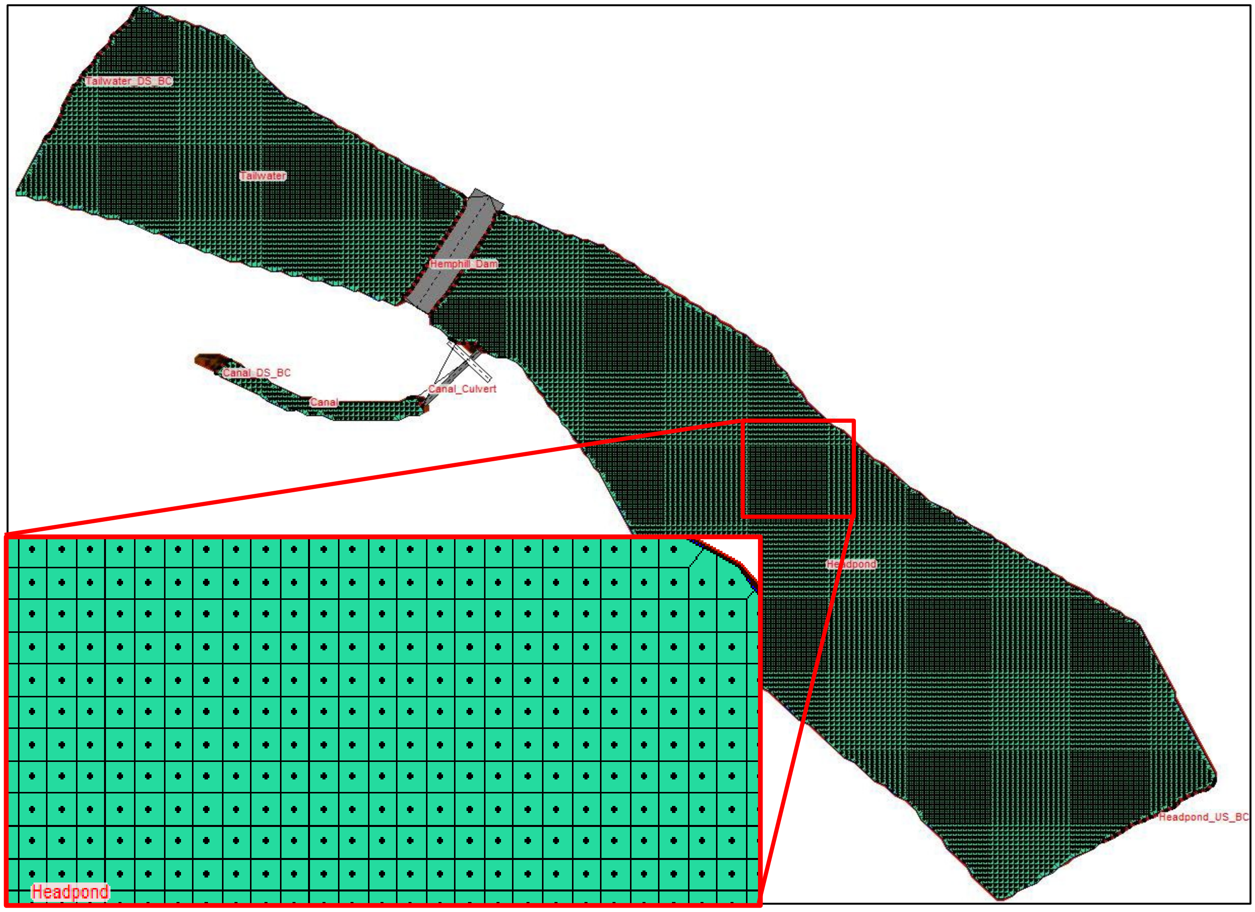

FIGURE 4-1 HEC-RAS 2D MODEL GEOMETRY

The upstream and downstream Auburn Ravine grids were connected by a 1D representation of the structure. The upstream grid was connected to the canal grid using a 1D representation of the culvert. Both the structure and the culvert geometry were based on the surveyed elevation data.

Two different model geometries were developed for simulations: an existing condition geometry that included the structure without flashboards and an alternative geometry where the diversion structure was removed. The purpose of the alternative geometry is to estimate the resulting water depths in the stream if the structure is removed. The geometry was not adjusted to account for movement and/or removal of sediment accumulated upstream of the diversion structure that is likely to occur after removal. Data regarding the volume and characteristics of the sediment to approximate future river conditions would be used in the sediment transport model to estimate future river conditions. Based upon Photo 3-7 and Photo 3-8 that show significant accumulations of sediment near the diversion structure, it is likely that the channel bottom would incise multiple feet at the diversion structure. The channel would also incise in a decreasing amount in the upstream direction until the point where modifications to the diversion structure would not affect the water surface elevation. The channel bottom elevation downstream of the diversion structure would vary as sediment is transported downstream until such time as the system reaches stabilization.

4.2 MODEL BOUNDARY CONDITIONS

The model boundary condition on the Auburn Ravine upstream grid was specified as an inflow hydrograph for each simulation. The inflow hydrograph varied depending on the specific simulation being examined. The boundary conditions for the Auburn Ravine downstream boundary and the Hemphill canal boundary were set to be normal depth based on the channel slope at the respective downstream boundaries.

4.3 MODEL FLOWS

Flows evaluated in the model ranged from 10 CFS to 100 CFS, which covers the Auburn Ravine's typical flow range during the irrigation season. Although higher flow rates occur in the stream, the lower flows are most critical for meeting the needed design flow into the canal when stream flows are low. The flow entering the canal during simulations depended upon the head pond elevation, which controls the flow into the canal.

4.4 MODEL CALIBRATION

For model calibration, a simulation using the existing structure with flashboards was performed. The results (i.e., what parts of the geometry were wetted) were compared to flow patterns noted on aerial photography of the site. Model results at a flow of 40 CFS showed good agreement with flow patterns seen on the aerial photography. The flow stays completely within the banks of the river. As flow approaches the structure, the velocity tracers indicate flow would be concentrated on the right side of the spillway because it is roughly four inches lower than the left side. Figure 4-2 provides a side-by-side comparison of the aerial photography with the model results.

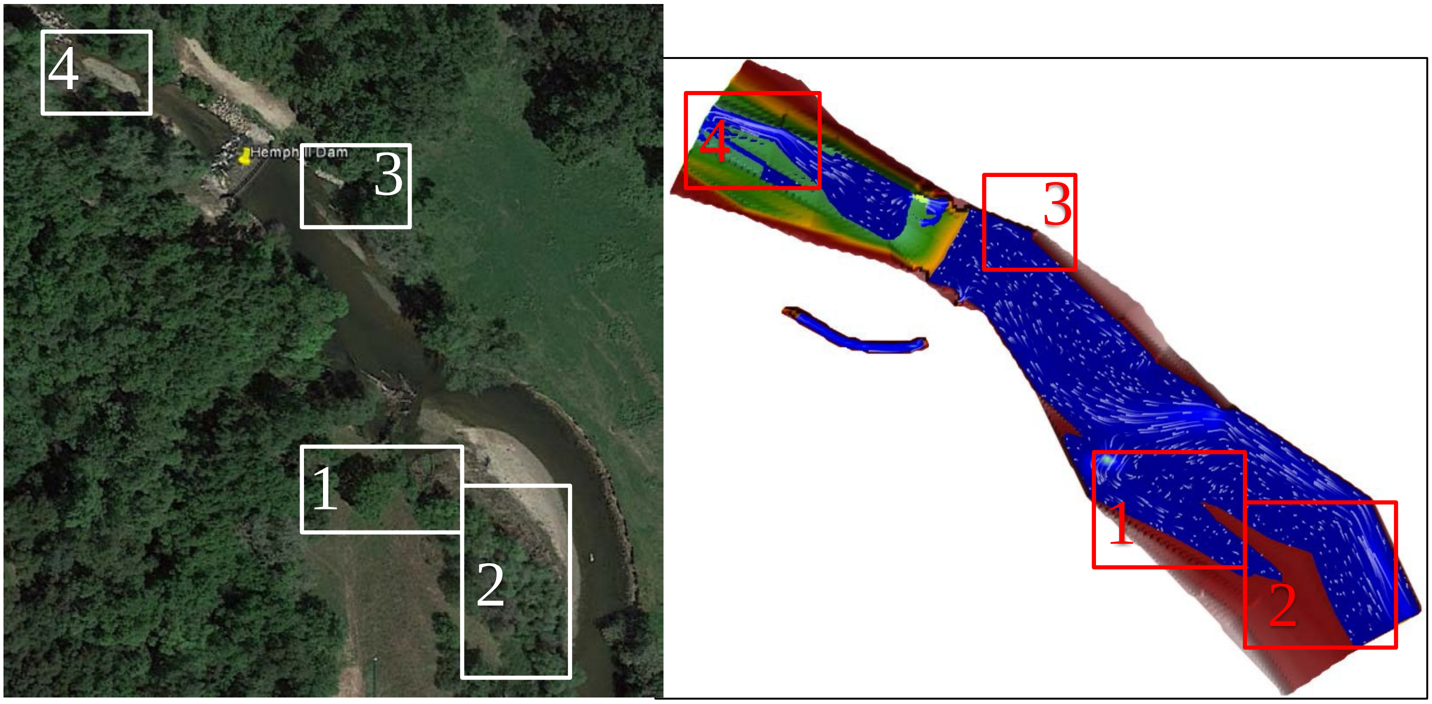

FIGURE 4-2 MODEL CALIBRATION AERIAL IMAGERY COMPARISON WITH HEC-RAS RESULTS

Figure 4-2 shows the correlation between the model and actual conditions. In both the aerial photography and the model there is a backwater area just downstream of the channel bend (see "1" and "2") and a small backwater area upstream of the structure on the right bank (see "3"). Additionally, the model captures the flow around the "island" of deposited material downstream of the structure (see "4"). This comparison shows that the model reasonably simulates the dynamics of the stream system and is considered to be calibrated with the available data.

5.0 ALTERNATIVES ANALYSIS

The following reviews both mechanical and non-mechanical options for providing flows to the canal with the diversion structure partially or fully removed. The hydraulic analysis has indicated that because of the minimal channel and the canal slopes, non-mechanical options to provide the required flow to the Hemphill canal with the diversion structure removed are very limited, if possible at all. With the diversion structure removed and assuming current conditions, a flow of at least 100 CFS in Auburn Ravine is required before water will flow into the Hemphill canal through the existing intake. Partial or full removal of the diversion structure will lower of the river bed in Auburn Ravine due to the natural movement or mechanical removal of accumulated sediment; thus requiring an even higher flow in Auburn Ravine before flows will enter the Hemphill Canal.

A detailed analysis of the physical and chemical characteristics of the accumulated sediment, assessment of upstream sediment sources, bank stabilization, potential removal means and methods, and future impacts of sediment transport would likely be required to finalize the design process for any modifications to the Hemphill Diversion structure. Permit costs will also need to be determined; however, they are somewhat dependent on what option, if any, is advanced.

5.1 STATUS QUO OPTION

This option is always considered as part of an alternatives analysis as it represents the baseline existing condition. Maintaining the current structure needs no further detail in this analysis. The status quo option is not considered to be a viable option because the existing diversion structure represents a partial fish barrier. Resource agencies have indicated this is an environmental concern that needs to be addressed by the District. The District is also investigating methods of passing fish at the Hemphill diversion structure.

5.2 OPTION 1 - REMOVE FLASHBOARDS ONLY (NON-MECHANICAL)

For this option, we evaluated not installing the flashboards during the irrigation season but leaving the diversion structure in place. The hydraulic modeling indicated that under these conditions no flow would enter the canal from Auburn Ravine until flows in Auburn Ravine exceeded 70 CFS. Based on the historical flow data, this flow is achieved or exceeded approximately 40 percent of the time during the irrigation season (Figure 2-1), which is unacceptable. This does not consider the impact of sediment transport, which would result in lowering of the river bottom and reducing the percent of the time that flows through the canal would be available. The requirement for fish passage would also need to be evaluated for this option. For these reasons, this option is not considered viable.

5.3 Option 2 - Diversion Structure Removed - Flow-Diverting Wing Wall (Non-Mechanical)

The second non-mechanical option evaluated removing the diversion structure and constructing a diversion wing wall that would direct a portion of the Auburn Ravine flow toward the canal intake (Figure 5-1). As flows are funneled toward and approach the canal intake, flow velocity would decrease, increasing water depth as a result of conservation of energy. If the velocity of the diverted flow is sufficiently high, the resulting increase in water surface elevation could potentially be sufficient to provide the head level needed to drive flow into the canal. The velocity energy in the water can be related to the depth of water using the relationship $H = \frac{V^2}{2g}$ where:

H = depth of water, feet
V = flow velocity, feet per second (fps)
g = acceleration due to gravity (constant), 32.2 feet per second squared

The HEC-RAS 2D model indicates that when the river flow is 40 CFS, the velocity upstream of the structure – with the structure removed – would be around 1.5 fps. Using the relationship above, the model indicates that only about 1 inch of additional depth could be recovered from the velocity energy, which is insufficient to provide the flow required in the canal. Geotechnical and environment studies would be needed to determine whether or not such a structure could be permitted and constructed. Additionally, as the velocity of the flow captured by the diversion wing wall decreased near the entrance to the canal, any suspended sediment would drop out of the water column and be deposited behind the wall. Sediment and debris accumulation would pose a permanent maintenance challenge since it would impede flow into the canal and would need to be periodically removed to prevent the system from failing to operate. Fish and debris screens would also be a requirement. Because of the minimal gain from the recovery of the flow velocity, the potential lack of suitable foundation conditions for the structure and the sediment/debris issue, this option, was not considered to be viable.

FIGURE 5-1 OPTION 2 CONCEPTUAL LAYOUT

5.4 OPTION 3 - DIVERSION STRUCTURE REMOVED - NEW PARTIALLY BURIED PERFORATED PIPE (NON-MECHANICAL)

This option considered installing a 3- to 4-foot diameter pipe at the approximate upstream limits of the existing impoundment, running perpendicular across Auburn Ravine, through the river-left overbank and then extending to a discharge point downstream of the Parshall flume in the canal. The total pipe length, assuming this positioning, would be approximately 700 feet (Figure 5-2). The exact location for the upstream portion of the pipe is subject to further study.

The upstream section, located in the river channel, would be a wedge wire section surrounded with rock and wrapped in a geotextile material to prevent the pipe from clogging with sediment. A minimal slope of approximately 0.05 percent would be needed to provide the future flow demand to the canal. The pipe would have a valve at the outlet to control the flow into the Hemphill canal.

FIGURE 5-2 OPTION 3 CONCEPTUAL LAYOUT

Due to the large diameter of the pipe and the elevations in the channel and canal, the upstream section in Auburn Ravine would only be partially buried. Once into the overbank area, the pipe would be buried, daylighting near the downstream exit in the canal. The wedge wire pipe sections would allow the system to take advantage of water flowing over, into, and around the pipe. The pipe section in the river would be covered in riprap to protect it from being damaged by passing debris and to minimize sediment from accumulating on the screen.

A diameter of three to four feet is needed to minimize the flow velocity, reducing head losses because of the minimal hydraulic gradient between water in the stream and the canal. Additionally, the large diameter would increase the surface area through which water could enter the pipe. The hydraulic analysis of the pipe, assuming the pipe has a 4-foot diameter and a slope equal to 0.05 percent, shows that at a flow of 15 CFS, the normal flow depth in the pipe would be 1.9 feet, and the velocity would be approximately 2.6 feet per second. The resulting head losses would be approximately 0.5 foot over the length of the pipe. This flow approximates the future average flow demand forecast for the canal. As shown on Figure 5-2, the water surface elevation at the upstream end of the pipe would be approximately 197.9 feet. Assuming this elevation, there would be roughly 1.6 feet of available head, which is sufficient to pass the required flows through the pipe into the canal. The self-cleaning velocity to avoid accumulation of sand and other sediment is two to three feet per second (Sturm, 2001), which is met by normal depth velocity under design flow conditions. Sedimentation may still occur if the pipe flow drops below 6 CFS, which is the minimum flow demand in the canal.

A valve at the downstream end of the pipe would be installed to regulate the flow to meet demand and prevent diversion of water to the canal outside of the irrigation season. While a portion of pipe would be above the river bed at the upstream end, a rock ramp would be installed both on the upstream and downstream side to provide protection for the pipe and also allow fish passage. Because the Hemphill canal demand represents only a small portion of the total Auburn Ravine flow during the irrigation season, a substantial flow would be passing over the pipe at all times. If Placer County Water Agency (PWCA) and/or NID were to remove water from Auburn Ravine upstream of the Hemphill location, an impact to the flows would be expected and would require further evaluation to define the significance of the impact. Evaluating that impact is not within the scope of this study.

This option has potential; however, the exact length and location of the pipeline will need to be field verified. The upstream end of the pipeline could be extended further upstream to a location that provides more elevation difference, thus reducing the required pipe diameter. Further study would be needed to determine the best location for such an intake system and would have to account for the District's authorized point of diversion location. If the diversion structure is removed, sediment analysis would be critical for identifying the extent the channel riverbed would decrease at the proposed intake location.

The major concern with this option would be the impact of sediment transport in and around the pipe. The potential for the pipe to accumulate sediment over time is also a concern. Accumulated sediment can cause several problems, including reduced flow area, and clogging of the screens that allow flow into the pipe. This system would require regular maintenance and the frequency of system flushing would be dependent on the sediment load in the stream. One possible option would be to include a port at the upstream end of the pipe to which a hose could be attached to flush sediment from the pipe.

5.5 OPTION 4 - DIVERSION STRUCTURE REMOVED – RIVER BANK FILTRATION SUMP PUMP SYSTEM (MECHANICAL)

This option includes installing a large-diameter concrete or steel sump pit immediately adjacent to the canal side of the ravine. The sump would need to be a minimum of 12 to 15 feet in diameter and at least 10 to 15 feet deep. The buried portion of the sump would be perforated to allow water to enter and fill the sump. The sump pit would also be designed to receive surface water from Auburn Ravine when flows are high. Water would be pumped from the sump into the canal using a two-pump system. The pumps would be sized so that one pump would be able to provide the majority of the flow demand to the canal.

Two vertical sump pumps would allow for consistent operation and adaptation to varying flow demands during the irrigation season. The pumps would operate singularly or together to provide a flow ranging from 6 to 15 CFS (2,700 to 6,700 gallons per minute), pumping water approximately 50 feet to openly discharge into the canal. The pumps could also be fitted with variable frequency drives to provide flow stability, better efficiency, and energy savings. The dual pumps would also offer system redundancy in case of a unit outage. The overall head of pumped water is estimated to be around 10 to 12 feet. Each of the proposed pump units would have a 20.5-inch-diameter suction intake with an approximate 32-inch height and can be fully submerged. Unit dimensions were sized to minimize head losses to about 1.50 feet.

The proposed motors for each pump (20 horsepower) would be designed to operate normally around 16.6 horsepower at an efficiency of about 80.7 percent and flow of 7.2 CFS. An external power source of at least 230 volts and preferably 480 volts would be supplied to run both pumps simultaneously. Maximum power demand is estimated to be 30 kilowatts (kW). A small building or platform would be installed over the sump to house the pumps and associated controls and to provide security.

Soil borings would be needed to determine the hydraulic conductivity of the soil in the sump construction area. Piezometers positioned in the bank would monitor the water table level to ensure that the sump pit is constructed to a depth sufficient to ensure that bank storage would fill the sump.

A design concern regarding this option would be porosity of the surrounding soil to allow sufficient inflow while minimizing sediment inflow into the sump pit. This could be addressed by constructing a filter berm of specifically graded material to allow flow while minimizing sediment inflow. Sedimentation of the sump would likely occur and require frequent cleaning. The frequency of sump cleaning would be dependent upon the sediment load in the river and the frequency of high flow events that would increase sediment entering the sump. The other major issues for this option include the cost of supplying the power required to run the pumps and the annual operating costs for the system. The pumps would also require regular maintenance and occasional replacement parts for the life of the system.

5.6 OPTION 5 - DIVERSION STRUCTURE REMOVED - RANNEY WELL PUMP SYSTEM (MECHANICAL)

Another mechanical alternative is to construct a Ranney well system (Layne, 2015) and install two axial flow (vertical turbine) pumps in the well. The Ranney well system consists of a large diameter vertical caisson installed in the bank of the stream with horizontal collection arms drilled in a fan array under the river to provide inflow from the saturated soil to the caisson. Ranney well systems generally provide high yields of flow and have lower operating and maintenance costs compared to traditional well systems. In addition, they are less intrusive to the environment, eliminate problems such as sediment buildup, and are less sensitive to fluctuating water table depths, unlike more conventional pump and well systems (Layne, 2015). Based on data from the vendor, installation time for this type of system varies from 6 to 9 months. The pumps for this proposed alternative are identical to those described in Section 5.5, Option 4.

The two major concerns regarding this option would be the composition (porosity) of the material underlying the river bed and providing the power to operate the pumps. The pumps would require regular maintenance and occasional replacement parts for the life of the system. Similar to Option 4, the hydraulic conductivity of the soil in the bank as well as under the streambed would need to be determined by collecting soil borings and piezometer measurements of bank storage water levels.

5.7 OPTION 6 - LINCOLN CANAL / AUBURN RAVINE 1 CONNECTION

Another option for providing water to the Hemphill canal in the event of the removal of the diversion structure includes providing flow via a pipeline from nearby canals, such as the

Lincoln canal / Auburn Ravine 1 (AR1). While simple on paper, an extensive study would be required to ensure that an adequate flow is available in the supply canal. Current data for the Lincoln canal indicate that it does not currently have sufficient capacity. Modifications to expand carrying capacity in the Lincoln canal would be needed in order to consider this a possible option. Construction of the pipeline and the required permitting could greatly increase costs, and these would also be major factors in assessing viability.

5.8 Option 7 - Abandonment of Hemphill Canal

With the removal of the Hemphill diversion structure and abandonment of the Hemphill canal, the canal water could be replaced with treated water or recycled water from the Lincoln wastewater plant. The details and viability of providing treated or recycled water are outside the scope of this analysis; additional information would be required to conduct a feasibility of this option.

5.9 Alternative Options Matrix

Table 5-1 summarizes the options presented. There are six categories related to the viability of each option along with a qualitative score based on the currently available information. A score of “1” indicates that the option has low environmental impact, has low relative cost, would be relatively simple to construct, and would be an effective method; a score of “2” indicates a moderate to high impact; and a score of “3” indicates a very high impact, costly and/or difficult to construct and minimally effective. The lower the total scores, the better option. Based on the scores, the river bank filtration option has the lowest score of the options considered and is, therefore, the most viable option. All options would require additional detailed studies to confirm their viability and cost. Additional studies include, but are not limited to, detailed engineering feasibility, environmental assessments, and geotechnical analysis.

TABLE 5-1 OPTIONS MATRIX

OPTION	CONSTRUCTABILITY	ENVIRONMENTAL ISSUES	O&M CONCERNS	COST	ADDITIONAL STUDIES	EFFECTIVENESS	TOTAL
OPTION 1 - FLASHBOARD REMOVAL	1	3	1	1	1	3	**10**
OPTION 2 - DIVERSION STRUCTURE REMOVAL WITH WING WALL	2	2	2	2	3	3	**14**
OPTION 3 - PARTIALLY-BURIED PIPE SYSTEM	2	2	2	2	2	2	**12**
OPTION 4 - RIVER BANK FILTRATION WITH PUMPS SYSTEM	1	1	1	2	2	1	**8**
OPTION 5 - RANNEY WELL SYSTEM	2	1	1	3	2	1	**10**
OPTION 6 - LINCOLN CANAL CONNECTION	2	2	2	2	3	2	**13**
OPTION 6 - AR1 CONNECTION	2	2	2	2	3	2	**13**
OPTION 7 - TREATED WATER /RECYCLED WATER	3	3	1	3	3	1	**14**

6.0 OPINION OF COST

6.1 OPTION 1 - STATUS QUO (FLASHBOARD REMOVAL)

An opinion of cost was not prepared for Option 1 because the hydraulic analysis found this option unfeasible; however, at a minimum, costs would include a continuation of debris removal and required structure maintenance. The District has information on the annual costs for these tasks.

6.2 OPTION 2 - DIVERSION STRUCTURE REMOVED - FLOW-DIVERTING WING WALL

An opinion of cost was not prepared for Option 2 because the hydraulic analysis found this option unfeasible.

6.3 OPTION 3 - DIVERSION STRUCTURE REMOVED - NEW WEDGE WIRE PIPE

Table 6-1 shows a breakdown of the costs to construct Option 3. The cost estimate does not include costs for the removal of the Hemphill diversion structure. Since Options 3, 4, 5, 6, and 7 require identical structural alterations, the removal cost for the Hemphill Diversion is the same for all and was not included in this comparative analysis. Unit costs (including material, equipment and labor) for excavation, backfilling, and the pipe were obtained from RSMeans using 2016 pricing, open shop labor, and California price adjustments (RSMeans, 2015).

The cost estimate assumes that wedge wire pipe would only be used for a portion of the pipeline (approximately 100 linear feet). High-density polyethylene (HDPE) pipe was assumed for the remainder of the pipe length.

TABLE 6-1 OPTION 3 COST OPINION

SYSTEM COMPONENTS	QUANTITY AND UNIT	UNIT COST	ESTIMATED COSTS
48” Diameter Wedge Wire Pipe	100 linear feet	$109.5/linear foot	$ 11,000
48” Diameter Corrugated HDPE Pipe	600 linear feet	$59.16/linear foot	$ 35,500
Excavation Pipe Trench	2,000 cubic yards	$4.91/cubic yard	$ 10,000
Clearing/Grubbing	1 acre	$6,700/acre	$ 7,000
Backfilling Pipe	1,675 cubic yards	$3.71/cubic yard	$ 6,000
Rock for Lining Perforated Pipe Section	567 cubic yards	$52.25/cubic yard	$ 30,000
Flow Control Valve	1 unit	$59,000/unit	$ 59,000
Mobilization & Demobilization (15% of Subtotal)	Lump Sum	$23,800	$ 23,800
Engineering & Contingency (10% & 20% of Subtotal)	Lump Sum	$ 47,600	$ 47,600
TOTAL ESTIMATED COST			**$ 229,900**

6.4 OPTION 4 - DIVERSION STRUCTURE REMOVED – RIVER BANK FILTRATION SUMP PUMP SYSTEM (MECHANICAL)

Table 6-2 shows the estimated cost of constructing Option 4. As noted above, costs do not include structure removal. Unit costs (including material, equipment and labor) for excavation, backfilling, and the sump pit was obtained from RSMeans using 2016 pricing, open shop labor, and California price adjustments (RSMeans, 2015). The opinion of cost does not include construction of a permeable berm.

The estimated pricing for each pump and the motor unit would be approximately $22,000 to $25,000, and the estimated lead time would be 6 to 8 weeks. The cost estimate assumes a 10-foot deep caisson and a fabricated steel head. The electrical hookup is a lump sum estimate assuming $14 per foot for 400 feet of 3-phase power line extension, three utility poles, transformers supplied by the local utility, installation charge of $60 per hour for 80 hours of labor by an electrician, meters and service, and $4,000 for a power panel and meter socket. The annual electricity cost is estimated assuming $0.152 per kilowatt hour for 6 months of continuous operation. The annual electricity cost would be incurred for the life of the system and would change based on electric rates. Based upon 2015 rates the cost of power for the pump system would be approximately $20,000 for a year.

TABLE 6-2 OPTION 4 COST OPINION

SYSTEM COMPONENTS	QUANTITY AND UNIT	UNIT COST	ESTIMATED COSTS
Pump Unit Cost	2 units	$22,000 – $25,000/unit	$ 44,000 - 50,000
Sump Pit Excavation	300 cubic yards	$18.19/cubic yard	$ 5,500
Sump Installation (Labor and Equipment)	5 days	$3,400/day	$ 17,000
Sheet Pile Sump (Material)	400 square feet	$27.30/square foot	$ 11,000
Estimate of Electrical Hookup	Lump Sum	$20,000	$ 20,000
Pre-Engineered Steel Building	750 square feet	$12.60/square foot	$ 16,200
Mobilization & Demobilization (15% of Subtotal)	Lump Sum	$18,000	$ 18,000
Engineering & Contingency (10% & 20% of Subtotal)	Lump Sum	$35,900	$35,900
TOTAL ESTIMATED COST			**$173,600**

6.5 OPTION 5 - DIVERSION STRUCTURE REMOVED - RANNEY WELL SYSTEM

Layne, a California contractor, specializing in the construction of Ranney well systems, provided a concept level price for the construction of the Ranney well system. The conceptual price was a comprehensive estimate for the system but did not include the cost of the pumps or the electrical hookup. The electrical hookup is a lump sum estimate assuming $14 per foot for 400 feet of 3-phase power line extension, three utility poles, transformers supplied by the local utility, installation charge of $60 per hour for 80 hours for labor by an electrician, meters and service, and $4,000 for a power panel and meter socket. The annual electricity cost is estimated assuming $0.152 per kilowatt hour for 6 months of continuous operation. The annual electricity would be incurred for the life of the system and would change based on the electrical rates. Based upon 2015 rates, the cost to power both pumps would be approximately $20,000 per year.

Table 6-3 shows a breakdown of the costs to construct Option 5. We have assumed that the Engineering and Contingency costs are included in the lump sum pricing provided by Layne.

TABLE 6-3 OPTION 5 COST OPINION

SYSTEM COMPONENTS	QUANTITY AND UNIT	UNIT COST	ESTIMATED COSTS
Pump Unit Cost	2 units	$22,000 – $25,000/unit	$44,000 - 50,000
Layne-Ranney Well System Complete Installation (Start to Finish)	Lump Sum	$1,500,000 – 2,000,000	$1,500,000 - 2,000,000
Estimate of Electrical Hookup	Lump Sum	$ 20,000	$20,000
TOTAL ESTIMATED COST			**$2,070,000**

6.6 OPTION 6 - LINCOLN CANAL OR AUBURN RAVINE 1 CONNECTION

An opinion of cost was not prepared for Option 6 because it is not deemed to be a viable option at this time because supply for the Hemphill canal is not addressed in the City of Lincoln's Water Master Plan. Additionally, studies would need to be performed to determine whether or not there is adequate flow in the supply canal to make the option possible.

6.7 OPTION 7 - ABANDONMENT OF THE HEMPHILL CANAL

An opinion of cost for this option would include removal of the diversion structure and costs for providing flow for either treated water and/or recycled water. Because supply for the Hemphill canal from these sources has not been addressed in the City of Lincoln's Water Master Plan, no specific routing can be determined and therefore, we cannot develop a cost estimate for providing treated and/or recycled water. Further, permitting costs and their impact on the construction process are unknown. For these reasons, no opinion of cost was developed.

7.1 SUMMARY AND CONCLUSIONS

The Hemphill diversion structure is a low head structure whose specific purpose is to increase water depth sufficient to provide gravity flow to the Hemphill canal. It has been proposed that the District remove the structure to provide fish passage to areas upstream of the diversion. This study was done to review potential options for providing flow to the canal, given existing conditions, with and without the structure removed. Given environmental concerns and associated regulatory pressure to provide increased fish passage, keeping the diversion structure was not considered a viable option.

To evaluate options for removal, a hydraulic analysis was performed for the Auburn Ravine in and around the Hemphill diversion structure to determine the water levels that would result in the removal of the structure. This data provided the information needed to assess the performance of various options. The results of the analysis determined that with the removal of the structure, water levels for all but high flow conditions in Auburn Ravine at the existing canal intake structure would be well below the invert to the canal. Further, the resulting increased velocity of flows in Auburn Ravine would initiate sediment transport resulting in further lowering of water levels at the canal intake.

Options reviewed included:

1. constructing a diversion wall upstream of the existing canal intake to direct a portion of the river flows to the canal intake;
2. installing a wedge wire pipe in the stream at a location upstream from the existing diversion to allow water to flow into the pipe which would then be directed into the canal;
3. installing a sump pit and pump system adjacent to the existing canal intake;
4. installing a Ranney well system adjacent to the existing canal intake;
5. diverting water from the nearby Lincoln canal;
6. diverting water from the nearby Auburn Ravine 1; and
7. abandoning the use of the Hemphill canal and replacing the canal water with water from the Lincoln wastewater plant.

Other options outside the scope of this analysis include potential water supply from the Lincoln Wastewater treatment plant and providing treated water from the Regional Water Supply Project to replace the Hemphill canal supply. These two options are not considered viable as they are not evaluated in the City of Lincoln's Water Master Plan the RWSP EIR and would involve a considerable cost to implement, including full permitting and environmental review and analysis.

A matrix was developed to rate the various options. Matrix categories included constructability, permitting, environmental factors, operation and maintenance requirements, cost, and anticipated effectiveness. The results of that analysis indicated that Option 4, River Bank Filtration sump pit option is, at this stage in the analysis, the top rated option.

8.0 REFERENCES

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