4. GROUNDWATER

4.1. Introduction

4.1.1. The feasibility of obtaining water for the hatchery from a right bank well field was investigated. Two general locations were considered: north of or at the hatchery site, and upstream of the seepage blanket. The area around or immediately north of the hatchery site does not appear suitable to provide sufficient water. Further north there is better potential but land ownership in this area is not known. State Park land upstream of the seepage blanket is probably the most suitable area for a well field. However, a test well would have to be installed and pumped to determine if a well field is feasible. It is estimated that at most an 11/2-mile by 2000 foot area in the park would be suitable for well installation.

4.2. Hatchery Site or North of Hatchery Site

4.2.1. A water supply well field does not appear to be a viable possibility at the hatchery site or immediately north of the proposed site. Several test wells have been drilled in this area and had very poor water production. A test well was drilled slightly northeast of the hatchery site in 1986 (by the "Colville Tribal Fish Hatchery"). This well encountered shallow bedrock at 106 feet and only produced 5 gpm during a test. The Corps has drilled at least two test wells in this general area (1988 and 1990) and both had poor test production. Shallow wells in this area, that tap the superficial, perched, aquifer are producing on the order of 50 gpm. It is unlikely that a well field could be installed in the proposed hatchery area that could produce anywhere close to the hatchery water requirements. The only likely location for wells would be well north of the hatchery site along Jack Wells Road. This area is probably located in a former river channel, and is likely to have good water potential, with enough room for a sizable well field. However, property status and water rights in this area are unknown. The extent of contamination from agricultural activities here is also unknown. It is known that the city of Bridgeport drilled a well at the mouth of this valley that produced 500 gpm in a 1950 test.

4.3. Upstream of Seepage Blanket (State Park or Golf Course)

4.3.1. The best potential for groundwater would appear to be the permeable gravel aquifer located beneath impermeable till. This material ranges from 30 to 100 feet thick, and extends for at least 2000 feet shoreward from the reservoir. The upstream extent of this aquifer is not fully defined, but is believed to extend to the upstream park boundary. The presence of these gravels is the reason for the installation of the impervious seepage blanket and the relief tunnel. Because of dam safety concerns, any wells will have to be installed upstream of the seepage blanket. This means that the wells would have to be installed on State Park property approximately 2 miles upstream of the dam or 2 _ miles upstream of the hatchery Head Box.

4.3.2. It is certain that such a well field could supply some portion of the required water, However, several steps would need to be taken to determine the quantity of water potentially available from this source.

4.3.3. First, the exact downriver boundary of the potential well field needs to be defined. The initial discussions concerned the main 2,500-foot impervious blanket that extends upstream to the vicinity of piezometer 170. However, there is also an additional seepage blanket that extends 1,500 feet upstream from the primary blanket. This 3-foot thick blanket of silty fine sand was installed in 1957 in an attempt to reduce seepage at the time of its construction, and extends upstream to the vicinity of piezometer 296. If wells would have to be installed upstream of this secondary blanket because of dam safety concerns, this would significantly reduce the available size of the well field. The upstream limits of the well field would be dictated by property boundaries, and the possible slope stability constraints on the upstream end (increased probability of slumping at upstream end of park). Based on park boundaries and downstream aquifer mapping, there is at most an1 1/2 mile by 2000 foot area where wells could be installed within the park.

4.3.4. Second, more information is needed about aquifer parameters to determine minimum well spacing and anticipated yields. Although it’s possible to estimate well performance and aquifer characteristics based on historical data, this is only available for an area downstream of the seepage blanket, where recharge to the aquifer has purposely been modified with the blanket to reduce right bank seepage. The only way to determine the necessary information for the unmodified aquifer upstream of the seepage blanket would be to drill a test well and conduct a pumping test. If the test well was located reasonably close to the network of piezometers, they might be used as observation wells for the test. Data from such a test would provide the information needed to determine well spacing and ultimately a better estimate for how much water could be produced from a right bank well field.

4.3.5. Without some actual aquifer testing, it’s difficult to confirm that a well field located upstream of the seepage blanket could produce all or most of the ground water needed for the hatchery. It is interesting to note, though, that a well tested in the state park in 1967 was capable of producing 1400 gpm.

5. HYDRAULIC ANALYSIS

5.1. Introduction

5.1.1. This analysis looked at various options for supplying water to the proposed Colville Tribes Fish Hatchery near Chief Joseph Dam. The hatchery needs 45 cfs from the reservoir behind the dam and another 35 cfs from ground water for both summer/fall and spring Chinook. The different sources of water provide variations in temperature needed for hatchery operations. Two possible ways to tap these sources include wells and/or the drainage tunnel (known as the relief tunnel) on the right side of the dam and the irrigation diversion structure that was built into the right side of the dam. Comments made regarding these options are from a hydraulic standpoint.

5.2. Review of Existing Information

5.2.1. Various documents, maps and drawings were reviewed to aid in this analysis. Information gathered includes:

5.2.1.1. <u>Relief Tunnel Data</u>: The relief tunnel is an 8-foot tall, 5-foot wide, 1000-foot long drainage tunnel that extends into the right embankment of Chief Joseph Dam. The purpose of the structure is to prevent excessive pore-water pressure from developing in the right embankment material. The main tunnel empties into a 10.5-ft long by 4.5-ft wide by 4.5-ft deep sump. From the sump, the water is discharged through a 4-ft diameter conduit into the stilling basin. It appears the relief tunnel currently generates, on average, 22 to 25 cfs of flow. Evidently this flow was up around 90 cfs back in the 1960's, but has declined in the past. Plate 1, at the end of this section, includes a plot of hourly relief tunnel flow readings between 2000 up through 2003. This plot indicates that there are times when the flow does drop down to the 15 to 17 cfs range.

5.2.1.2. <u>Irrigation Diversion Structure</u>: This structure was built into the dam to supply irrigation water at some point in the future. It consists of two 4-ft wide by 5-ft height passages through the right side of the dam with inverts at elevation 920 feet. This structure has provisions for a stop log closure and a trashrack. The intake side is sealed with concrete while the outlet side is sealed with bricks. It is unclear at this point exactly how this structure was intended to be used.

5.3. Development of Supply Options

5.3.1. Pumping from Relief Tunnel Sump: From a hydraulic standpoint this appears to be feasible. Some assumptions made for this analysis include:

- Delivery point to hatchery is elevation 870 feet
- Full sump elevation is 782 feet
- Flow from tunnel is always a constant 25 cfs
- Steel pipe material is used
- A centrifugal-type pump is used

- The design river tailwater elevation is 795 feet

5.3.1.1. With these assumptions, it appears that from a hydraulic standpoint, a system could be constructed. Assuming a design discharge of 25 cfs, and making some assumptions as to system hydraulic head losses, the system would include a properly sized centrifugal-type pump and a 22-inch diameter welded steel discharge pipe. The pump would be designed to run continuously, therefore making a holding tank at the delivery point unnecessary. At the delivery point there would be a gate valve. To avoid cavitation problems, the pump would need to be located at an elevation that is not too far above the sump water surface elevation. Ideally there would be enough room in the sump access chamber. For one of the pumps looked at in this analysis, the estimated maximum vertical distance above the sump water surface was about 24 feet or around elevation 806-feet. This pump had a 30-inch diameter suction tube. Based on standard sump design methods, the existing sump appears to be too small to support pumping at this rate. Calculations indicate that, for the design flow, the sump should measure about 22-feet long, 7.5-feet wide, and 9.5-feet deep. In order to prevent vortexing and other undesirable hydraulic conditions, the sump should always maintain a water depth of at least 6-feet. Ideally, the incoming water velocity would be 1 ft/sec or less. Sump designs were made using guidelines from U.S. Army Corps of Engineers publication EM-1110-2-3105.

5.3.1.2. Figure 1 shows a basic schematic of the system and Figure 2 shows a cross-sectional view of the sump and needed modifications to the sump and the chamber ceiling.

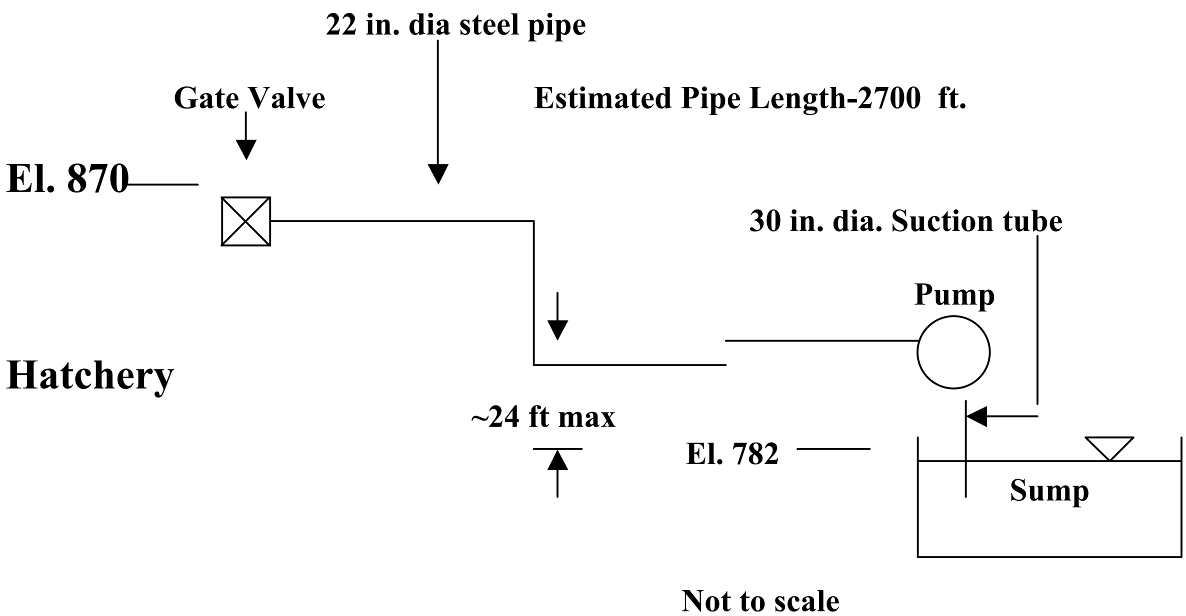

Figure 1. Conceptual Relief Tunnel Pump Schematic

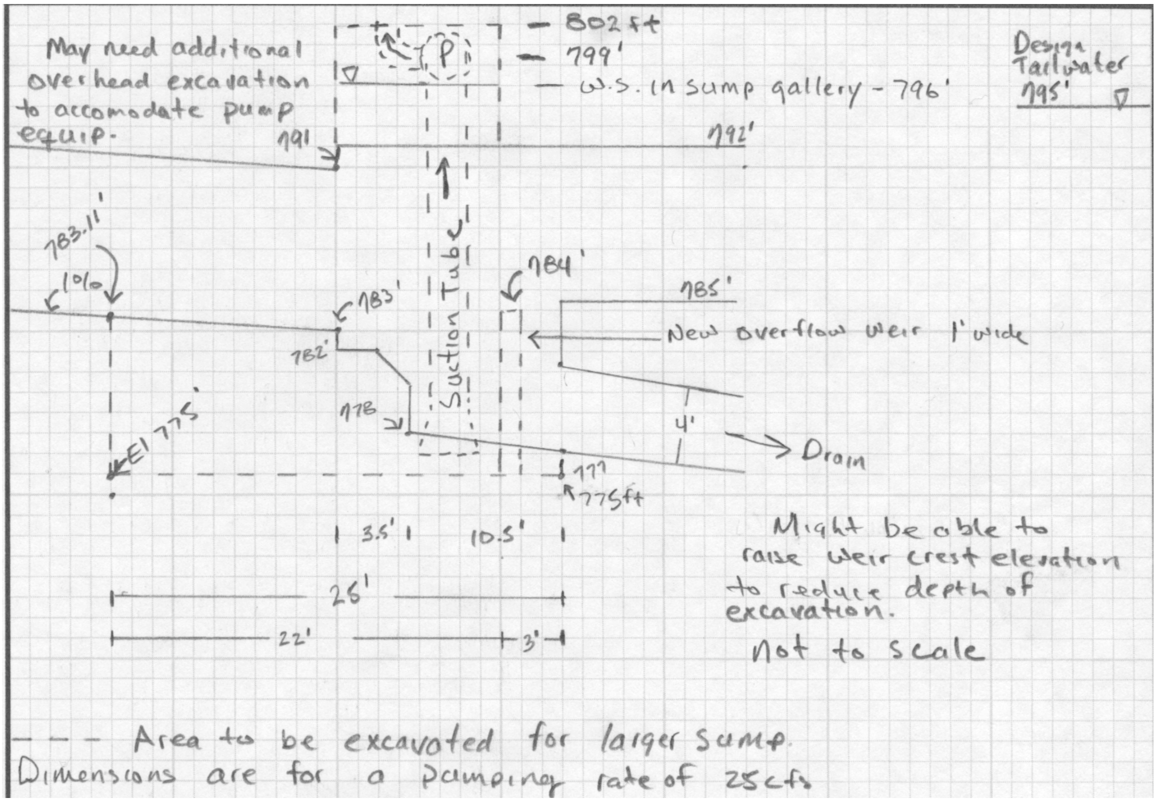

Figure 2. Conceptual Sump Cross Section for Pumping 25 cfs

5.3.1.3. In addition to the increased sump dimensions, Figure 2 shows other sump chamber modifications that include raising part of the sump chamber ceiling to keep pumping equipment in the dry up to the design tailwater elevation and construction of a weir to always insure the sump is always at a minimum elevation of 784 during low tailwater conditions. Under this configuration, as tailwater elevations exceed 795, the area in the chamber where the pump and motor are located would start to become inundated with water and equipment damage could be expected. When comparing Figures 1and 2 it might be noticed that the sump water surfaces differ by two feet. This is because the first step was to estimate system head losses and the sump water surface was initially chosen to be at 782. As the sump modifications were developed the elevation was changed to 784. This difference is not a factor at this point in the conceptual designs.

5.3.1.4. The velocity of the inflow to the sump is also of concern. Ideally, this value should be something on the order of 1 ft/sec or less. Plate 1 indicates that the velocity is closer to 4 ft/sec most of the time. Other structural measures may be needed to make the system function correctly. This may be addressed by raising the elevation of the weir to create a backwater that extends farther into the relief tunnel. Other disciplines would need to weigh in on problems (for example right abutment drainage might be impacted) that could arise (possibly hinder the drainage of the right abutment) from permanently having the relief tunnel flooded to some elevation.

5.3.1.5. The main hydraulic issue other than the size of the sump involves pumping at the maximum relief tunnel flow. Pumping at the maximum relief tunnel flow would be problematic if slight variations in relief tunnel flow occur. During periods of low tailwater, the sump could easily be pumped dry, causing pump and supply problems, and during periods of high tailwater elevations (above about elevation 784-785-when the sump chamber would start to flood above the sump) river water could be sucked into the sump due to a lower water surface in the chamber then that in the river. This would cause the pump to supply a mixture of relief tunnel water and river water. It is felt that it would be preferable to pump at some rate that is less than the relief tunnel flow, probably about 20 cfs based on what is known at this point.

5.3.1.6. The chance of river water entering the sump should also be looked at, even if the relief tunnel is pumped at a rate lower than the flow rate of the relief tunnel. There might be instances where the tailwater elevation could temporarily exceed that of the sump chamber, causing the flow to switch from chamber-to-tailwater to tailwater-to-chamber, causing river water infiltration. The proposed flow deflectors on the spillway may also have some affect on this issue. This could be alleviated with the installation of a one-way valve in the 4-foot drainpipe. There would be some issues to consider with installation of this valve as well. There would be additional installation and O&M costs as well as issues regarding valve failure/plugging and associated sump chamber/relief tunnel flooding.

5.3.1.7. The current pump capacity of the sump in its current configuration is about 10 to 14 cfs. Due to the presence of the 4-foot drain at the end of the sump, the installation of a weir structure similar to that in Figure 2 would be needed to insure that the sump is always full of water. This weir would take up space and further cut down pumping capacity. The other issues discussed above, such as the tailwater elevation, still would apply.

5.3.1.8. In the event that an unconventional sump design was needed, it is possible that a physical model study would be required to verify correct operation.

5.3.2. Constructing a Well Field to Supply Groundwater

5.3.2.1. This option would require wells be drilled in the area on the right side of the dam. These wells would be tapping groundwater that would most likely have similar characteristics as the relief tunnel water. At this point in the conceptual design, the wells would pump into a nearby storage tank that would in turn supply the hatchery. Conveyance between the storage tank and hatchery would be accomplished via a 24-inch welded steel pipe. The pipe size was arrived at based on estimates of length, number of bends, valves, etc. Based on the available mapping, the hatchery delivery point was assumed to be elevation 870, and the minimum tank water surface was assumed to be 1035.

5.3.2.2. Based on the estimate of well yields (150 gpm), about 100 wells would be needed to supply the total ground water requirement of 35cfs for both summer/fall and spring Chinook. In addition, a storage tank 30-feet in diameter and 20-feet tall would be required. This amount of storage would require the well pumps to be running about 90% of the time. If this value needs to be lower, the capacity of the tank would need to be larger. Figure 3 shows a schematic of the system.

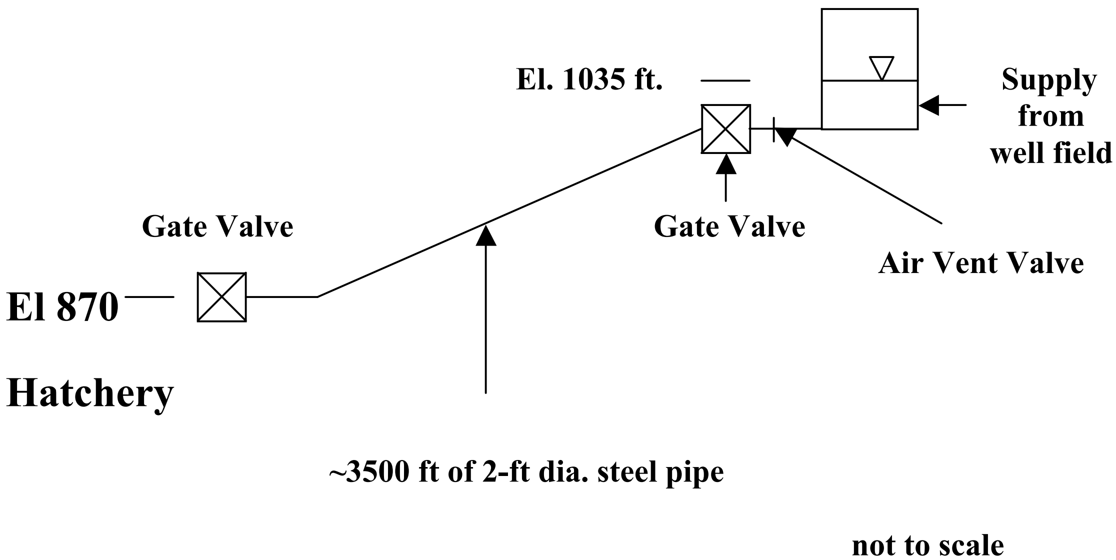

Figure 3. Well Field Supply System Schematic

5.3.2.3. More data on the volume of water that could be expected from wells is needed to further develop this option. Depending on the cost of various components of the two options for harvesting groundwater (wells and relief tunnel), it might make sense to use a combination of the two options to provide the needed supply.

5.3.3. Irrigation Diversion Structure

5.3.3.1. Chief Joseph Dam has a feature built into the right side of the structure that evidently was to be used as a diversion for agricultural water at some point. Project drawings do indicate provisions (but no designs) for a stop log type closure gate on the upstream side as well as a trash rack. The diversion has two inlets and outlets measuring four feet wide by five feet tall. The sill elevation is at elevation 920 feet. From the drawings it appears that the inlet entrances are radiused to minimize entrance head losses. Currently the inlets and outlets are plugged with concrete.

5.3.3.2. The hatchery would require a 45 cfs supply of forebay water. Again, assuming a delivery point elevation of 870 feet, and using the Chief Joseph minimum operating pool of 930 feet (60 feet of head), calculations indicate that a 30-inch diameter (assumed steel for calculations) would supply the required 45 cfs under minimum pool conditions. Using this scheme, one of the inlets and outlets would be opened, and the required pipe placed through the dam and the space between the outside of the pipe and the inlet would be sealed. The pipe system would continue on to the hatchery delivery point and terminate with a gate valve. A trash rack would need to be fabricated as well as a stop log system. For safety purposes a gate valve and an air vent valve (to prevent low pressures that could occur under some conditions) would also be installed near the intake end of the pipe. The system would be operated with the upstream gate valve in the fully opened position with flow to the hatchery controlled by the downstream gate valve. Figure 4 shows a schematic diagram of the system using the irrigation diversion.

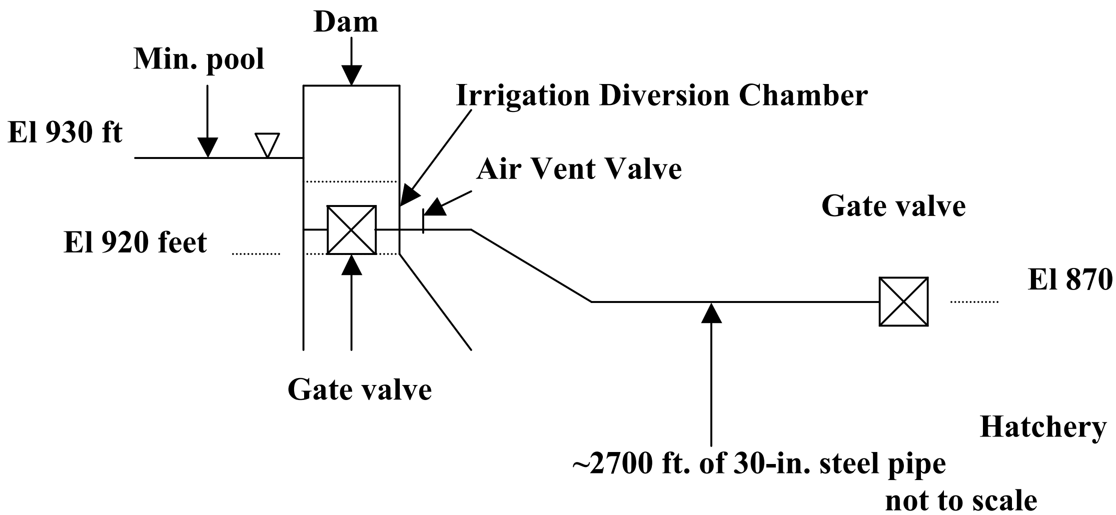

Figure 4. Irrigation Diversion Schematic

5.4. Recommendations

5.4.1. Groundwater Supply

5.4.1.1. In light of the lack of well field information, at this point the best option for supplying groundwater appears to be pumping 20-22 cfs from the relief tunnel (the exact value to be determined after analysis of relief tunnel flows) and supplying the balance from wells. This lower pumping rate would help alleviate the problems with the sump being pumped dry (and resulting supply problem) and with intrusion of river water. Figure 5 shows a conceptual sump chamber cross-section and Figure 6 shows the plan view of the sump. Due to the lower volume being pumped, theses two figures show a smaller sump than that shown in Figure 2. The sump would measure 18-feet long by 6-feet wide by 7 feet deep, compared to 22-feet long by 7.5-feet wide by 9-feet deep. As with the concept shown in Figure 2, it would require three additional feet of length for the weir structure and the weir overflow well. The volume of material to be removed for the sump shown in Figures 5 and 6 would be about 417 cubic feet compared to 1287 cubic feet for the version shown in Figure 2. As pumping rates are reduced so is the required sump size. The diameter of the supply pipeline would also be slightly reduced due to the lower pumping rate. Preliminary calculations indicate that a 20-inch diameter steel pipe would be required as opposed to a 22-inch diameter pipe for a 25 cfs pumping rate.

5.4.1.2. The elevation of the weir crest is important to the amount of excavation required. There are several factors (discussed below) that would determine this value for a final design. If the crest could be located at a higher elevation, the amount of vertical excavation would be less as long as the required amount of pump intake submergence is met. While raising the weir crest would reduce the amount of excavation, it also would back water up into the relief tunnel. It would need to be determined what affect this would have on relief tunnel performance.

5.4.1.3. Finally, depending on cost, difficulty, logistics, etc. of excavating the sump, it is possible that a sump could be designed that incorporates smaller dimensions than discussed above but provides satisfactory performance for the same pumping rate. To arrive at an "unconventional" design would require reviewing details of other sumps that have been constructed, working with a pump manufacturer(s) and possibly a physical model study. Depending on the other, non-hydraulic issues identified by other disciplines, this added effort might make a lot of sense.

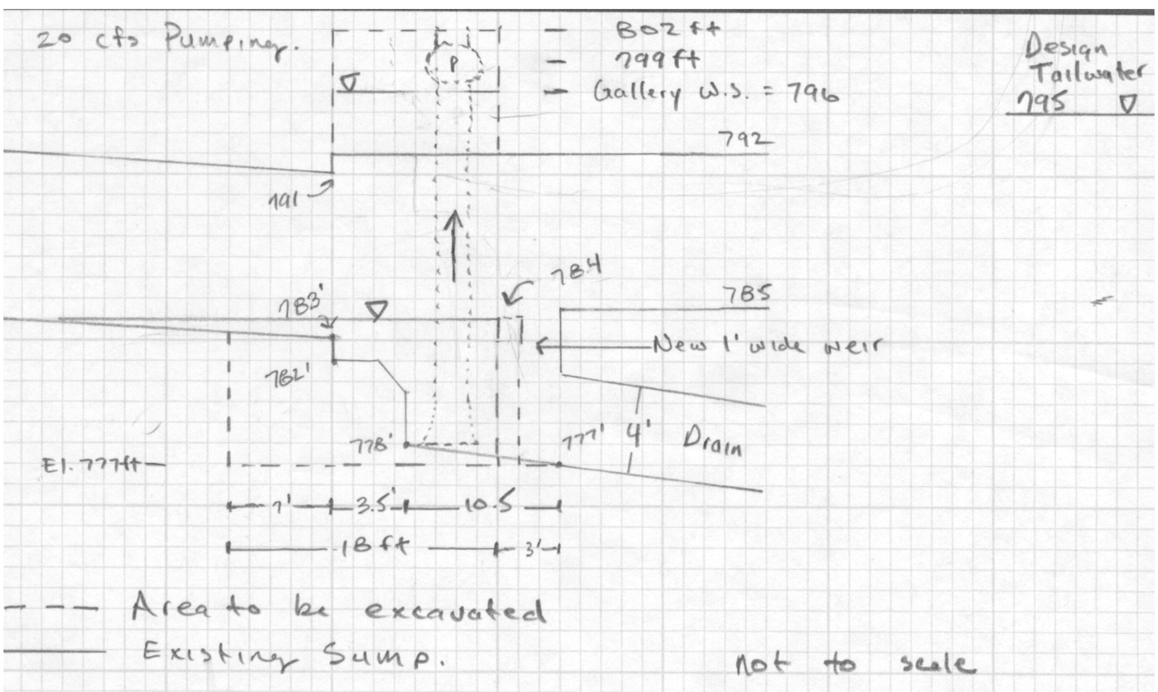

Figure 5. Proposed Conceptual Sump Chamber Cross Section

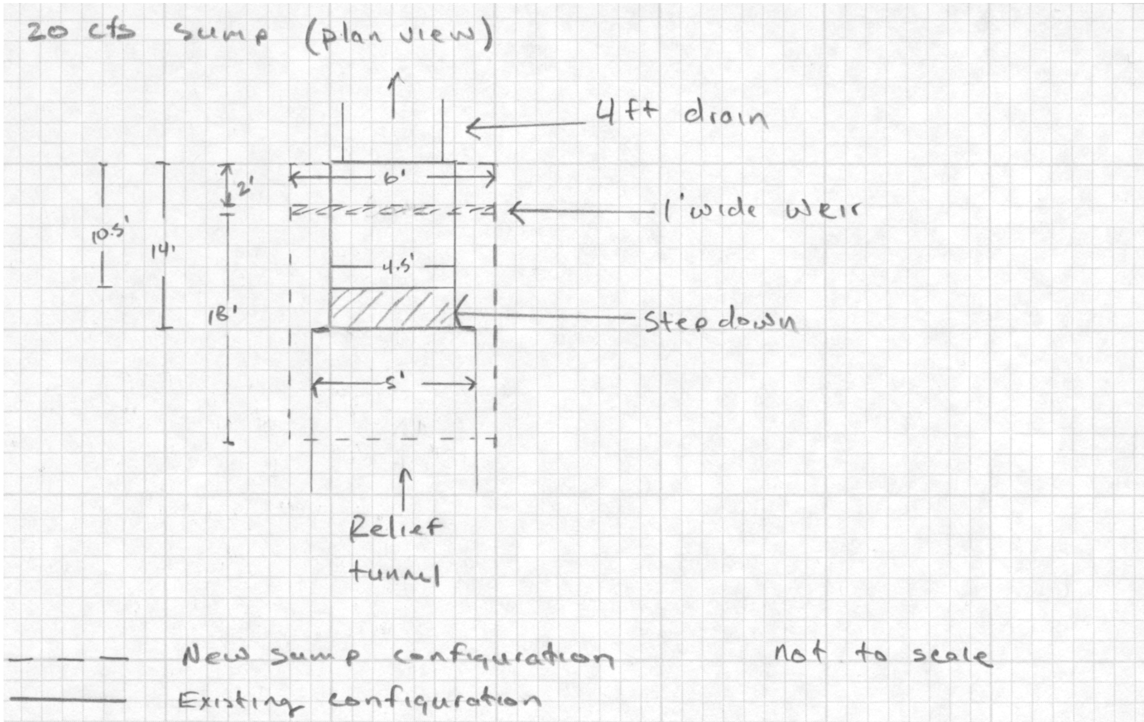

Figure 6. Proposed Conceptual Sump Chamber Plan

5.4.1.4. Some considerations for further development of this concept:

5.4.1.4.1. Pumping Equipment**:** To this point this option was developed using a conventional, centrifugal-type pump with a motor that would need to operate in the dry. This created the need to elevate the sump chamber ceiling to provide this dry area during high tailwater conditions. An investigation into motors designed to run in either the wet or the dry might yield one that would work in this case and reduce or eliminate the distance the ceiling needs to be raised. A quick call to a pump manufacturer found a submersible motor that did not produce the required horsepower for this application. Possibly, a suitable unit could be found or the sump could be configured to use two smaller pumping units. More investigation into the difficulty of raising the ceiling, pumping equipment availability and sump modification needs to be done.

5.4.1.4.2. Relief Tunnel Flow: An investigation into flow variation of the relief tunnel needs to be undertaken to arrive at a pumping rate that will be hydraulically stable.

5.4.1.4.3. Stability of Tailwater: The amount of variation in the tailwater elevation due to surging, wave action, flow deflector installation and spillway influences for a given flow needs to be evaluated. The results will influence sump weir crest elevation and influence the decision to install a one-way flow valve in the 4-foot drain.

5.4.1.4.4. One-way Flow Valve Installation: In addition to the tailwater issue, factors that need to be evaluated to determine one-way flow valve suitability are O&M, valve failure and debris blockage. Blockage or failure could flood the sump chamber, relief tunnel, and access gallery. This could in turn cause damage to pumping equipment and possibly abutment stability problems if the relief tunnel could not effectively reduce pore water pressures due to flooding. During periods of high tailwater it would be practically impossible to access the valve for repairs. From a hydraulic standpoint, it appears that this valve would not be necessary. The flow from the relief tunnel should almost always (if not always) maintain an equilibrium sump/tailwater head differential (sump water surface being higher than tailwater) such that flow would be from the sump to the tailwater, resulting in the sump almost always containing relief tunnel water only. Surging and other factors that cause the tailwater to rapidly rise could possibly allow a small amount of river water to enter the sump chamber for a brief time until the equilibrium head differential is restored. After a more detailed hydraulic analysis, if there is still concern about tailwater intrusion, temperature and conductivity data loggers could be placed in the sump for a period of time (probably when spill is likely to occur) to detect the presence of river water.

5.4.1.4.5. Flow Regime of Relief Tunnel: The velocity of the flow entering the sump needs to be analyzed to insure suitable sump operation. Ideally this should be 1 ft/sec or less. Plate 1 seems to indicate that the velocity is higher than 1 ft/sec. Raising the elevation of the sump weir might also be a means to reduce incoming velocities. Other disciplines would need to provide input as to the ramifications of doing this. Whether the flow is sub critical or super critical also needs to be determined. If it is supercritical, a hydraulic jump might form at the interface of the tunnel flow and the backwater created by the new weir. If this happens, then a determination needs to be made as to what measures, if any, are required.

5.4.1.4.6. Design Tailwater Elevation: This value needs to be looked at to determine if it is realistic for design. If pumping equipment is used that would be damaged by flooding of the chamber, then it is important to determine a ceiling elevation that represents the amount of risk that is acceptable in this respect. The costs associated with raising the ceiling and/or damaged equipment would need to be looked at. A tailwater elevation exceedance analysis would be needed to make this determination.

5.4.1.4.7. Reliability: Thus far this concept has been developed using just one pump. Would some redundancy, such as a backup pump need to be incorporated?

5.4.2. Reservoir Water Supply

5.4.2.1. At this point, modifying the irrigation diversion chamber seems to be the best option for supplying reservoir water. Initially a pumping/siphon system was looked at, but due to the simplicity and reliability of a gravity system it was not developed. A pumping/siphon system would require an intake structure capable of working with pool elevations down to elevation 930 (this is the minimum operating pool) to always guarantee operation. For a true siphon, the maximum elevation differential between the reservoir surface and the maximum pipe elevation is about 28 feet. For full pool conditions this requirement would probably be met but for the lower elevations a pump assist system would likely be needed to make up any head differential in excess of 28 feet. Even though it would be a rare occurrence that the pool would get this low (depending on the routing of the pipeline, it is possible that such a system would be able to operate in siphon mode most of the time), an assumption was made that the system would always be functional. Even if a system were to operate only as a siphon, some type of pumping system would be required to prime it. Also, since the intake structure would be placed in the reservoir, upstream of the dam, a longer pipeline (and excavation) would be required for a pumping/siphon system than for the irrigation diversion. From a hydraulic standpoint the diversion structure appears to be a good option for supplying reservoir water at this point.

5.4.2.2. With all options, more hatchery design information would be helpful. Hydraulic systems within the hatchery (head losses, supply-duration requirements, delivery points, etc) may have an impact on the assumptions made to develop these options at this stage and could have a bearing on further development.

5.4.2.3. It should be noted that these hydraulic designs are very conceptual at this stage. Assumptions and estimates (pipe materials, number of bends, certain elevations, constructability, etc.) were made. As the designs evolve, aspects of them could change.

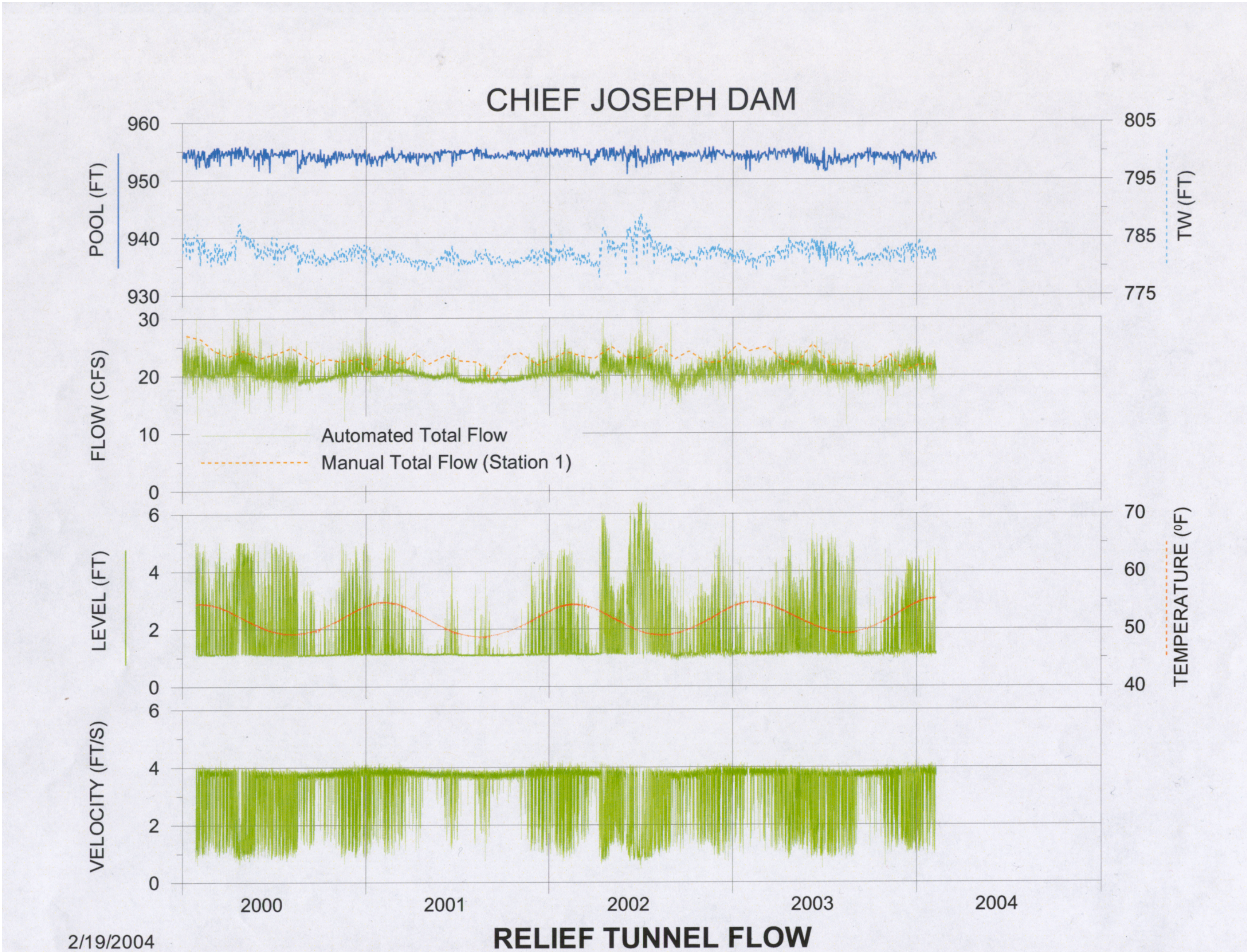

Plate 1. Relief Tunnel Flow Plot (2000-2003)