Power Extraction from Irrigation Laterals and Canals in the Columbia Basin Project

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Dedication

Mom and Dad

You told me to write….

1. Introduction

The Columbia Basin Project is located in central Washington, across Adams, Douglas, Franklin, Grant, Lincoln and Walla Walla Counties. Originally occupied by families who had acquired their land via the homestead laws of 1910 [1], the area was used for dry land framing and as rangeland. Due to the inability of the settlers to make a living, however, the majority of the Columbia Plateau was abandoned by June of 1937 [2]. During this time, a few years of higher than average rainfall [1] revealed the agricultural potential of the land when supplied with water and inspired the creation of what is now called the Columbia Basin Project.

In 1918, the idea to redirect the Columbia River through central Washington via the Grand Coulee Dam and Pumping Plant was formulated but the technology was not developed enough for such a design. By 1943, however, a draft of the Columbia Basin Project was completed [1] and quickly followed by the irrigation system's construction, providing thousands of jobs for returning soldiers of World War II. Although the project was designed to irrigate 1.1 million acres, only 671,000 acres of farmland are currently reached [3].

Starting at the Grand Coulee Dam, the Columbia Basin Project extends south 125 miles across the Columbia Plateau [1], terminating near the intersection of the Snake and Columbia Rivers, as seen in Figure 1.1. The general design of this irrigation system uses canals to distribute water over the majority of the Columbia Basin Project area, with laterals delivering the water to farms.

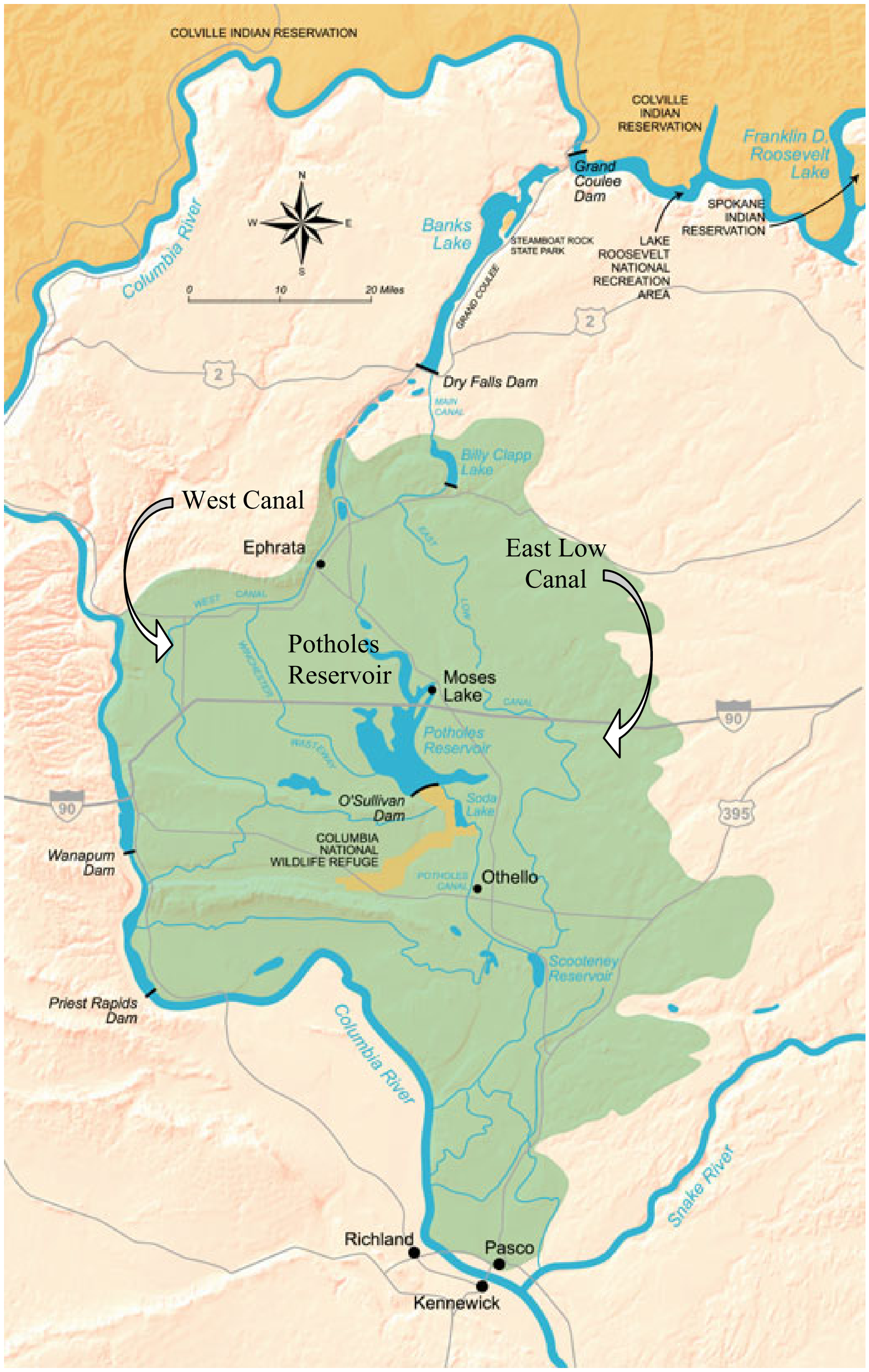

Figure 1.1: Columbia Basin Project [4]

The Main Canal, with a design capacity of 13,200 cubic feet per second (cfs) [3], transports water from Banks Lake to the Bifurcation Works, which then divides the canal into the West and East Low Canals. The West and East Low Canals are then responsible for delivering water across the northern portion of the Columbia Basin Project at flow rates of 5,100 cfs and 4,500 cfs [1], respectively. The Potholes Reservoir, the largest reservoir within the project, collects all unused or leftover water from the northern portion of the project and funnels the flow into the Potholes canal, which continues on to irrigate the southern part of the project. In addition to these 330 miles of canals there are also 1,993 miles of laterals and 3,163 miles of drains and waste ways [3].

Prior to creation of the irrigation system, the only crops produced in this area were cereal grains. With the current irrigation, however, the area is now a large supplier of alfalfa hay, ensilage crops, dry beans, fruit, grain, sugar beets, potatoes, sweet corn and seed, as well as many other specialty crops [1]. By providing this formerly profitless area with water, the annual crop value has now increased to over $550 million [3]. In addition to providing water for agriculture, the project also provides flood control, recreational and wildlife sanctuary as well as being authorized to generate over 6.48 million kilowatts of electrical power [1].

As the majority of the irrigation canals are free from fish and wildlife, Grant County PUD sought to explore the possibilities of hydropower generation within the canals and laterals. In particular, the use of hydrokinetic turbines in these canals seemed appropriate as this developing technology could take advantage of the relatively high flow velocities and absence of environmental conflicts. Since the priority of the irrigation project is to provide water for agriculture, it was agreed that canals would not be appropriate for this study. Although their high flow rates and velocities would be optimal for a kinetic based power generation system, the canals transport the majority of the water and alterations could affect the ability of the project to supply its customers with water.

Laterals were determined to be the best place to explore the technology and its effect on the water flow, as they are smaller in scale and have less impact on overall water delivery. By evaluating the effects and potential for power generation on these smaller channels, a better understanding of the technology and its larger issues could be observed. In cooperation with both Grant County PUD and the Columbia Basin Project, three sites were chosen for both hydrokinetic and conventional hydropower evaluation. The purpose of this feasibility study is to examine the structures of the chosen sites and determine the optimal design to incorporate hydropower based on cost, power generation, and downstream effects. Hydrokinetic and conventional hydropower are considered to allow the most cost effective situation to be determined, and preferred options are presented for each site.

2. Hydrokinetic Power

2.1 Introduction

Declining fish populations, sediment transport issues, and other environmental considerations [5] have caused an increase in dam removal over the past 10 years (see Figure 2.1). The effects on the environment and fish have led to the removal of several power generating dams, some of which originally produced over 3 MW [6]. This loss of a renewable energy source has challenged dam operators to find alternative forms of hydropower that do not have deleterious consequences.

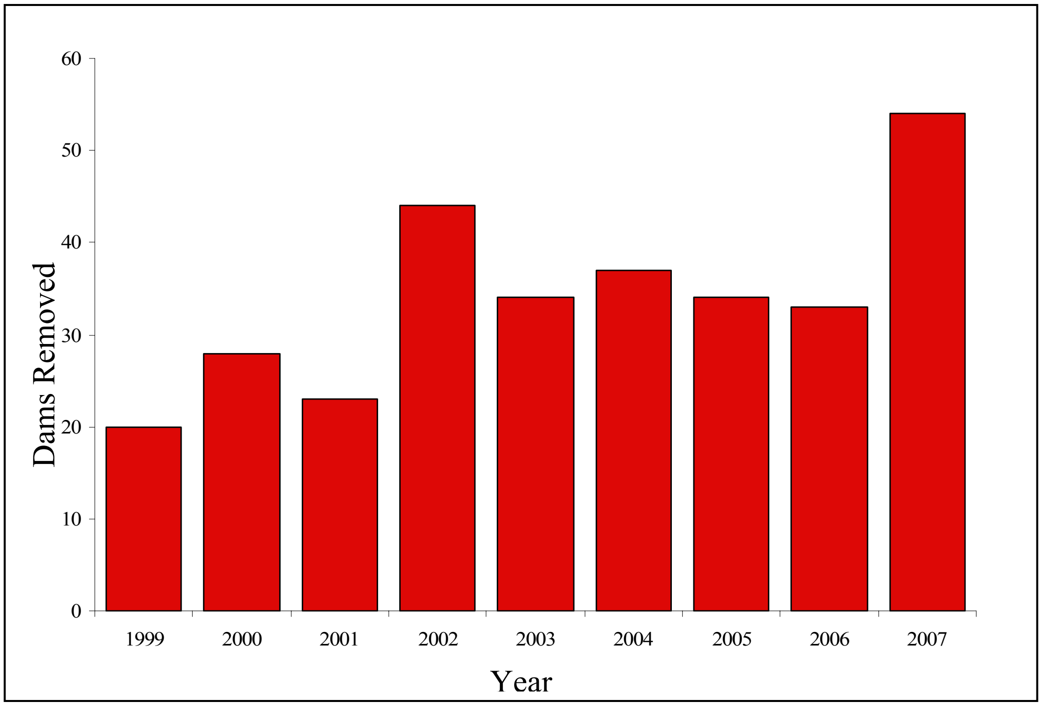

Figure 2.1: Dam Removals per Year since 1999 [7]

The newest form of hydropower technology is hydrokinetic power, which harnesses the kinetic energy of the flow. Unlike conventional hydropower, which disrupts the flow by an impoundment, hydrokinetic power generation is based on the kinetic energy of the flow, with the turbine placed in a river or channel. Hydrokinetic turbines, similar to wind turbines, generate power based on the velocity of the water and the size of the turbine. The heightened environmental sensitivity to hydropower in general has limited its development, however.

In response to interest in hydrokinetic power, the Department of Energy (DOE) prepared a list of potential environmental issues, the majority of which are related to the health and well being of aquatic life [8]. Other issues pertain to erosion and flow alteration, especially for natural channels such as rivers, streams and ocean floors.

2.2 Theory

Since hydrokinetic turbines are based on the kinetic energy of the flow, an estimate of the kinetic power resource can be found using Equation 2.1.

Equation 2.1

$$KE = \tfrac{1}{2}\, \rho\, A_c\, v^3\, 10^{-3}$$

In this equation ρ is the density of water (kg/m^3), A_c is the swept area of the turbine (m^2), v is the velocity of the flow (m/s) and KE is the kinetic power (W). Though the above equation describes the available kinetic power in a flow, the generated power is less due to the efficiency of the turbine and its effects on the channel. There are also energy losses due to the mixing of the free-stream flow with the wake following the turbine, resulting in more power extracted from the channel than generated. This theory is further explained in Chapter 5.

2.3 Turbines

The majority of hydrokinetic turbines are still in the prototyping phase, with a small number of companies offering commercial products. Figure 2.2 shows example sketches of the most common hydrokinetic turbine designs.

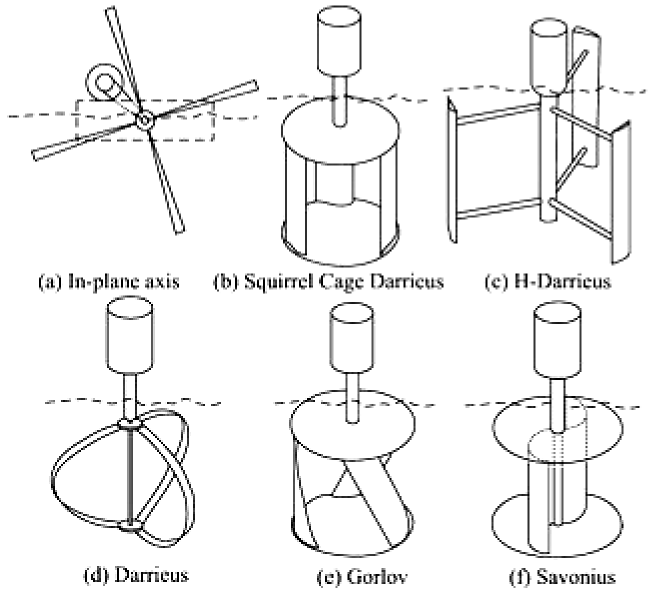

Figure 2.2: Examples of Hydrokinetic Turbine Designs [9]

The New Energy Corporation located in Calgary, Alberta, is one of the few companies with products available for purchase. The New Energy EnCurrent Power Generation product line consists of various sizes of the Darrieus-type turbine as seen in Figure 2.3.

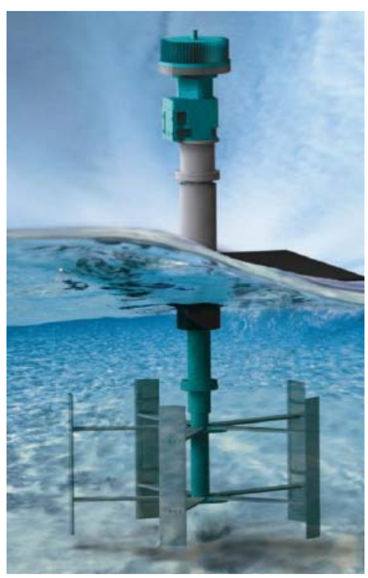

Figure 2.3: New Energy Corporation EnCurrent turbine [10]

These turbines, ranging from 5 kW to 250 kW, cost around $4,000 per installed kilowatt [10]. Compared to other forms of power generation (see Figure 2.4) hydrokinetic turbines are relatively expensive, but it is likely the cost will decrease in the next few years as the market matures and more turbines reach the commercial stage of development. For this study, the EnCurrent turbine is used at its projected size and cost. Installation costs and other specifications of the EnCurrent product line can be seen in Appendix B.

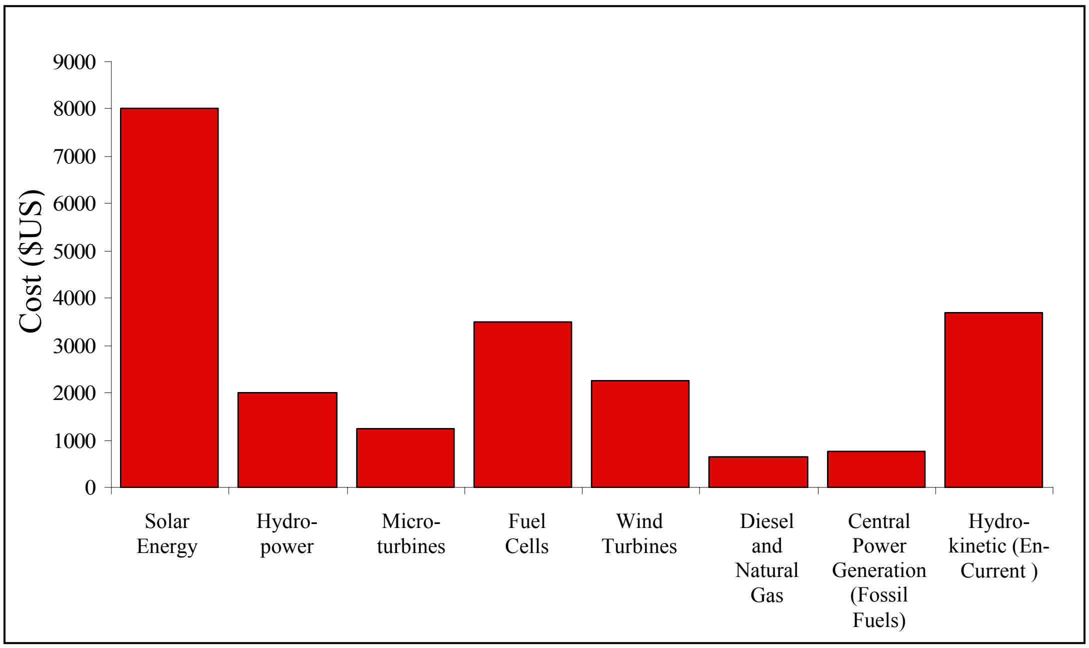

Figure 2.4: Average Cost per Installed Kilowatt for Various Energy Sources [11]

3. Conventional Hydropower

3.1 Introduction

Currently, hydropower provides 96% of the renewable electrical energy and 10% of the overall electrical energy within the US [12]. This technology is considered one of the most effective forms of renewable energy as some designs have an efficiency of up to 90% [12]. Since several dams were built during the 1960's, the long-term effects on salmon populations in particular have been observed [5] and a push for dam removal has resulted in the elimination of 715 dams nation wide [7]. Since the issues related to conventional hydropower are not of major concern within the irrigation system due to its lack of aquatic life, this form of hydropower is also considered for this project.

3.2 Theory

Conventional hydropower is based on the potential energy of water. Typically the design for conventional hydro consists of a dam used to build a reservoir, which then supplies water into a penstock. Turbines located at the downstream end of a penstock then utilize the head pressure of the water to generate energy. A common design of this system can be seen in Figure 3.1.

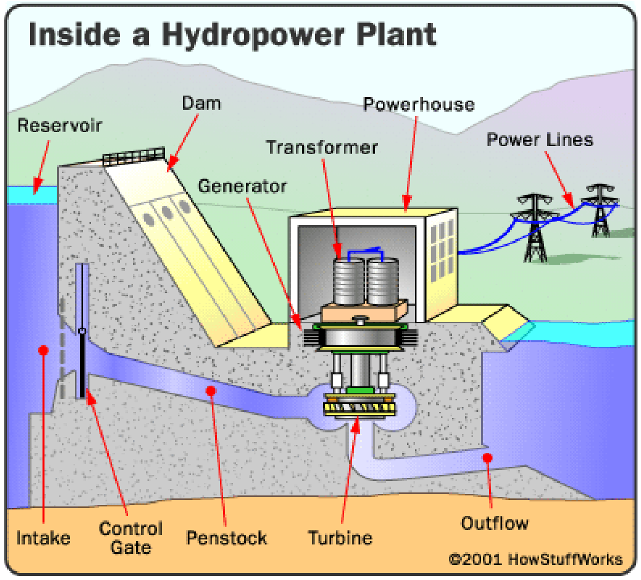

Figure 3.1: Typical Design of a Conventional Hydropower System [13]

Since this design is based on the head created by the reservoir, the potential energy equation can be used to achieve an estimate for the power resource (see Equation 3.1). For the purposes of this study, a complete analysis of conventional hydropower will not be completed but will be used to compare against hydrokinetic technology.

Equation 3.1

$$PE = \rho\, Q\, g\, H\, 10^{-3}$$

In this case *PE* is the power (W), ρ is the density of water (kg/m^3), Q is the flow rate (m^3/s), g is the acceleration of gravity (m/s^2), and H is the available head (m).

4. Site Assessment

4.1 Irrigation System Design

The Columbia Basin Project is an irrigation system consisting of canals, laterals and turnouts. Laterals branching from the West and East Low canals each divert 100 to 500 cfs of water closer to farmland. Once water enters the laterals it is delivered to farms via turnouts. Check stations on the laterals are used to regulate the upstream depth and velocity to assure adequate flow and also to assure that erosion does not occur due to excessively high velocity. The amount of water that enters each turnout is controlled by the lateral check station located downstream of each diversion. Figure 4.1 is a general depiction of the canal/lateral/turnout system including the common location of check structures.

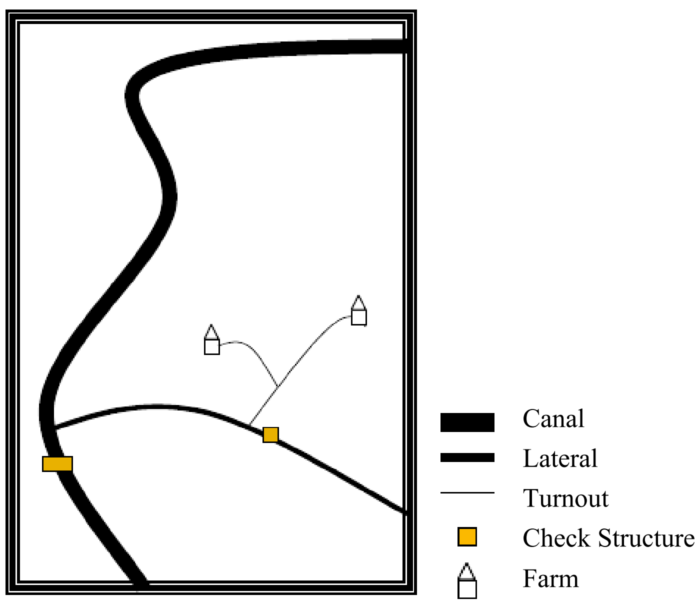

Figure 4.1: Irrigation System Design

Most check structures are designed with one to four radial gates and are monitored by Supervisory Control and Data Acquisition (SCADA) [14]. SCADA monitors the water depth and velocity upstream of a check structure and notifies a central computer if adjustments to the gates' positions are necessary to maintain desired conditions. Since the majority of the channels are earthen-lined, the velocity of the flow should not exceed 2 ft/s [15], to prevent erosion. Although not all check structures are connected to the SCADA system, enough are equipped so that the flow throughout the irrigation system can be maintained and supervised without need for on-site visits. Figure 4.2 is an aerial view of a check structure located on the West Canal with a lateral diverting water upstream. This structure is located just south of Ephrata, WA.

Figure 4.2: Arial View of Check Structure and Lateral Diversion [16]

4.2 East Low Lateral 68: Check 2

Check 2 is a unique check structure located on Lateral 68 of the East Low Canal (EL68) north of Othello, WA. Created primarily to slow the velocity of the flow, it was also constructed to regulate the flow, preventing check structures downstream from being washed out by surges [17].

This check structure (Figure 4.3) consists of a 10 ft radial gate adjacent to four rows of stop planks that serve as an overflow weir. The radial gate is controlled by the SCADA system and can be adjusted remotely while the stop planks provide manual control of the channel depth [17]. The stop planks also act as a spillway when the gate position is in need of alteration, as this process is carried out slowly to prevent surges [18]. During an alteration of the gate position, the spillway also prevents the flow from overtaking the structure and causing severe damage.

Figure 4.3: East Low Lateral 68 Check 2 (early September 2008)

The channels both upstream and downstream of Check 2 are earthen-lined, trapezoidal channels with a base width of 20 ft [19]. The upstream and downstream channel depths are maintained at 6.9 and 6.7 ft, respectively, with the design flow rate being 414 cfs. Although water can be seen passing over the stop planks in Figure 4.3, the majority of the flow passes under the radial gate. After the water passes under the gate the flow becomes “supercritical” (fast flowing and relatively shallow with Froude Number, $F > 1$) and drops over 15 ft down a Portland cement concrete lined channel [19]. At the base of the decline are baffle blocks, commonly known as “dragon’s teeth”, which in turn cause a hydraulic jump that dissipates energy, returning the flow to a subcritical state.

The concrete-lined, rectangular channel transitions back to a trapezoidal channel followed by a concrete apron to further slow the flow. The channel then returns to an earthen-lined, trapezoidal channel, shown in Figure 4.4 (not drawn to scale). In Figure 4.4, “a” is the shallow subcritical, “b” the hydraulic jump, and “c” is the subcritical depth flow domain. The blueprints for Check 2 can be found in Appendix C.

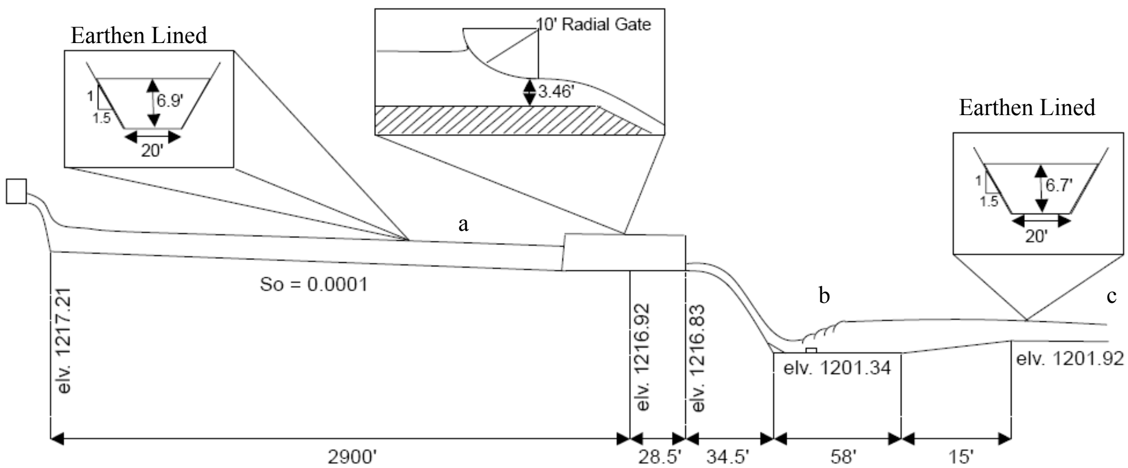

Figure 4.4: Original Design of Check 2 on East Low Lateral 68

Power generation must not interfere with the system’s ability to deliver water to farms. Ideally it would be best to replace “planned energy dissipation” of the current design with generated power, thus leaving the downstream conditions unaffected. To do so, the first step in evaluation is determining locations where energy is actually lost.

Earthen-lined channels have a moderate Manning’s coefficient (around 0.02 [20]) and result in energy loss due to friction. With the present design, 7.7 kW is lost to channel friction in the upper canal. Lining the channels with Portland cement concrete could lessen this loss, but this has not been done because of the high cost of civil works. By lining the channel with concrete, only 3.9 kW would be lost resulting in 3.8 kW of power that could be used for generation without affecting downstream conditions. Details of this calculation are presented in Appendix D.

In addition, the baffle blocks located at the base of the drop were designed to dissipate a considerable amount of energy. By evaluating the conditions before and after the baffle blocks and the resulting hydraulic jump it is determined that almost 600 kW are lost in this section of the present system (see Appendix D). Although necessary to slow the velocity of water in the channel and revert the flow to a subcritical state, it is possible to use hydropower, rater than baffle blocks, to provide the same energy dissipation. Examination of this system is the primary goal of this study.

The next step in this evaluation is to determine the amount of available water power, both kinetic and potential, that is present in the system. Utilizing Equation 2.1, the kinetic resource on a channel cross-section is 3 kW (see Appendix D). The available hydraulic head of Check 2 is similar to most conventional low-head small hydropower designs. When the canals were constructed, however, the low cost of electricity made power generation a low priority and so it was not included [14]. A potential head of approximately 23 ft is available. Using Equation 3.1 it is found that there is nearly 800 kW of potential power in the flow.

4.3 East Low Lateral 68: Check 5

Check 5, located downstream of Check 2, was originally a 10 ft wide radial gate with stop planks on either side. In 1996 the structure was washed out and, since there are no active turnouts upstream, the need for flow control was no longer necessary. The structure was never rebuilt and so in its place is a drop resulting in a hydraulic jump, as can be seen in Figure 4.5.

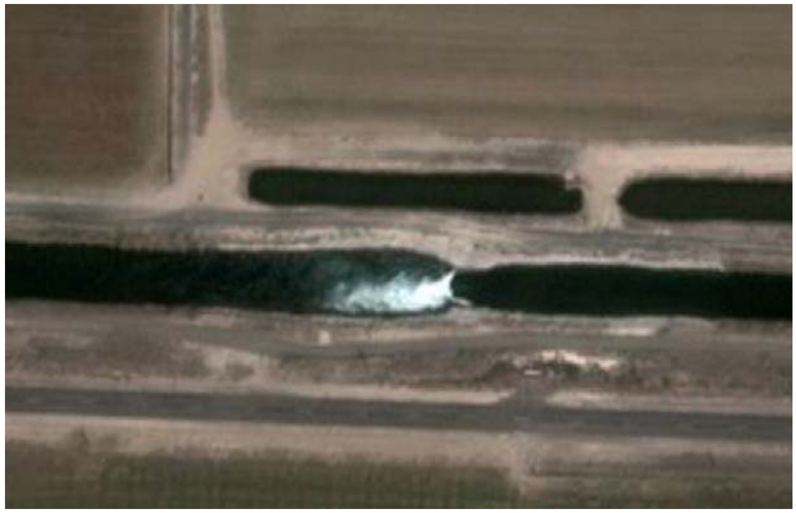

Figure 4.5: Aerial View of Check 5 Located on East Low Lateral 68 [21]

Similar to Check 2, the channels on either side of the washed out check are trapezoidal, earthen-lined, with the upstream channel having a base width of 20 ft and the downstream channel having a base width of 18 ft. The water depths in the upper and lower channel are approximately 5 ft and 6.7 ft, respectively. Overall, the flow drops over 10 ft and has a designed flow rate of 379 cfs. Figure 4.6 (not drawn to scale) provides a sketch of the channel design and the approximate water depths. A blueprint of this check giving its elevations can be seen in Appendix E.

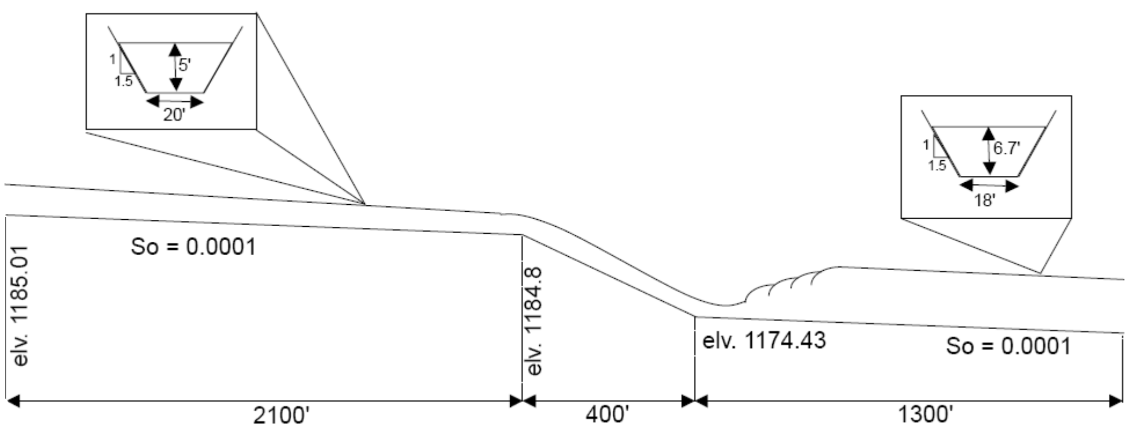

Figure 4.6: Design of Check 5

As there are no active turnouts upstream, this part of the lateral has the potential to be greatly altered. The hydraulic jump currently dissipates 155 kW of power. In addition to this, replacing the upper channel with concrete would allow for 9 kW to be extracted without affecting downstream conditions. Details of these calculations are presented in Appendix F.

The kinetic energy of the upper channel is similar to Check 2, with a resource of 5 kW. The potential energy, however, is significantly higher. From the top of the channel to the bottom of the drop there is an estimated head pressure of 15.6 ft, resulting in almost 340 kW of potential energy in the flow, using Equation 3.1.

4.4 East Low Lateral 29: "Cemetery" Check

This check, located on Lateral 29 off of the East Low canal (EL29), is named the "Cemetery" as it is located near the Moses Lake Cemetery. This structure consists of a single center hinge gate with spillways on both sides and turnouts located upstream. Due to this, alterations to the structure are unlikely as the upstream flow is sensitive to any changes. Below the check structure, however, is a long, sloped, trapezoidal chute (see Figure 4.7) which carries the water over a quarter mile and drops over 30 feet in elevation.

Figure 4.7: Downstream of “Cemetery” Check (early September 2008)

This chute leads to a curved drop of 8.85 ft. Located at the base of this decline are a set of baffle blocks, designed to dissipate energy and reduce the velocity of the flow before it returns to an earthen-lined, trapezoidal channel. Figure 4.8 (not drawn to scale) is a sketch of the “Cemetery” Check. Blueprints of this check can be found in Appendix G.

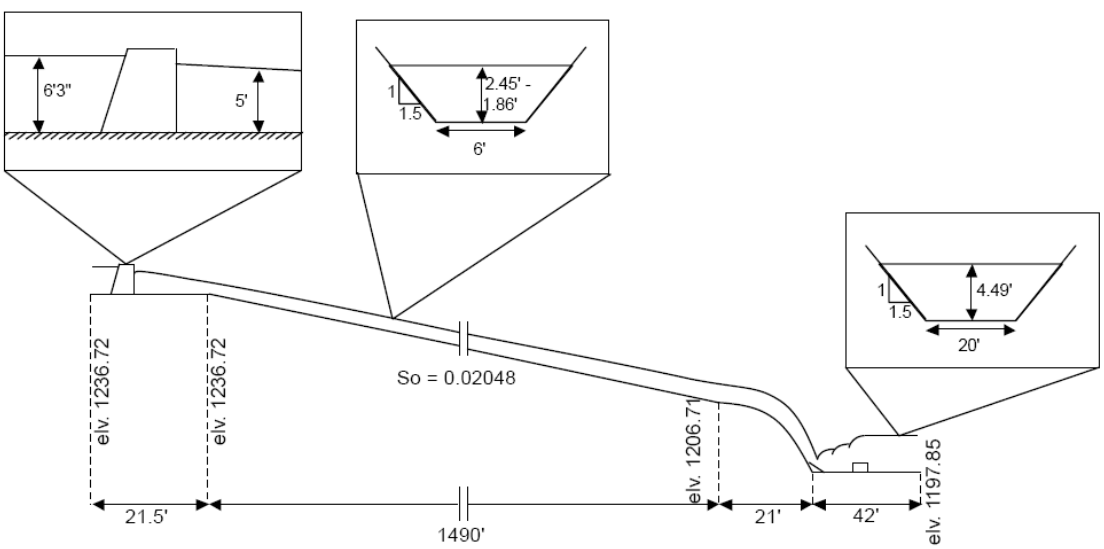

Figure 4.8: Sketch of “Cemetery” Check on East Low Lateral 29

Since this portion of the channel is already lined with Portland cement concrete, the largest existing loss of energy occurs in the hydraulic jump created by the baffle blocks. It is estimated that 300 kW are dissipated in the jump. The head pressure within the chute, including the drop, is about 40 ft with the potential power of the flow being 1180 kW at a flow rate of 316 cfs. Since the flow in the chute becomes supercritical, resulting in a higher velocity, the kinetic resource in this section of the design is around 63 kW. Details of these calculations may be found in Appendix H.

5. Evaluation Process

5.1 Hydrokinetic Design Variables and Constraints

The channel width, number of turbines, and turbine size are the three characteristics varied to determine the design for optimal power generation.

For hydrokinetic turbines to be effective, a velocity of about 3 m/s is needed. In addition to this, the water depth must be sufficient so that the turbines remain fully submerged. In general, the natural velocity in the channel can be increased by reducing the width, keeping the flow subcritical to provide a reasonable depth. It is important to constrict the channel for as short a length as possible to keep wall friction loss low and minimize construction cost.

When constricting the flow, however, it is crucial to avoid reaching the critical depth flow condition, as this would cause the flow to choke. Choked flow occurs when there is an unsuitable mix of potential and kinetic energy to maintain the given flow rate. The flow rate is maintained by the upstream depth increasing with a corresponding reduction in velocity. This can cause a “backwater” of increased depth for a considerable distance up-channel, depending on the channel slope. Because of this, the channel will not be constricted past its “critical width”, which is the width that would cause an alteration to the flow [22].

The size of the hydrokinetic turbine will be varied as well. The rated power of a turbine increases with its cross-sectional area; only 5, 10 and 25 kW rated turbines are commercially available. Therefore, the effect on the flow and power generation will be evaluated only for these sizes. Specifications for these turbines are listed in Appendix A.

Table 5.1: Relation of Rated Power to Cross-Sectional Area for a Turbine

Turbine Size (kW)	Cross-Sectional Area (ft^2)	Required Water Depth (ft)	Required Channel Width (ft)
5	12.43	2.49	4.99
10	24.87	4.99	4.99
25	62.22	5.58	11.15

The final variable of the hydrokinetic design is the number of turbines placed in a channel. To provide a range and give an overall understanding of the effect multiple turbines can have, cases are considered with up to 10 turbines evenly spaced along the length of the channel.

5.2 Standard Step Method

The Standard Step Method is the primary technique used in this study to model the steady-state flow rate, depth and velocity variations within a channel. This method is used to determine the water depth when a channel is narrowed as well as how much the upstream depth increases due to the blockage of the flow by the turbine. This method will also help determine the effects on water depth when power is extracted and the changes in the downstream conditions.

The Standard Step Method assumes that the flow domain varies gradually (known as gradually varied flow) and breaks the channel into sections of finite length. The underlying theory of the Standard Step Method is that the energy of the upstream end of a section is equal to the energy at the downstream end plus the energy loss between the sections. Equation 5.1 is a simplified energy balance for the Standard Step Method.

Equation 5.1

$$\boldsymbol{E_1 + z_1 + H_L = E_2 + z_2}$$

As shown in Figure 5.1, E is the energy of the flow (expressed as energy/unit weight, ft), z is the elevation (ft) and H_L is the head loss based on the friction of the channel. In Figure 5.1, L is the distance between Station 1 and Station 2, and the flow moves in the direction indicated by the arrow. For a more thorough explanation of the Standard Step Method, see Appendix I.

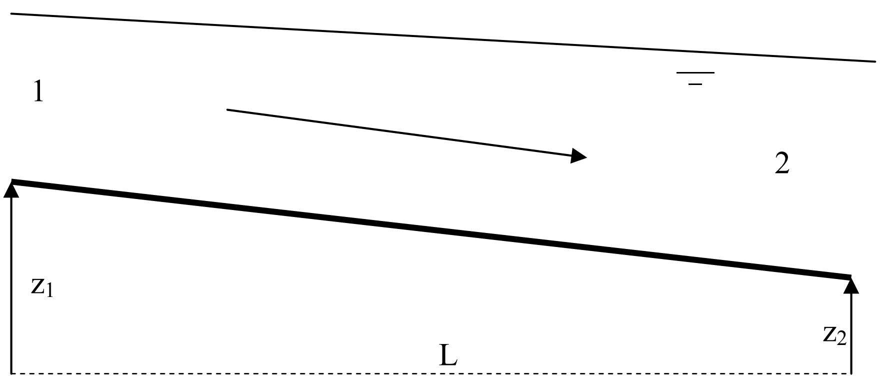

Figure 5.1: Diagram for Standard Step Method

In this study, the Standard Step Method is used to determine the original channel depths for each site, as well as the water depth when hydrokinetic turbines are extracting power from the flow. The calculations of initial channel depths for each case can be found in Appendix J.

5.3 Channel Blockage and Channel Constrictions

The first step in determining the effects of channel constrictions and blockages is to determine the initial depths of the channel using the method described in Section 5.2. Once completed, the depth at the upstream portion of the channel is determined and so the channel can now be evaluated for flow alterations due to the constriction of the channel.

The channel is constricted to accelerate the subcritical flow for higher kinetic power. In order to simulate a channel constriction, a Standard Step calculation is completed for the section (labeled 1 in Figure 5.2) starting at the upstream check structure. When the channel is narrowed, an energy balance is performed between the first section (1) and the start of the narrowed section (labeled 2a). Assuming that there is no energy loss due to the contraction of the channel and no change in the bottom elevation, E_1 is equal to E_2 and the depth at the beginning of section 2 can be determined. The second section is then evaluated using the Standard Step Method to determine the depths and velocities along the narrowed portion. At the end of the narrow section (labeled 2b) another energy balance is performed to determine the depth of the flow at the beginning of section 3. Once this depth is determined, the Standard Step Method is used to evaluate the remainder of the third section to the downstream check.

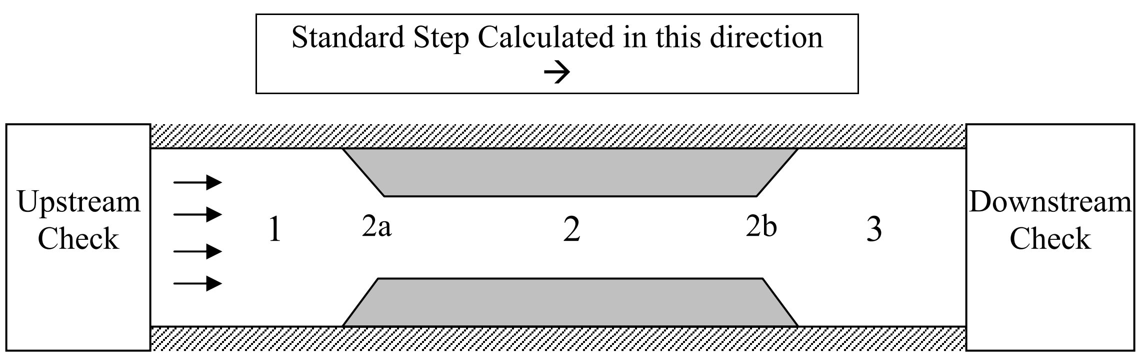

Figure 5.2: Sketch of Channel Constriction

The next step in this process is to simulate the blockage caused by a hydrokinetic turbine. A Darrieus style turbine, which is used in this study, has been found to create a blockage equal to ¼ of its cross-sectional area when placed in a channel [23]. This blockage has an effect that is similar in theory to a bridge pier and can be represented as a constriction of the channel (see Figure 5.3).

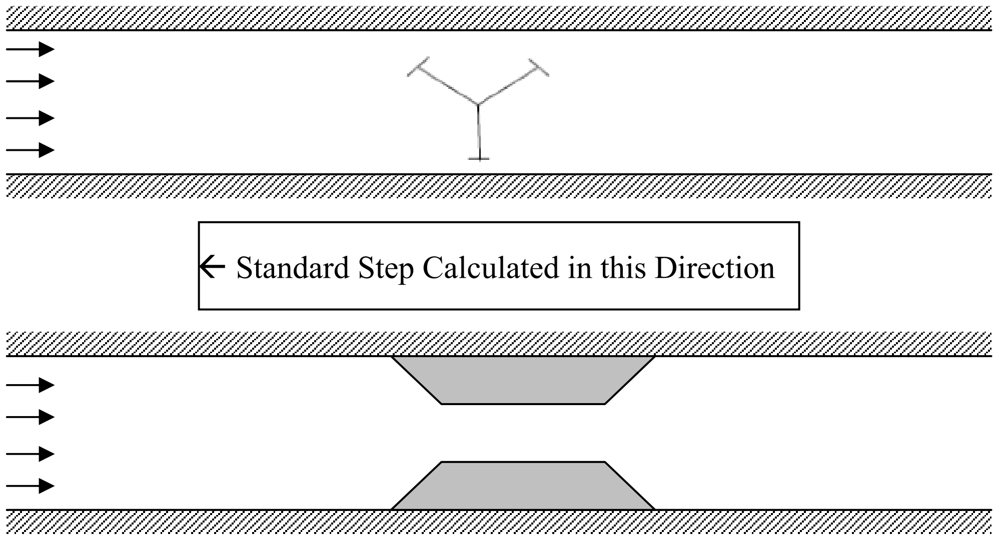

Figure 5.3: Sketch Turbine in Flow Replace by a Channel Constriction

The effective cross-section of the channel with the turbine is reduced by ¼ of the turbines cross-sectional area. Since the turbine acts as a blockage in the flow, it is assumed at this point that the flow upstream increases in depth relative to conditions without the turbine. In order to determine how much the upstream depth increases the Standard Step method is used, this time using the downstream end as the known depth (as noted in Figure 5.3). When the turbine is placed in the flow, the downstream depth is known from baseline conditions and evaluation proceeds upstream to determine the new upstream depth.

5.4 Garrett and Cummins

The next step is to incorporate power generation into the evaluation. The equations presented by Garrett and Cummins [24], which include the effects of the channel width and the mixing of the turbine flow wake, are used. Figure 5.4 shows the stream tube theory used in their analyses of a single turbine in a channel.

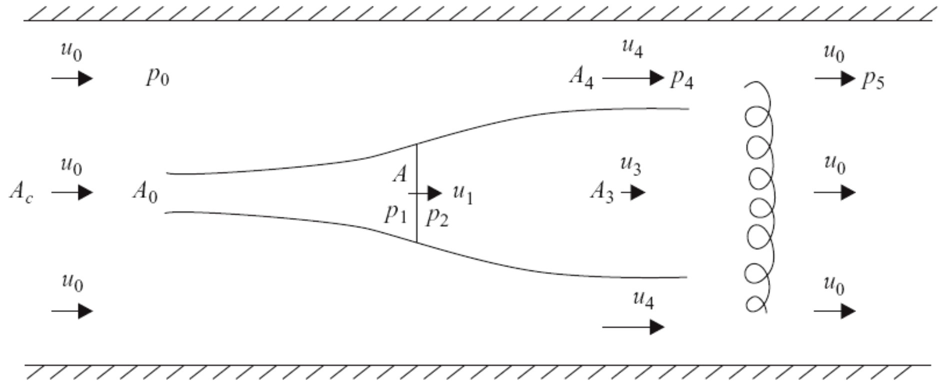

Figure 5.4: Definition Plan View Sketch of a Single Turbine in a Channel [24]

In Figure 5.4 above, A_c is the flow cross-section area and u_o is the one-dimensional (1-D) average incoming velocity of the flow. With A being the cross-sectional area of the turbine, the theory predicts that the velocity of the flow going around the turbine (u_4) will increase due to conservation of mass, while the velocity within the stream tube decreases as power is extracted. These two streams then combine downstream of the turbine and additional energy is lost to turbulence in their mixing.

Garrett and Cummins' study indicates that the total energy extracted from a channel is not only the power extracted by the turbine but also the resulting thermal energy lost due to the mixing in the wake. To calculate the total loss, the power generated must first be determined. Using conservation of mass, energy, and momentum it is found that the power generated depends on the cross-sectional area of the turbine, as well as the velocity of the water before, after, and adjacent the turbine (Equation 5.2).

Equation 5.2

$$P = 0.5\, A\, \frac{u_3\,(u_4 + u_3)\,(u_4^2 - u_3^2)}{u_4 + 2\,u_3 - u_o}$$

For this calculation P is the power extracted by the turbine (W), A is the cross-sectional area of the turbine (m^2), u_o is the approaching velocity of the water (m/s), u_3 is the velocity of the flow exiting the turbine (m/s), and u_4 is the velocity of the water going around the turbine (m/s). All velocities are 1-D average values.

The total power removed from the flow, however, is greater than the extracted power, due to the additional energy lost when the free-stream mixes with the wake. The total power removed, referred to as the "reference power" or P_{ref}, can be found using Equation 5.3, where u_1 is the velocity in the stream tube at the turbine. For a more complete explanation of these formulas, please refer to Appendix K.

Equation 5.3

$$\frac{P}{P_{ref}} = \frac{u_1}{u_o}$$

Once the reference power is determined it is then put in terms of a head loss using Equation 5.4 and included in the Standard Step evaluation. For this equation, P is the power (kW), H_L is the head loss (ft) and Q is the flow rate (cfs). The coefficient 0.085 is used to balance the units so that the power is in terms of kilowatts

Equation 5.4

$$P_{ref} = 0.085\, H_L\, Q$$

Using the previously determined upstream depth for a channel, the Standard Step Method is run again in the direction of the flow (from Station 1 to Station 2 in Figure 5.5). This time, however, the head loss created by power extraction from the channel is included, which results in lower energy downstream.

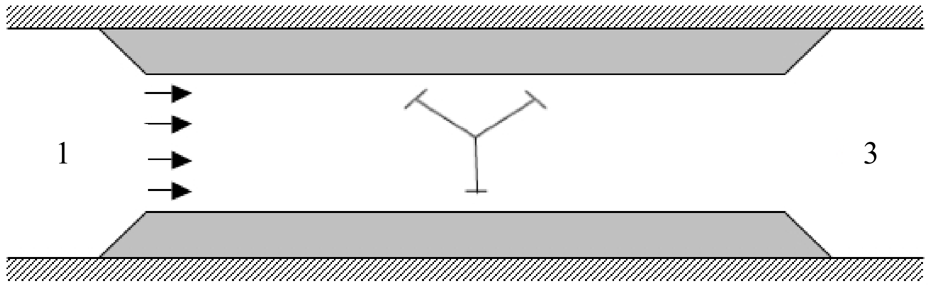

Figure 5.5: Channel Constriction with Turbine

When placing a hydrokinetic turbine in a lateral, the flow may choke due to the blockage of the turbine and the head loss created when power is extracted. The flow may begin to choke when multiple turbines are placed in a channel as the head loss at each turbine decreases the depth of the water, bringing the flow to closer to the critical condition. In some cases, the number of turbines is not the issue but the turbine blockage ratio (ratio of cross-sectional area of turbine to cross-sectional area of the channel).

5.5 Determining the Optimal Design

Once all evaluations are completed, the capital cost per kilowatt for each case is compared to determine the optimal design. As a rule, any cases that produce less than 10 kW or any cases whose capital cost exceeds $600,000 are excluded. The threshold of 10 kW is picked since the cost of installation would make anything impractical. The maximum cost is set at $600,000 as costs higher than this are excessive for the amount of power generated. The top 5 situations having the lowest cost per kilowatt will then be presented, with the cheapest cost per kilowatt being the optimal design for the site.

6. East Low Lateral 68 Check 2: Results

6.1 Hydrokinetic Evaluation: Technical and Economic

Check 2 provides several challenges and opportunities in terms of power generation potential because of the complexity of the check structure and the upstream channel design. The goal of this design is to harness most of the energy that is currently dissipated across the baffle blocks with hydrokinetic turbines. Although the energy dissipated is significant, the amount of extractable kinetic power is less due to the constraints of the channel design.

Since hydrokinetic turbines are most effective in a high velocity, replacing the baffle blocks with hydrokinetic turbines at the base of the drop was considered, but found to be infeasible. The supercritical flow is not only too shallow to submerge a turbine, but the turbines would almost certainly trigger a hydraulic jump. Turbines are not designed to operate within a hydraulic jump and would likely be damaged. Because of this, the turbines are placed upstream of the gate (see Figure 6.1). This uses the gate as a control mechanism and dissipates power prior to the baffle blocks without placing the turbines in supercritical flow. Baffle blocks could be adjusted for reduced friction loss in order to maintain the downstream conditions.

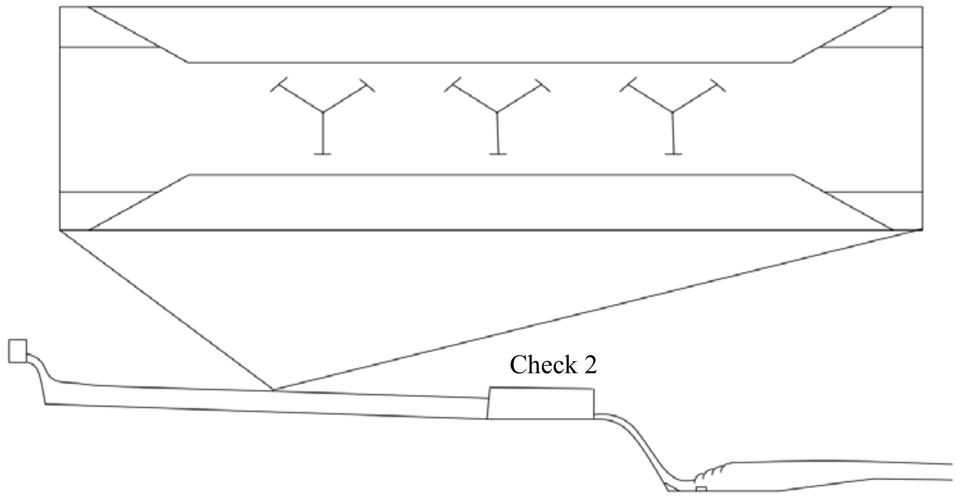

Figure 6.1: Placement of Turbines in Relation to Check 2.

As mentioned in Chapter 4, the kinetic resource on a cross-section of the original channel upstream of Check 2 is found to be less than 3 kW. Assuming a turbine efficiency of 30% and a blockage ratio of 1/4, only 0.6 kW of power could be generated by a hydrokinetic turbine (see Appendix L). The hydrokinetic power potential of the channel can be improved by narrowing the width of the channel, increasing the velocity of the water (as described in Section 5.3). Using Equation 2.1, the kinetic resource for various channel widths upstream of Check 2 are evaluated and shown in Figure 6.2. Figure 6.2 shows that by constricting the flow, almost 60 kW of kinetic power becomes available without changing the flow rate. The calculations for the kinetic resource are given in Appendix L.

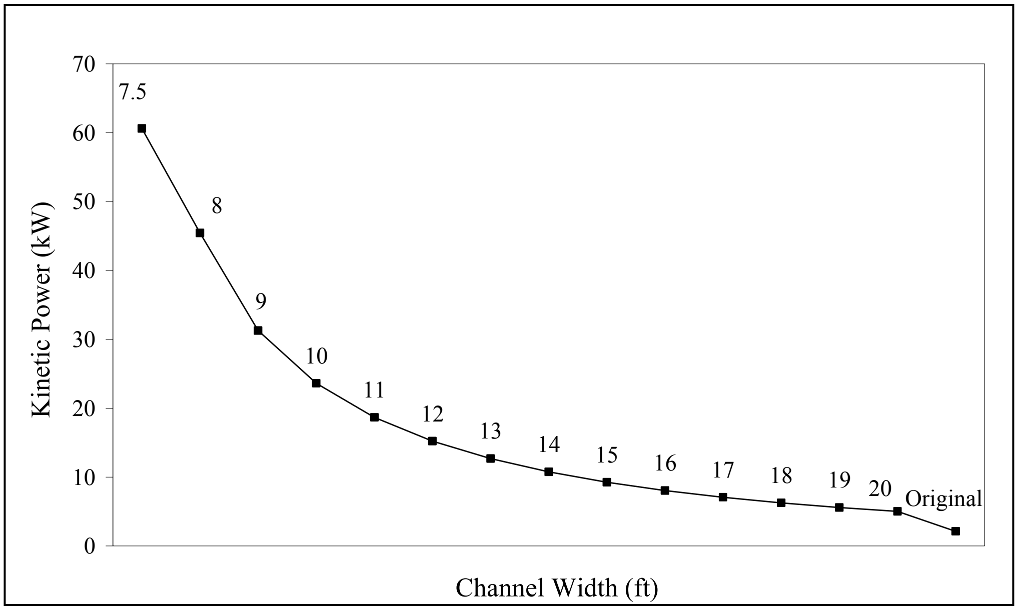

Figure 6.2: Kinetic Power Resource at Check 2 for Various Channel Widths

Although cases such as the 7.5 ft wide channel appear promising for hydrokinetic power, it is important to determine how close each design pushes the system towards the critical point. For the channel upstream of Check 2, which has a design flow rate of 414 cfs, the critical channel width is calculated to be 7.18 ft (see Appendix L). As a precaution, the channel width should never be constricted to this value to assure stability of the flow, thus the 7.5 ft channel would not be an appropriate choice.

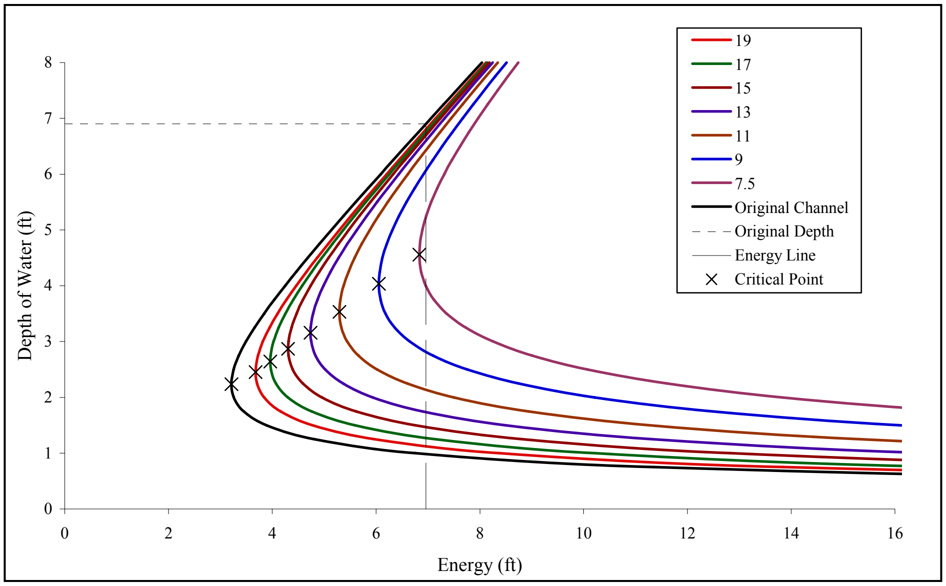

Figure 6.3: Sample of Specific Energy Curves at Various Channel Widths for Check 2

The specific energy curve for each channel design is shown in Figure 6.3. Energy includes both kinetic and potential energy, with the upper (subcritical) curves being predominantly potential energy and lower (supercritical) curves being mainly kinetic energy. The left-most curve in the figure is for the original channel upstream of the gate. The channel is of trapezoidal cross section, with a base width of 20 ft and side slope of 1.5:1. All other curves in this figure assume the design flow rate of 414 cfs for a rectangular channel of width as listed in the legend. The short dashed line represents the design depth of the original channel (6.9 ft), while the larger dashed line denotes its corresponding specific energy (6.9 ft). As the critical energy of the channel denoted by an 'x', located at the critical point, approaches the original energy of the flow, the channel becomes more prone to choking.

The difference in specific energy from the flow conditions to the critical point is the maximum amount of energy that can be extracted from the channel. As a result, narrow channels are rich in kinetic energy but offer less power for generation via hydrokinetics due to the constraints of the channel; with the narrowing of the channel, there is less extractable energy in the flow. Further details of the hydrokinetic calculations relating to Check 2, are given in Appendix L.

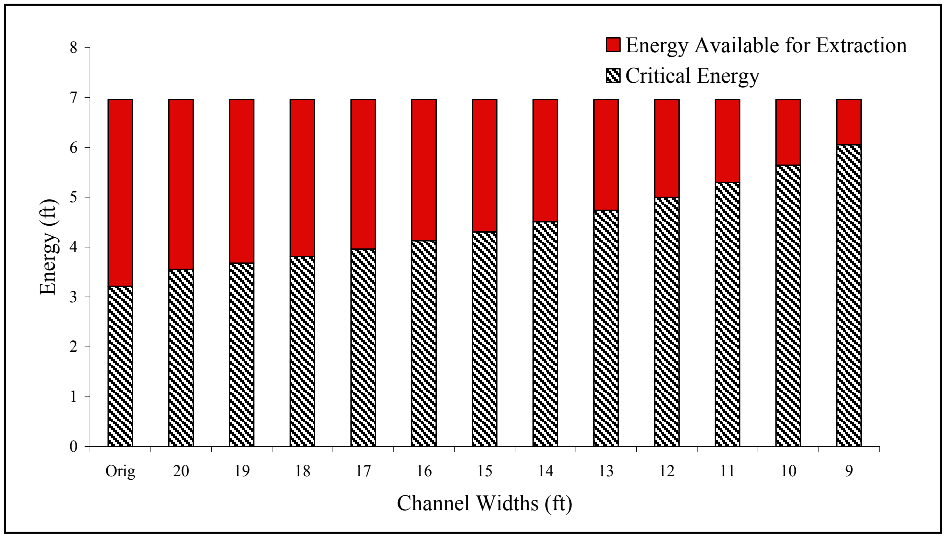

Figure 6.4: Difference between Channel Specific Energy and Critical Specific Energy for Check 2

Using the available energy for each channel width (found in Appendix L), the amount of power that can be extracted based on the width of the channel is calculated and plotted in Figure 6.5. Based on the approach used by Garrett and Cummins, it is found that approximately 10% of the energy removed from a channel is lost to mixing with the wake, leaving the remaining 90% to be extracted for power generation.

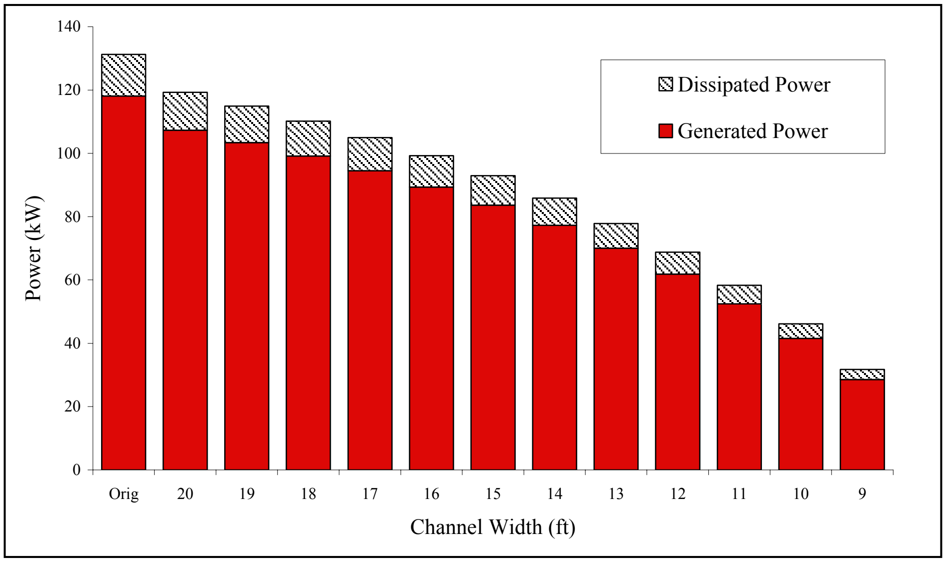

Figure 6.5: Expected Power that can be Extracted Based on Channel Width for Check 2

Power generation is now evaluated using the extraction capabilities of the turbines. Using the methods described in Chapter 5, the channel is evaluated in combinations of turbine size, number and channel width. A sample of the results is given in Figure 6.6, in which the power generated for 1, 2 and 3 turbines, each rated at 10 kW, is shown.

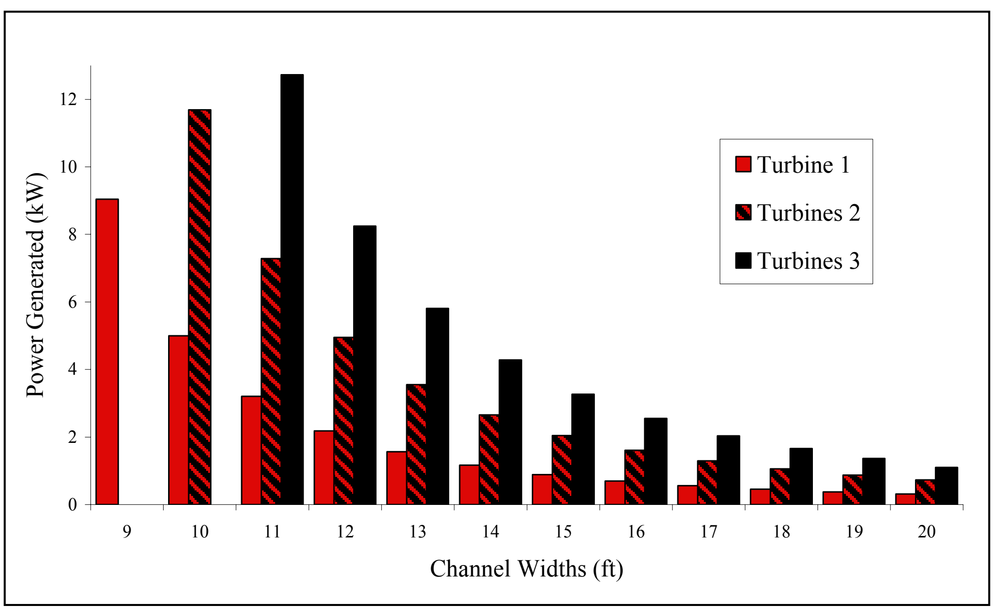

Figure 6.6: Power Generation for a 10 kW Turbine at Various Channel Widths

As expected, the power generated in a wide channel is less than in a narrow one because the slower velocity of the water results in less kinetic energy. This appears to contradict the previous figure (i.e. Figure 6.5), however, as this shows more power is available in the wider channels. In fact, the majority of the available power in Figure 6.5 is based on potential energy for the wider channels and cannot be harnessed using hydrokinetic turbines.

Consider the 20 ft wide channel case. Figure 6.5 shows that almost 120 kW of power is available for extraction, but Figure 6.6 demonstrates that because of the slow velocity of the water, a single turbine generates less than one kilowatt. To take advantage of the available power in the channel with the same 10 kW rated turbine, it is estimated that over 120 turbines would be needed to generate the full amount of extractable power. Not only would this design cost well over $6 million, but the length of the channel may not be long enough to accommodate the turbines.

It can also be seen for a wide channel (at the middle and right of the Figure 6.6) that 2 turbines produce approximately twice the amount of power of a single turbine, and 3 turbines produce approximately triple. It would be expected that for 2 turbines in a 9 ft channel, 18 kW of power would be generated as this is twice the amount of power as a single turbine generates in this condition. 18 kW of extraction would also not cause the flow to choke, as seen in Figure 6.5, so this design appears to be feasible. This 2 turbine design, however, cannot be confirmed for this channel width because of the limitations of the equations used for power evaluation. When the water passes through the first turbine the down channel water depth decreases. This results in a larger ratio between the turbine size and the cross-sectional area of the channel. The Garrett and Cummins method is only applicable for small ratios (less than 0.30 [25]). When the cross-sectional area of the flow decreases, this ratio increases past 0.30, making the equations unfit to use. Until a better method is presented, this ratio is used conservatively, since it is not recommended to let the channel near its critical point.

The economic analysis is also important to this evaluation. Only installation costs are considered for this study. These capital costs include the cost of the turbine, as found in Appendix B, and also some civil works. For the details of the installation costs related to the hydrokinetic design, see Appendix M. An example of the cost per kilowatt evaluation can be seen in Figure 6.7, which evaluates the same 10 kW rated turbine cases as treated in Figure 6.6.

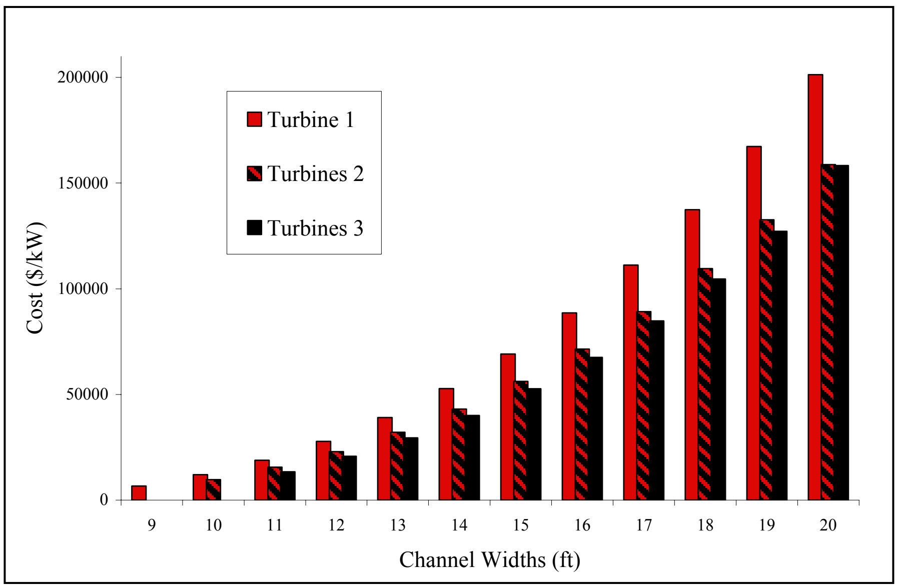

Figure 6.7: Cost per Kilowatt for a 10 kW Turbine at Various Channel Widths

By narrowing the channel, more power is produced leading to a smaller cost per kilowatt. It can also be seen that the cost per kilowatt decreases with the number of turbines in the channel. This is because the head loss caused by the power extraction results in a shallower flow behind the turbine (see Figure 6.8).

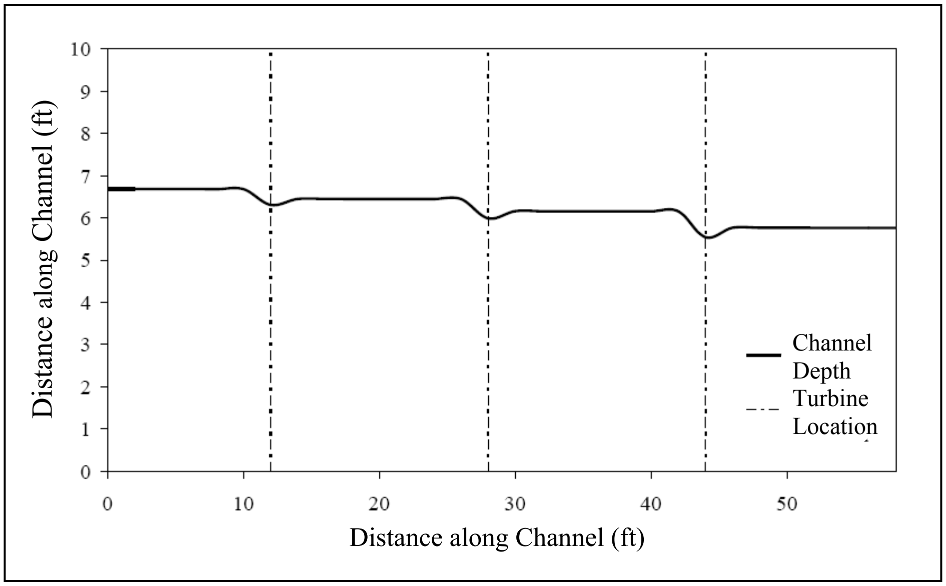

Figure 6.8: Example of Water Depth Profile with Power Extraction

Since the flow rate remains constant, this decrease in the channel depth results in an increase in the velocity of the water following a turbine. An example of this increase can be seen in Figure 6.9.

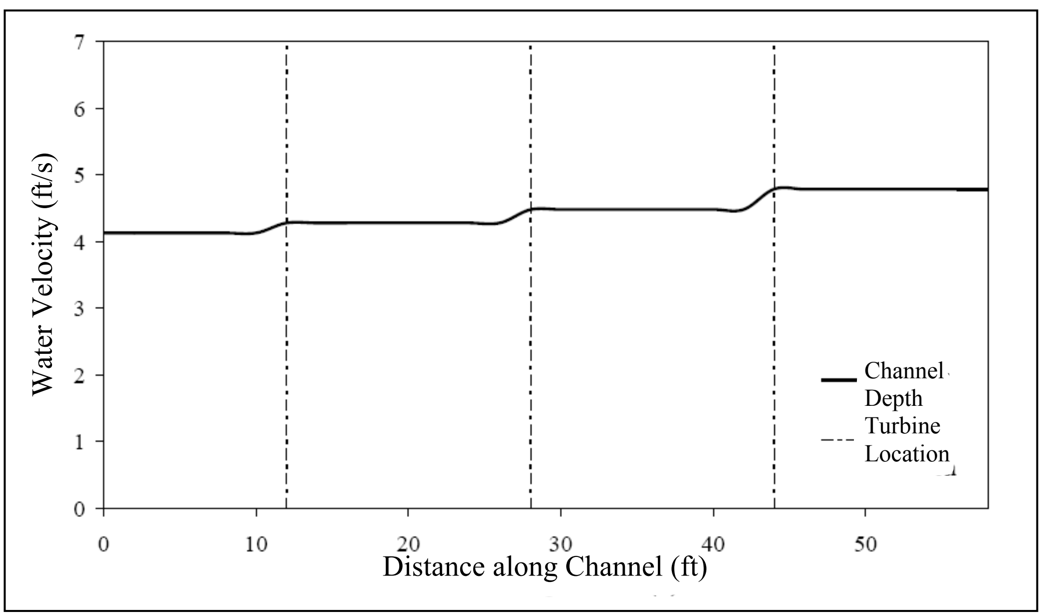

Figure 6.9: Approximate Change in Velocity in a Channel due to Power Extraction

After all cases were evaluated, the cases that exceeded $600,000 in capital costs or generated less than 10 kW were removed. These cases are then ranked based on their cost per kilowatt. The 5 lowest cost cases are compared and can be seen in Figure 6.10.

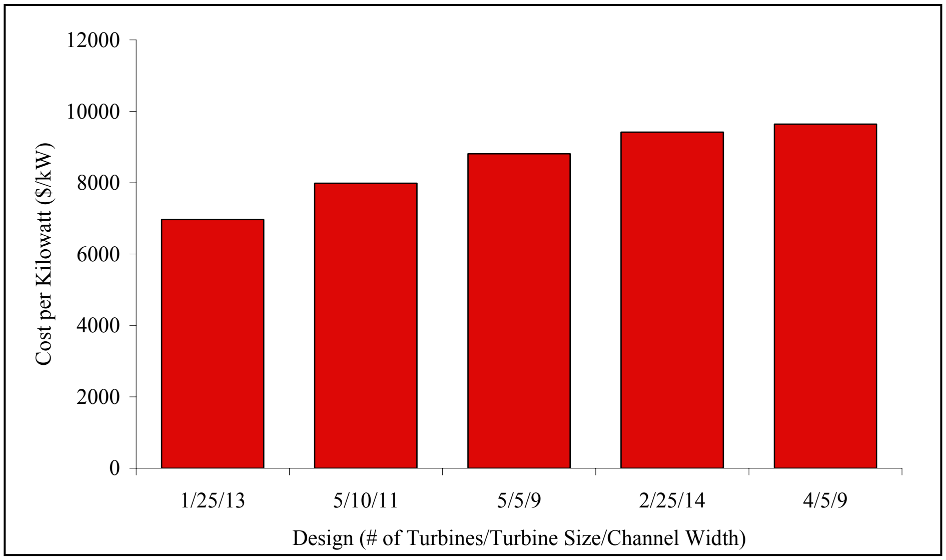

Figure 6.10: Top Five Eligible Low-Cost Design Options for Check 2

For Check 2 the optimal combination of number of turbines, turbine size, and channel width would be a single 25 kW rated turbine in a 13 ft wide rectangular channel. This design would generate almost 18 kW of power and cost over $122,500. Table 6.1 details the amount of power generated and the costs for the 5 options. A list of all designs as well as a cost vs. generated kilowatt comparison for all cases can be seen in Appendix N.

Table 6.1: Top Five Hydrokinetic Design Options for Check 2

Rank	Turbine Size (kW)	# of Turbines	Channel Width (ft)	Expected Power (kW)	Capital Costs ($US)	Cost per Kilowatt ($US/kW)
1.	25	1	13	18	$122,500	6,800
2.	10	5	11	21	$170,000	8,000
3.	5	5	9	17	$153,000	9,000
4.	25	2	14	25	$237,000	9,500
5.	5	4	9	13	$125,000	9,600

6.2 Conventional Hydropower Evaluation: Technical and Economic

Based on the characteristics of the site, it is first important to select the appropriate turbine for conventional hydropower generation. Using Figure 6.11 it is determined that the optimal turbine for this site is a Kaplan turbine since the site has a flow rate of 414 cfs (11 m^3/s) and a pressure head of 23 ft (7 m).

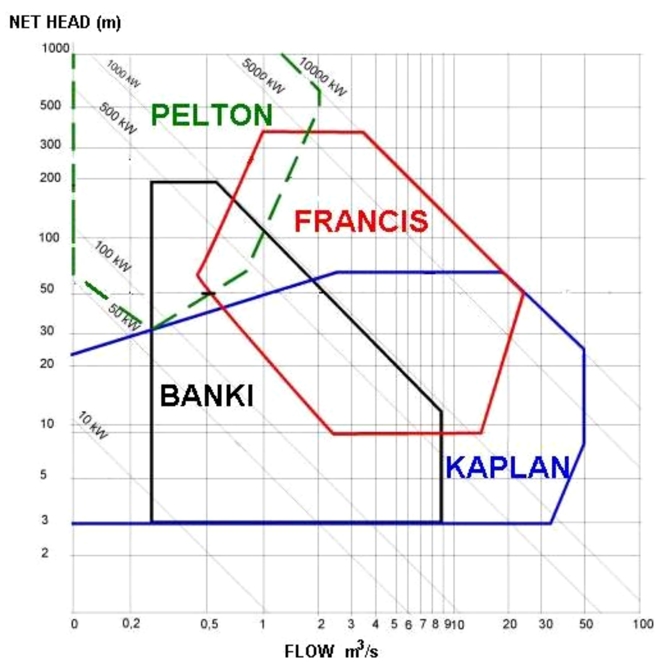

Figure 6.11: Hydropower Turbine Selection Graph [26]

The next step is to estimate the amount of power that could be produced using the potential energy at the site. This was calculated using a modified version of Equation 3.1 to account for the turbine efficiency (η).

Equation 6.1

$$P = \eta \rho Q g H$$

An efficiency of 90% is expected when the turbine is functioning under ideal conditions, although the operating efficiency may be lower [27]. When operating under optimal conditions, a conventional hydropower design could generate over 700 kW (see Appendix O for calculations). According to the Wales ECO Centre, a conventional hydropower system that has less than 60 meters of head will cost typically around $5,000 per installed kilowatt [28]. Using this information, the cost to build this conventional hydropower system is found to be about $3,500,000.

6.3 Preferred Option

For Check 2 the cheapest cost per installed kilowatt case using hydrokinetic turbines is at least $6,800/kW, while the average for conventional hydropower is around $5,000/kW. Though the difference of these two options is not large in cost efficiency, the conventional hydropower design has the capability to generate a considerable amount of power (over 500 kW), while the optimal hydrokinetic design produces about 20 kW. It is also significant to mention that the EnCurrent turbine has a life expectancy of about 20 years [29], while the expected life of a conventional hydropower design can range anywhere from 25 to 100 years. Since the cost per rated kW is similar, the choice may hinge on the power level desired – 10's of kilowatts versus 100's of kilowatts per site – and the capital outlay – $100,000 to $200,000 versus about $3.5 million.

7. East Low Lateral 68 Check 5: Results

7.1 Hydrokinetic Evaluation: Technical and Economic

The evaluation of Check 5 is similar to the evaluation of Check 2, however inactive turnouts and the absence of a check structure make this site more flexible for incorporating hydropower. Although the velocity of the flow on the drop is considerably faster than in other parts of the channel, its supercritical flow condition is unsuitable for hydrokinetic turbines. There is, however, potential for hydrokinetic power extraction in the upper section (see Figure 7.1).

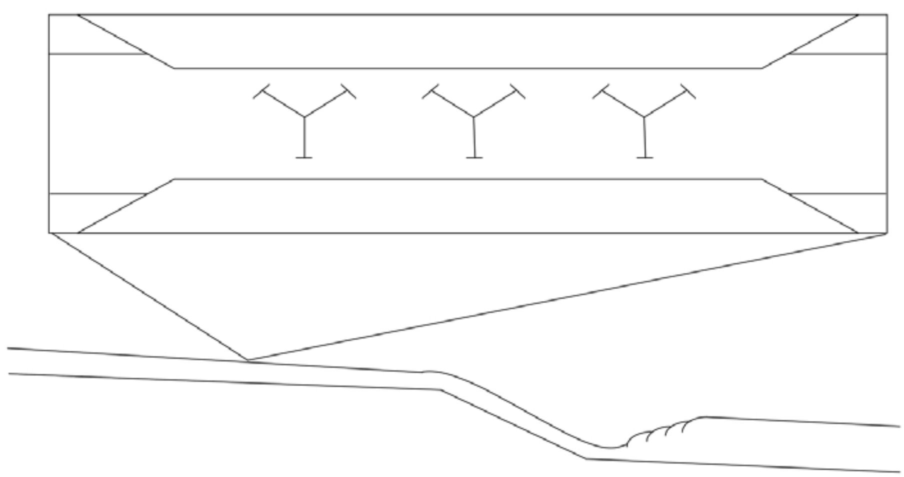

Figure 7.1: Placement of Turbines for Check 5

The original design of the upper channel is unlined with a trapezoidal cross-section. With the base width being 20 ft, the velocity of the water remains under 2.8 ft/s, with 1 kW of kinetic power on each transect. By narrowing the channel it can be seen in Figure 7.2 that the kinetic resource could be increased to over 50 kW if constricted to an 11 ft wide rectangular channel.

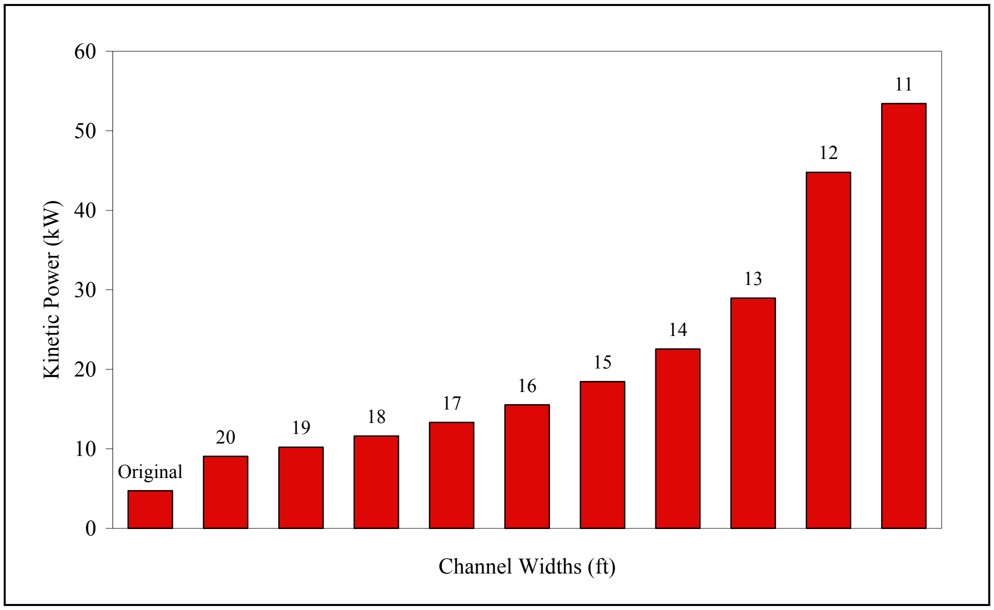

Figure 7.2: Kinetic Energy Resource for Check 5 at Various Channel Widths

As discussed for the previous case, constricting a channel can cause the flow to choke and so it is important to determine the critical width for the given flow rate. As calculated in Appendix P, the critical width for a rectangular channel, having a flow rate of 379 cfs (the rated capacity), is 11.9 ft.

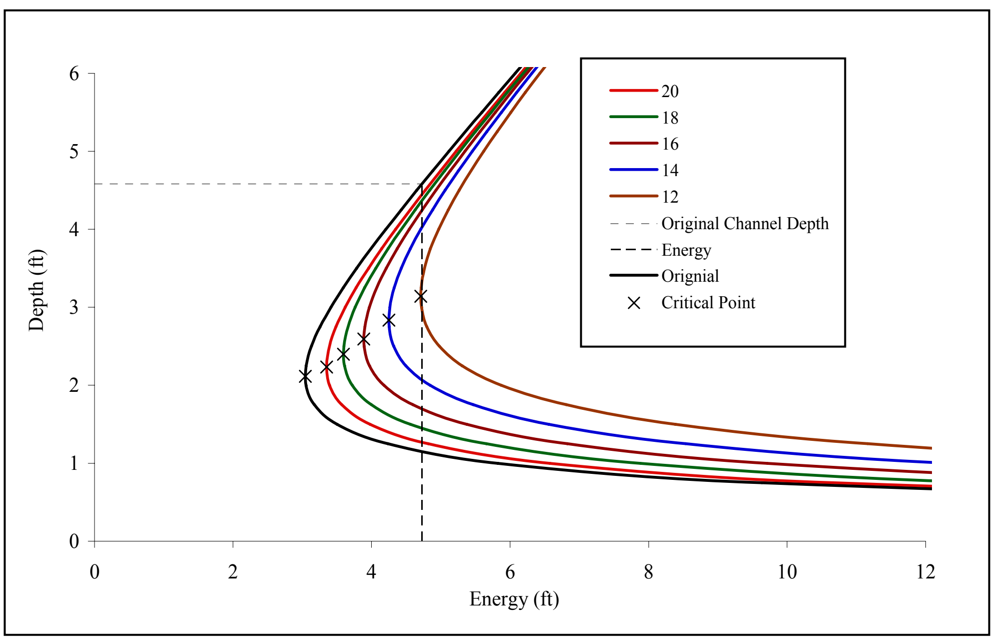

Figure 7.3: Sample of Specific Energy Curves at Various Channel Widths for Check 5

The specific energy curves in Figure 7.3, are used to determine the amount of energy that can be extracted without choking the flow Figure 7.4 shows that the channel has very little available power if constricted below 13 ft.

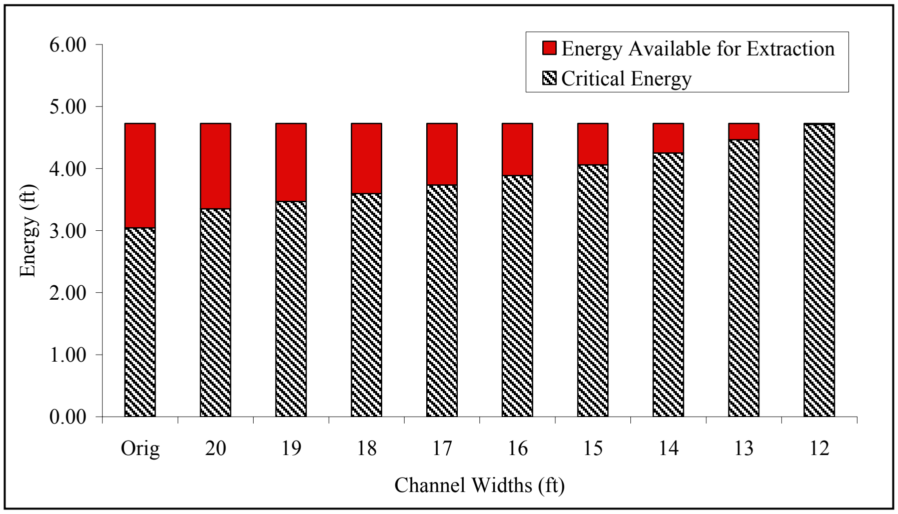

Figure 7.4: Difference between Channel Energy and Critical Energy for Check 5

Using the above information, the amount of power that can be extracted for each channel section is evaluated and shown in Figure 7.5, which also shows the estimated thermal energy losses in the device wake.

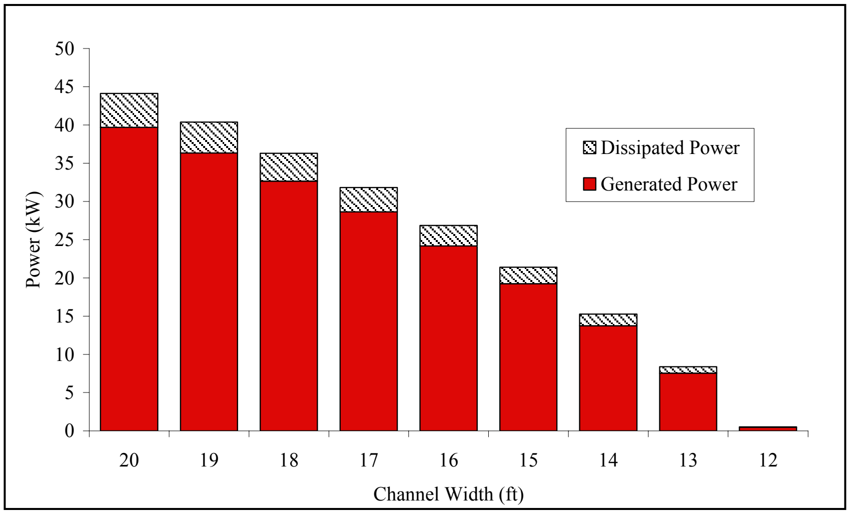

Figure 7.5: Expected Power that can be Extracted Based on Channel Width for Check 5

An evaluation of hydrokinetic turbines similar to that done in Chapter 6 is performed to determine the useful combination of turbine size, number of turbines, and channel width for Check 5. Select results are given in Appendix P.

Including the cost of the turbine and some civil works, as detailed in Appendix M, the cost per installed kilowatt is determined for each case. Cases that cost over $600,000 or generate less than 10 kW are excluded, leaving 20 cases eligible, as listed in Appendix Q. Ranking these results, with the lowest cost per kilowatt being favored, the top five options for hydrokinetic design are shown in Figure 7.6.

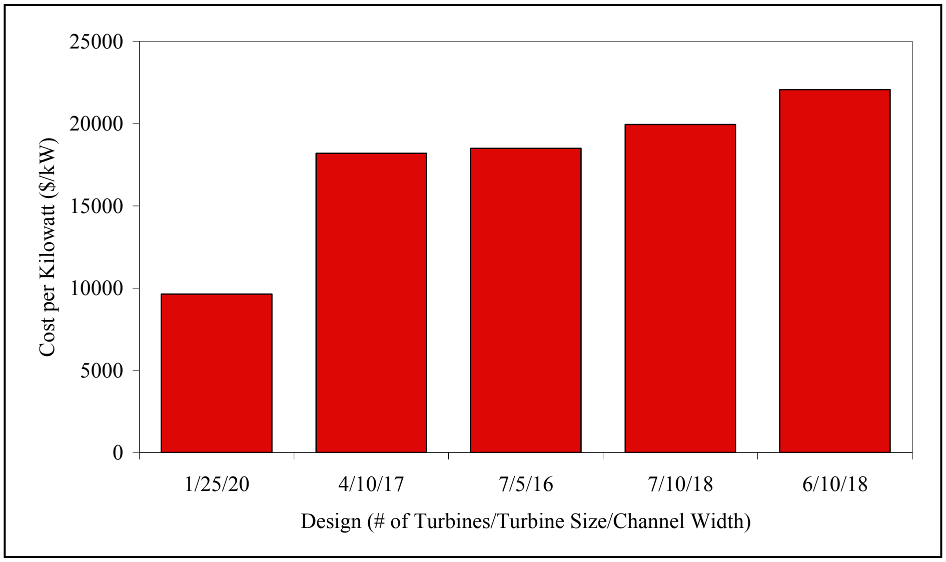

Figure 7.6: Top Five Low-Cost Design Options for Check 5

The lowest cost per installed kilowatt design is a single 25 kW rated turbine in a 20 ft wide rectangular channel. The total cost of this design and the other top five options are detailed in Table 7.1, with all eligible designs detailed in Appendix Q.

Table 7.1: Top Five Hydrokinetic Design Options for Check 5

Rank	Turbine Size (kW)	# of Turbines	Channel Width (ft)	Expected Power (kW)	Capital Costs ($US)	Cost per Kilowatt ($US/kW)
1.	25	1	20	13	$123,000	9,600
2.	10	4	17	13	$227,500	18,200
3.	5	7	16	12	$214,500	18,500
4.	10	7	18	20	$391,000	20,000
5.	10	6	18	15	$338,000	22,000

It can be seen from Table 6.1 and Table 7.1 that the cost per kilowatt from Check 2 is considerably cheaper than for Check 5. This is due primarily to the lower flow rate at Check 5.

7.2 Conventional Hydropower Evaluation: Technical and Economic

The evaluation process for conventional hydropower in this study is detailed in Chapter 6. Using Figure 6.11, for a flow rate of 379 cfs (10.73 m^3/s) and head of 15.6 ft (4.75 m) the preferred turbine is a Kaplan. About 200 kW or power could be generated (Equation 6.1) using this type of turbine at Check 5 (see Appendix R). With a cost per kilowatt around $5,000/kW, total cost of the structure is calculated to be over $1,500,000.

7.3 Preferred Option

For this case, the difference between the capital cost per kilowatt for a hydrokinetic design and a conventional hydropower design is a factor two. With the lowest cost design for hydrokinetics being approximately $10,000/kW and for conventional hydropower being $5,000/kW, it appears that the conventionally hydropower design is a better choice if a capital expenditure of over $1 million is feasible. Conventional hydropower will produce upwards of 300 kW while the hydrokinetic turbines are only capable of 10 to 20 kW.

8. East Low Lateral 29 “Cemetery” Check: Results

Turnouts above of the “Cemetery” Check are active, making alterations to the channel upstream of the structure difficult. Following this check structure, however, is a fast moving, supercritical flow zone that may be well suited for conventional hydropower. As the velocity of the supercritical flow is very high and the depth is very shallow, hydrokinetic turbines will not be considered for this design.

The chute following the structure could be replaced with a penstock to deliver pressurized flow to a conventional hydro plant and also capture the energy that would be dissipated in the baffle blocks located after the drop. Two conventional hydro designs will be considered.

8.1 Option 1

The first option for the “Cemetery” Check is to run a penstock the full length of the chute and drop, as seen in Figure 8.1. This design would most likely run along side the original structure as to keep civil work costs low.

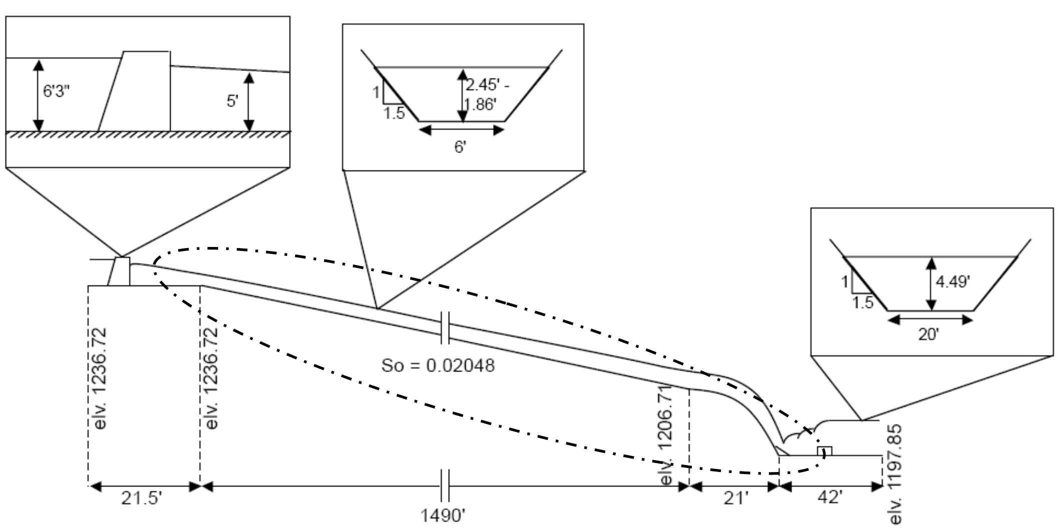

Figure 8.1: Option 1 for the “Cemetery” Check

The flow rate for this check is 316 cfs (8.94 m^3/s), with a designed head of 40.97 ft (12.49 m), including both the chute and drop. Friction losses would be about 11.5 ft for a 5 inch diameter steel penstock (see Appendix S). Based on Figure 6.11, either a Francis or Kaplan turbine could be the selection for this site considering its characteristics. Using the modified equation for potential power generation, it is found that over 800 kW of power could be generated using conventional hydropower (see Appendix S). With the average cost per installed kilowatt described in Appendix M, the total installation cost of this structure would be over $4 million.

8.2 Option 2

The second design option for the "Cemetery" Check, would be to capture the potential energy of the flow in the drop located at the end of the Portland cement concrete chute (see Figure 8.2).

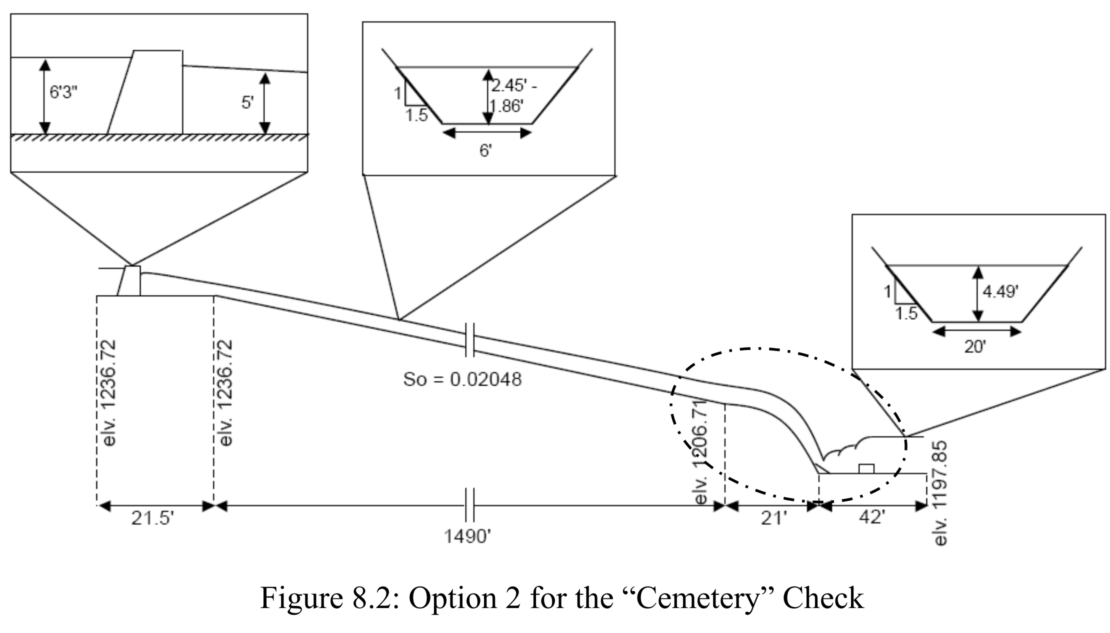

Figure 8.2: Option 2 for the "Cemetery" Check

The head for this design is considerably less, with only 8.86 ft (2.7 m) of drop while still having a flow rate of 316 cfs (8.94 m^3/s). Using Figure 6.11 it is determined that a Kaplan turbine would be the best choice. With this design, an expected 200 kW of power could be produced using a conventional hydropower design, and would cost over $1 million. This design may be more complicated, because of requirement for transition of the supercritical flow of the chute into the inlet of the turbine. The formal structure would need to be designed carefully to account for this. Because of the special attention that will be required for this design, the actual cost of this structure could be more expensive than the preliminary estimate of $5000/kw.

8.3 Preferred Option

Between the two options, the first appears preferable not only because of its higher power potential, but also because of its design. Capturing the flow while it is still in a subcritical state keeps the design simple as compared to channeling a supercritical flow into a turbine, which may provide a challenge. Cost per kilowatt might be less with Option 1 because of the considerably larger amount of power generated. However, Option 2 would require considerably less civil works.

9. Conclusions

For all of the sites presented in this study, it appears that conventional hydropower is not only somewhat cheaper per kilowatt but can also produce considerably more power than hydrokinetic turbines. However, the capital outlay for the conventional hydro systems will be much larger, because of their greater power size. The limitations on hydrokinetic power are a result of channel design and the flow relation to its critical point. Although a narrowed channel can increase the kinetic energy of a flow, it also increases the critical energy value, leaving little available energy for generation. A traditional hydropower is more effective for capturing a large fraction of the available water power. It is possible that the hydrokinetic availability of the system could be increased through a radical restructuring of the channels to "smooth out" the drops, but the cost and scope of such a project is beyond the aims of this study.

The hydrokinetic design recommended for Check 2 is a 25 kW rated turbine in a 13 ft wide channel. In this configuration, 18 kW of electrical power is generated at a unit capital cost of nearly $7,000 per kilowatt. A conventional hydropower turbine at the same site has a much greater power potential, and could generate over 700 kW at about $5,000 per installed kilowatt. Such a system would be designed to replace the current check structure and the down channel baffle blocks.

Check 5 is similar in its capability with respect to both hydrokinetic and conventional hydropower, but has a larger difference in its cost per kilowatt. The best hydrokinetic case for this site generates 12 kW and is almost $10,000 per kilowatt. Almost 300 kW of power could be generated using conventional hydropower and cost less per kilowatt. Consequently, a conventional hydropower design appears to be better suited for this site. The energy lost in the hydraulic jump and to friction account for less than 200 kW. To incorporate any form of hydropower, this amount would be the maximum amount that could be removed to maintain downstream conditions.

The “Cemetery” Check was unsuitable for hydrokinetic turbines because of the supercritical flow. Two different conventional hydropower designs were considered. The first option which spanned the concrete chute and the 8 ft drop could potentially generate over 800 kW of power. The other design which only used the 8 ft drop at the end of the chute could generate approximately 200 kW. The amount of power dissipated at the baffle blocks following this drop is 300 kW. Option 2 would allow for 200 kW of power to be generated, but require some modifications to the existing down channel baffle-hydraulic jump system.

Future research on these sites should include evaluating various turbine placement designs, as only rows of turbines were evaluated in this study. Since this study is limited to evaluating the sites at their design flow rates, in the future seasonal flows should be taken into consideration as the design flow rate is rarely reached. Further economic analyses should also be completed to include feed-in tariffs, operating costs, and year round power production.

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Appendices

Appendix A: List of Nomenclature and Notations

Δx – Distance between Station 1 and Station 2
ε – Ratio of the cross-sectional area of a turbine to the channel
η – Efficiency
ρ – Density (slugs/ft^3 or kg/m^3)
ν – Viscosity (ft^2/s or m^2/s)
A – Cross-sectional area of turbine (ft^2 or m^2)
A_o – Initial cross-sectional area of channel (ft^2 or m^2)
A_3 – Cross-sectional area of the channel inside of the stream tube (ft^2 or m^2)
A_4 – Cross-sectional area of the channel outside of the stream tube (ft^2 or m^2)
A_c – Cross-sectional area of channel (ft^2 or m^2)
b – Base width (ft or m)
b_o – Initial width (ft or m)
b_c – Critical width (ft or m)
block h – Height of baffle blocks (ft or m)
C_c – Coefficient of contraction
Cd – Coefficient of baffle blocks
DOE – Department of Energy
E – Energy (ft or m)
E_o – Initial energy (ft or m)
E_c – Critical energy (ft or m)
EL29 – East Lateral 29
EL68 – East Lateral 68
Fr – Froude number
g – Gravity (ft/s^2 or m/s^2)
H – Head pressure (ft or m)
H_L – Head loss (ft or m)
$H_{L,\ Lined}$ – Head loss of unlined channel (ft or m)
$H_{L,\ Unlined}$ – Head loss of unlined channel (ft or m)
KE – Kinetic power (kW)
L – Length of section for standard step method (ft or m)
m – Side slope of trapezoidal channel
n – Manning's coefficient
P – Power extracted by the turbine (kW)
P_{Blocks} – Power across baffle blocks (kW)
P_{Jump} – Power across hydraulic jump (kW)
P_{Lined} – Power in a lined channel (kW)
P_{ref} – Reference power (kW)
$P_{Unlined}$ – Power in a unlined channel (kW)
P_w – Wetted parameter (ft or m)
PE – Potential power (kW)
PUD – Public Utility District
Q – Flow rate (cfs or m^3/s)

Re – Reynolds number
R_h – Hydraulic radius (ft or m)
s or S_o – Bottom slope of channel
S_f – Slope of water surface
$S_{f, Average}$ – Average slope of water surface
SCADA – Supervisory Control and Data Acquisition
u_o – Velocity of the channel (ft/s or m/s)
u_1 – Velocity of the flow through the turbine (ft/s or m/s)
u_3 – Velocity of the flow in the stream tube following the turbine (ft/s or m/s)
u_4 – Velocity of the flow around the turbine (ft/s or m/s)
v – Velocity (ft/s or m/s)
x – Distance along channel (ft or m)
y – Water depth (ft or m)
y_o – Initial depth (ft or m)
y_c – Critical depth (ft or m)
$y_{downstream}$ – Depth downstream of the gate (ft or m)
y_g – Gate depth (ft or m)
$y_{upstream}$ – Depth upstream of the gate (ft or m)
z – Elevation (ft or m)

Appendix B: EnCurrent Turbine Specifications

Installation costs for the EnCurrent turbine, as further explained in Appendix M, includes the cost of the turbine and the base plate. The pontoon boat is not necessary to include as the channels in the irrigation system are dry from October to April, during which the installation and maintenance can be completed.

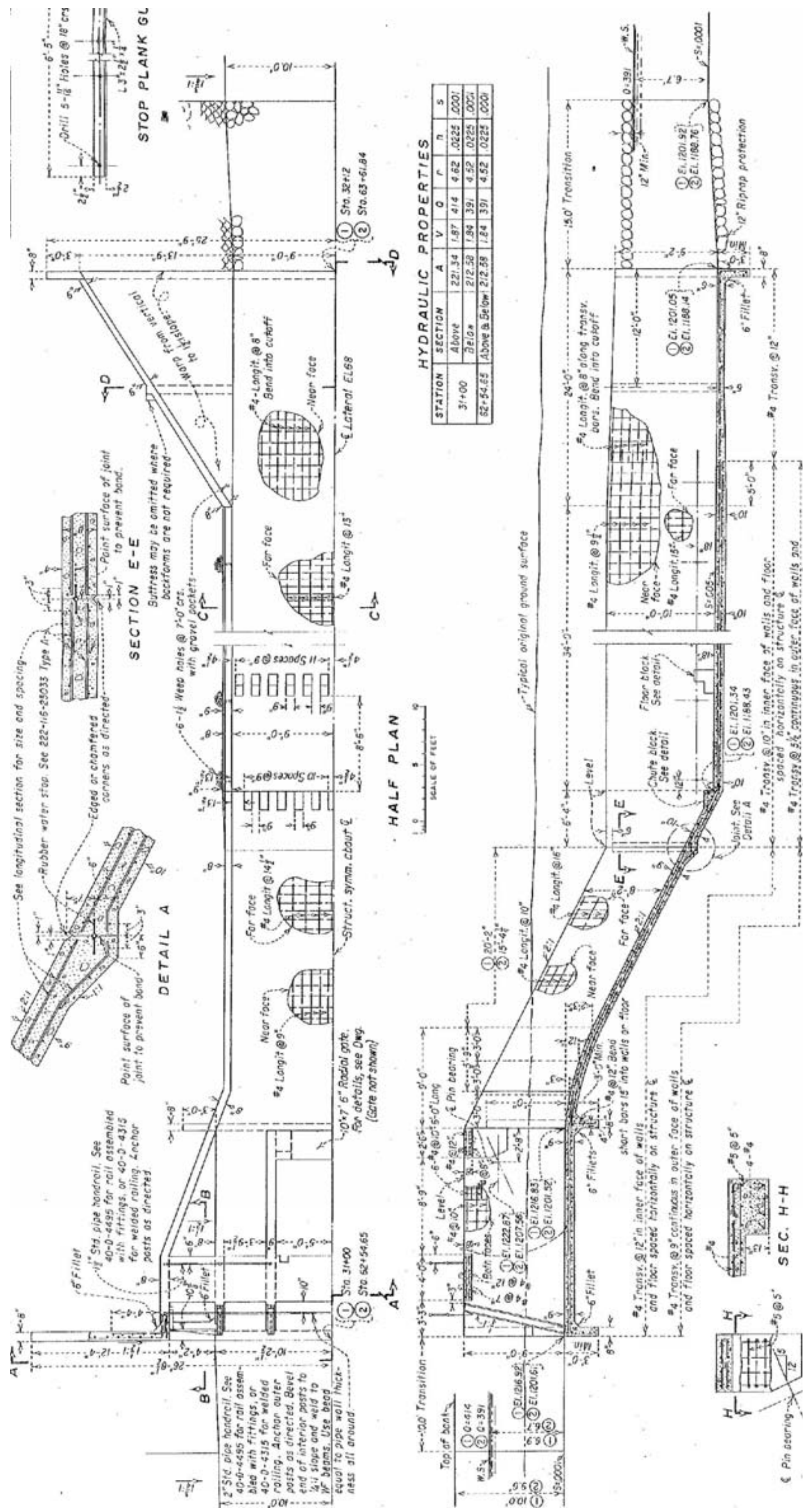
		Installation Cost		
Turbine Size	Model Name	Turbine Price	Base Plate	Pontoon Boat
5 kW	ENC-005-F4	$25,000	$3,000	$16,000
10 kW	ENC-010-F4	$50,000	$3,000	$21,000
25 kW	ENC-025-F4	$110,000	$4,500	$39,000
125 kW	ENC-125-F4	$300,000	TBD	TBD
250 kW	ENC-250-F4	$600,000	TBD	TBD

5 kW Rated EnCurrent Turbine		
Characteristic	ENC-005-F4	ENC-005-R5
Maximum Power Output	5 kW	5 kW
Water Velocity at Max Power	3 m/s	3 m/s
Rotor speed at Max Power	90 RPM	74 RPM
Overall System Mass	340 kg	360 kg
Overall System Height	2.25 m	2.25 m
Rotor Diameter	1.52 m	1.52 m
Rotor Height	0.76 m	0.76 m
Number of Blades	4	5
Distance from top of rotor to:		
Center of Bottom Bearing	0.467 m	0.467 m
Mounting Surface	0.654 m	0.654 m
Gearbox Ratio	13.5:1	13.5:1
Generator Output	0—198 V	0—165 V

10 kW Rated EnCurrent Turbine		
Characteristic	ENC-010-F4	ENC-010-R5
Maximum Power Output	10 kW	10 kW
Water Velocity at Max Power	3 m/s	3 m/s
Rotor speed at Max Power	90 RPM	74 RPM
Overall System Mass	640 kg	670 kg
Overall System Height	3.14 m	3.14 m
Rotor Diameter	1.52 m	1.52 m
Rotor Height	1.52 m	1.52 m
Number of Blades	4	5
Distance from top of rotor to:		
Center of Bottom Bearing	0.467 m	0.467 m
Mounting Surface	0.751 m	0.751 m
Gearbox Ratio	19.85:1	23.97:1
Generator Output	0—287 V	0—285 V

25 kW Rate EnCurrent Turbine		
Characteristic	ENC-025-F4	ENC-025-R5
Maximum Power Output	25 kW	25 kW
Water Velocity at Max Power	3 m/s	3 m/s
Rotor speed at Max Power	40 RPM	33 RPM
Overall System Mass	2200 kg	2350 kg
Overall System Height	4.08 m	4.08 m
Rotor Diameter	3.40 m	3.40 m
Rotor Height	1.70 m	1.70 m
Number of Blades	4	5
Distance from top of rotor to:		
Center of Bottom Bearing	0.467 m	0.467 m
Mounting Surface	1.056 m	1.056 m
Gearbox Ratio	61.3:1	61.3:1
Generator Output	0—390 V	0—321 V

Appendix C: East Low Lateral 68 – Check 2, "As Built" Plans



Appendix D: Energy Losses and Available Power Calculations for Check 2

<u>Constants for Check 2</u>

Constants				
	English		SI	
Q =	414	cfs	11.72	m^3/s
g =	32.20	ft/s^2	9.8	m/s^2
ρ =	1.94	$slugs/ft^3$	1000	kg/m^3
γ =	62.4	lb/ft^3	1000	kg/m^3
ν =	1.66E-05	ft^2/s	2.00E-6	m^2/s

<u>Loss due to Friction</u>
The amount of energy lost to friction by lining the channel upstream of Check 2 be earthen vs. Portland cement concrete is determined by using a Manning's coefficient of 0.02 and 0.013, respectively. A Standard Step Method is applied in which the Manning's coefficient is taken into account (see Appendix I) and the resulting depth and energy of the channel are determined. Using a power/energy relation the amount of power lost to friction for both cases is determined. If the channel were to be lined, the difference in the power lost to friction would be the amount of power that could be generated without altering the downstream effects.

Earthen-Lined Channel:

Conditions at the downstream end of the channel preceding Check 2

Upstream (English)		
b =	20	ft
y =	6.90	ft
m =	1.5	
s =	0.0001	
n =	0.02	
Re =	5.54E+05	Turbulent
Fr =	0.15	Subcritical
E =	6.96	ft

Evaluation of water depths using the Standard Step Method starting at Check 2 and proceeding upstream (details of calculations are found in Appendix J)

Station	x (ft)	z (ft)	y (ft)	A_c (ft^2)	v (ft/s)	Pw (ft)	Rh (ft)	S_f	E (ft)
2. Chk 2	0	1216.9	6.90	209.42	1.98	44.88	4.67	9E-5	6.96
1. Chk 1	-2900	1217.2	6.87	208.33	1.99	44.78	4.65	9E-5	6.89

$H_{L,\,Unlined} = E_1 + z_1 - E_2 - z_2 = 6.89 + 1217.21 - 6.96 - 1216.92 = 0.22$ ft

$P_{Unlined} = 0.085\ H_{L,\,Unlined}\ Q = 0.085\ (0.22\ \text{ft})\ (414\ \text{cfs}) = \mathbf{7.74\ kW}$

Portland Cement Concrete-Lined Channel:

Conditions at the downstream end of the channel preceding Check 2

Upstream (English)		
b =	20	ft
y =	6.90	ft
m =	1.5	
s =	0.0001	
n =	0.013	
Re =	5.54E+05	Turbulent
Fr =	0.15	Subcritical
E =	6.96	ft

Evaluation of water depths using the Standard Step Method starting at Check 2 and proceeding upstream (details of the Standard Step Method are seen in Appendix I)

Station	x (ft)	z (ft)	y (ft)	A_c (ft^2)	v (ft/s)	Pw (ft)	Rh (ft)	S_f	E (ft)
2. Chk 2	0	1216.92	6.90	209.42	1.98	44.88	4.67	4E-5	6.96
1. Chk 1	-2900	1217.21	6.72	202.21	2.05	44.24	4.57	4E-5	6.78

$H_{L,\,Lined} = E_1 + z_1 - E_2 - z_2 = 6.78 + 1217.21 - 6.96 - 1216.92 = 0.11$ ft

$P_{Lined} = 0.085\ H_{L,\,Lined}\ Q = 0.085\ (0.11\ \text{ft})\ (414\ \text{cfs}) = \mathbf{3.87\ kW}$

Difference in power loss by lining the channel with Portland cement concrete:

$P_{Unlined} - P_{Lined} = 7.74\ \text{kW} - 3.87\ \text{kW} = \boxed{\mathbf{3.87\ kW}}$

Loss due to Baffle Blocks and Hydraulic Jump

To determine the loss of power due to the baffle blocks and hydraulic jump, the characteristics upstream must be determined. Starting before the gate, the channel has the following characteristics:

Upstream of Gate (English)		
$b =$	20	ft
$y =$	6.90	ft
$m =$	1.5	
$A_c =$	209.42	ft^2
$v =$	1.98	ft/s
$Fr =$	0.15	Subcritical
$E =$	6.96	ft

The next step is to determine the supercritical depth following the gate ($y_{upstream}$) by solving an energy balance equation.

$$y_{upstream} + v_{upstream}^2 / (2g) = y_{downstream} + v_{downstream}^2 / (2g)$$

Using $y_{usptream}$, the gate opening (y_g) can then be determined via the following relation, where the coefficient of contraction (C_c) is assumed to be 0.7 for a radial gate:

$$y_g = y_{upstream}/C_c$$

Using the above equations, the depth following the gate is found to be approximately 2.4 ft, when the gate opening is 3.5 ft. The gate is quickly followed by a drop, with the supercritical flow approaching the normal depth. Using an energy balance equation that includes the difference in elevation, and assuming there is no energy loss on the spillway face, the depth at the bottom of the drop is found to be 0.61 ft. The next step is to determine the effects of the baffle blocks and the resulting hydraulic jump. A 1D conservation of momentum equation is used to determine the depth downstream of the hydraulic jump. This equation is as follows:

$$½ \gamma y_1 – ½ C_d h \rho v_1^2 -½ \gamma y_2 = (Q/b) \rho v_2 - (Q/b) \rho v_1$$

For this equation, all coefficients with a subscript of '1' are the conditions before of the hydraulic jump and baffle blocks while those with a subscript of '2' are those following the jump and blocks. The baffle blocks are assumed to have an average coefficient of drag (C_d) of 0.3 and a height (h) of 1.67 ft. Using this information and solving for y_2, it is found that the depth downstream of the hydraulic jump is just over 5 ft. The conditions before and after the hydraulic jump and baffle blocks are displayed in the following tables:

1. Before the Hydraulic Jump and Baffle Blocks		
z =	1201.34	ft
b =	18	ft
y =	0.61	ft
Re =	1.29E+06	Turbulent
Fr =	8.5	Supercritical
E =	22.54	ft

2. Following the Baffle Blocks and Hydraulic Jump		
z =	1201.34	ft
b =	18	ft
y =	5.06	ft
Re =	8.85E+05	Turbulent
Fr =	0.35	Subcritical
E =	5.38	ft

To determine the amount of power dissipated due to the baffle blocks and hydraulic jump, an energy balance is evaluated across the jump to determine the head loss in terms of ft and is then converted to kilowatts. This value is the amount of power that is lost in the hydraulic jump caused by the baffle blocks.

$$H_L = E_1 + z_1 - E_2 - z_2 = 22.54 + 1201.34 - 5.38 - 1201.34 = 17.16 \text{ ft}$$

$$P_{Blocks} = 0.085\, H_L\, Q = 0.085\,(17.16 \text{ ft})(414 \text{ cfs}) = \boxed{\mathbf{619.344\ kW}}$$

<u>Potential and Kinetic Resource</u>

To determine the potential and kinetic power present in the flow, Equation 2.1and Equation 3.1 will be used.

Kinetic:

$\rho = 1000 \text{ kg/m}^3$

$A = 19.41 \text{ m}^2$

$v = 0.604 \text{ m/s}$

$$KE = \tfrac{1}{2} \rho A v^3\, 10^{-3} = \tfrac{1}{2}(1000 \text{ kg/m}^3)(19.41 \text{ m}^2)(0.604 \text{ m/s})^3(10^{-3}) = \boxed{\mathbf{2.14\ kW}}$$

Potential:

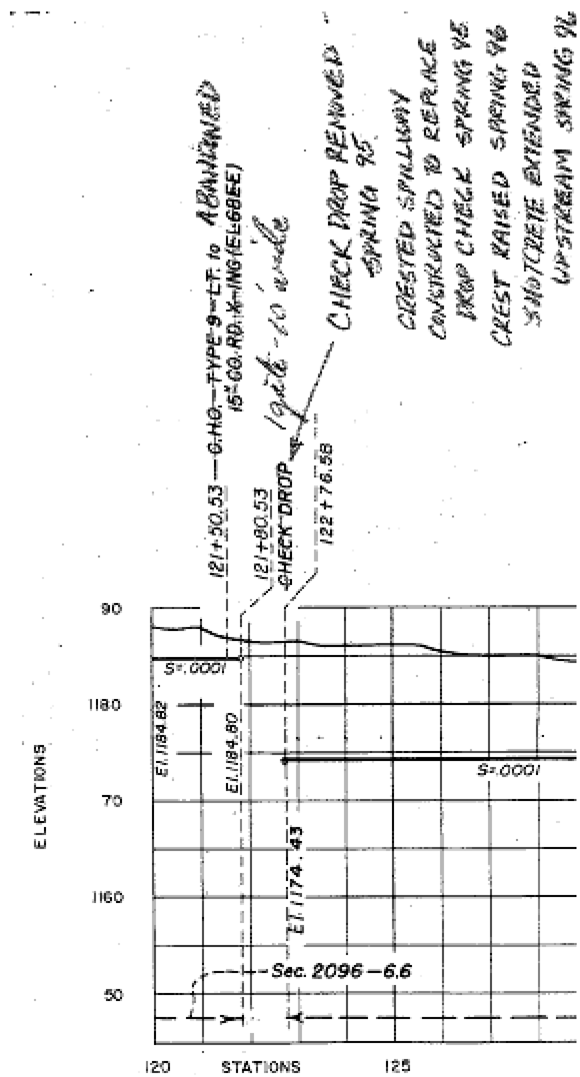
$\rho = 1000 \text{ kg/m}^3$

$Q = 414 \text{ cfs} = 11.72 \text{ m}^3\text{/s}$

$H = 22.4 \text{ ft} = 6.83 \text{ m}$

$$PE = \rho Q g H\, 10^{-3} = (1000 \text{ kg/m}^3)(11.72 \text{ m}^3\text{/s})(9.81 \text{ m/s}^2)(6.83 \text{ m})(10^{-3}) = \boxed{\mathbf{784.5\ kW}}$$

Appendix E: East Low Lateral 68 Check 5, "As Built" Plans



Appendix F: Energy Losses and Available Power Calculations for Check 5

<u>Constants for Check 5</u>

Constants				
	English		SI	
$Q =$	379	cfs	10.73	m^3/s
$g =$	32.20	ft/s^2	9.8	m/s^2
$\rho =$	1.94	$slugs/ft^3$	1000	kg/m^3
$\gamma =$	62.4	lb/ft^3	1000	kg/m^3
$\nu =$	1.66E-05	ft^2/s	2.00E-6	m^2/s

<u>Loss due to Friction</u>
To determine the amount of power lost to for the channel upstream of the washed-out Check 5 structure, the same method as Appendix D will be used.

Earthen-Lined Channel:

Conditions at the upstream end of the channel preceding Check 5

Upstream (English)		
$b =$	20	ft
$y =$	4.58	ft
$m =$	1.5	
$s =$	0.0001	
$n =$	0.02	
$Re =$	3.98E+06	Turbulent
$Fr =$	0.28	Subcritical
$E =$	4.72	ft

Evaluation of water depths using the Standard Step Method starting at the previous check (Check 4) and going downstream (details of calculation are found in Appendix J)

Station	x (ft)	z (ft)	y (ft)	A_c (ft^2)	v (ft/s)	Pw (ft)	Rh (ft)	S_f	E (ft)
1. Chk 4	0	1185.01	4.58	123.15	3.08	36.52	3.37	1E-4	4.72
2. Chk 5	2100	1184.80	5.37	150.55	2.52	39.35	3.83	2E-4	5.47

$H_{L,\ Unlined} = E_2 + z_2 - E_1 - z_1 = 5.47 + 1184.80 - 4.72 - 1185.01 = 0.54$ ft

$P_{Unlined} = 0.085\ H_{L,\ Unlined}\ Q = 0.085\ (0.54\ ft)\ (379\ cfs) =$ **17.40 kW**

Portland Cement Concrete-Lined Channel:

Conditions at the upstream end of the channel preceding Check 5

Upstream (English)		
b =	20	ft
y =	4.58	ft
m =	1.5	
s =	0.0001	
n =	0.013	
Re =	3.98E+06	Turbulent
Fr =	0.28	Subcritical
E =	4.73	ft

Evaluation of water depths using the Standard Step Method starting at the previous check (Check 4) and going downstream (details of the Standard Step Method are seen in Appendix I)

Station	x (ft)	z (ft)	y (ft)	A_c (ft^2)	v (ft/s)	Pw (ft)	Rh (ft)	S_f	E (ft)
1. Chk 4	0	1185.01	4.58	123.15	3.08	36.52	3.37	1E-4	4.72
2. Chk 5	2100	1184.80	5.07	140.11	2.71	38.30	3.66	1E-4	5.18

$H_{L,\,Lined} = E_1 + z_1 - E_2 - z_2 = 5.18 + 1184.80 - 4.72 - 1185.01 = 0.25$ ft

$P_{Lined} = 0.085\, H_{L,\,Lined}\, Q = 0.085$ (0.25 ft)(379 cfs) = **8.05 kW**

Difference in energy loss by lining the channel with Portland cement concrete (in terms of power):

$P_{Unlined} - P_{Lined} = 17.40 \text{ kW} - 8.05 \text{ kW} = \boxed{\textbf{9.35 kW}}$

Loss due to Hydraulic Jump

To determine the amount of energy lost across the hydraulic jump, evaluations of the upstream conditions are first made. The characteristics are then determined using a 1D conservation of momentum equation, as discussed in Appendix C. The results of this equation are seen in the following tables:

1. Before the Hydraulic Jump		
z =	1174.43	ft
b =	18	ft
y =	0.80	ft
Re =	1.09E+06	Turbulent
Fr =	5.19	Supercritical
E =	10.36	ft

2. Following the Hydraulic Jump		
z =	1174.43	ft
b =	18	ft
y =	5.44	ft
Re =	1.17E+06	Turbulent
Fr =	0.29	Subcritical
E =	5.55	ft

$H_L = E_1 + z_1 - E_2 - z_2 = 10.36 + 1174.43 - 5.55 - 1174.43 = 4.81$ ft

$P_{Jump} = 0.085\ H_L\ Q = 0.085\ (4.81\ \text{ft})(379\ \text{cfs}) = \boxed{\mathbf{154.95\ kW}}$

Potential and Kinetic Resource

To determine the potential and kinetic power present in the flow, Equation 2.1and Equation 3.1 are used.

Kinetic

$\rho = 1000\ \text{kg/m}^3$

$A = 12.68\ \text{m}^2$

$v = 0.84\ \text{m/s}$

$KE = ½\ \rho\ A\ v^3 10^{-3} = ½\ (1000\ \text{kg/m}^3)(12.68\ \text{m}^2)(0.84\ \text{m/s})^3(10^{-3}) =$

$\boxed{\mathbf{3.76\ kW}}$

Potential

$\rho = 1000\ \text{kg/m}^3$

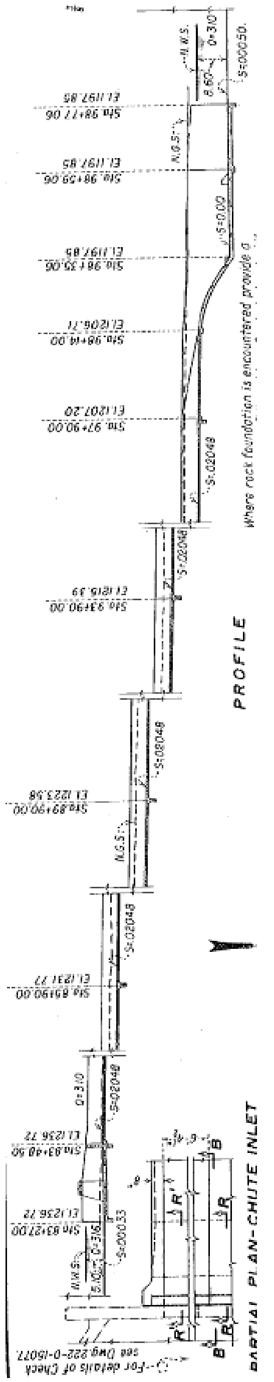
$Q = 379\ \text{cfs} = 10.73\ \text{m}^3/\text{s}$

$H = 10.6\ \text{ft} = 3.23\ \text{m}$

$PE = \rho\ Q\ g\ H\ 10^{-3} = (1000\ \text{kg/m3})(10.73\ \text{m}^3/\text{s})(9.8\ \text{m/s2})(3.23\ \text{m})(10^{-3})=$

$\boxed{\mathbf{339.6\ kW}}$

Appendix G: East Low Lateral 29 “Cemetery” Check, “As Built” Plans



Appendix H: Energy Losses and Available Power Calculations for Cemetery Check

<u>Constants for "Cemetery" Check</u>

Constants				
	English		SI	
Q =	316	cfs	8.95	m^3/s
g =	32.20	ft/s^2	9.8	m/s^2
ρ =	1.94	$slugs/ft^3$	1000	kg/m^3
ν =	1.66E-05	ft^2/s	2.00E-6	m^2/s

<u>Loss due to Hydraulic Jump</u>
To determine the amount of energy lost across the hydraulic jump, a similar method to the Appendix D is used. The height of the baffle blocks is determined to be 1.67 ft with a coefficient of drag of 0.3.

1. Before the Hydraulic Jump and Baffle Blocks		
z =	1197.85	ft
b =	6	ft
y =	1.37	ft
Re =	1.92E+06	Turbulent
Fr =	5.78	Supercritical
E =	16.55	ft

2. Following the Baffle Blocks and Hydraulic Jump		
z =	1197.85	ft
b =	12	ft
y =	4.49	ft
Re =	9.05E+05	Turbulent
Fr =	0.49	Subcritical
E =	5.02	ft

$$H_L = E_1 + z_1 - E_2 - z_2 = 16.55 + 1197.85 - 5.02 - 1197.85 = 11.53 \text{ ft}$$

$$P_{Jump} = 0.085\, H_L\, Q = 0.085\,(11.53 \text{ ft})(316 \text{ cfs}) = \boxed{\textbf{309.70 kW}}$$

<u>Potential and Kinetic Power Resource</u>
To determine the potential and kinetic power present in the flow, Equation 2.1and Equation 3.1 are used.

Kinetic

$A = 1.9\ m^2$

$v = 4.05\ m/s$

$$KE = \tfrac{1}{2}\,\rho\, A\, v^3\, 10^{-3} = \tfrac{1}{2}\,(1000\ kg/m^3)\,(1.9\ m^2)\,(4.05\ m/s)^3(10^{-3}) = \boxed{\textbf{63.1 kW}}$$

Potential

$Q = 316$ cfs $= 8.95\ m^3/s$

$H = 43.9$ ft $= 13.4$ m

$PE = \rho\ Q\ g\ H\ 10^{-3} = (1000\ kg/m^3)\ (8.95\ m^3/s)\ (9.8\ m/s^2)\ (13.4\ m)\ (10^{-3}) =$

$\boxed{\textbf{1175.1 kW}}$

Appendix I: Standard Step Method

This section follows lecture notes as given by Prof. Stephen Burges for CEE 477 taught Spring Quarter of 2008 at the University of Washington [30].

- Used to calculate depth (or water surface = stage) at a given station (specified x – location)
- Can be used for prismatic channels
- Must be used for non-prismatic channels

General Form:

$$S_o \, \Delta x + E_1 = S_{f,\,Average} \, \Delta x + E_2$$

<u>Knowns:</u>

S_o – Slope of the channel

Δx – Distance between Station 1 and Station 2

E_1 – Energy at Station 1

<u>Unknowns:</u>

$S_{f,\,Average}$ – Average slope of water surface based on unknown Station

E_2 – Energy at Station 2 $[y_2+v_2^2/(2\,g)]$

Equations:

- $E = y + v^2/(2\,g)$
- $S_f = v^2\, n^2/R_H^{4/3}$ (SI) **or** $v^2\, n^2/(2.22\, R_H^{4/3})$ (English)
- $H_L = \Delta x\, S_{f,\,Average}$

Procedure:

Calculate E_1 and S_{f1} at Station 1

⇩

Guess depth at Station 2 (y_2)

⇩

Calculate

$S_{f,\,Average} = ½\,(S_{f,1} + S_{f,2})$

Eq. 1: $H_1 - \Delta x\, S_{f,\,Average}$

Eq. 2: $E_2 + z_2$

⇩

Do Eq. 1 and Eq. 2 agree?

⇩ Yes ⇩ No

Yes ⇩ Assumed y_2 is correct

Appendix J: Initial Depths for Channels Using Standard Step Method

Check 2:

Constants	
Q =414	cfs
g =32.20	ft/s^2
ρ =1.94	$slugs/ft^3$
ν =1.66E-05	ft^2/s

Bottom of the Channel		
z =	1216.92	ft
b =	20	ft
y =	6.90	ft
m =	1.5	
n =	0.02	
A =	209.42	ft^2
v =	1.98	ft/s
Re =	5.54E+05	Turbulent
Fr =	0.15	Subcritical
E =	6.96	ft

Δx = -290

x	z	y	A_c	v	E + z	Pw	Rh	S_f	$S_{f,ave}$* Δx	E + z	Δ(E+z)
0	1216.92	6.90	209.42	1.98	1223.88	44.88	4.67	0.00009	-	-	0
-290	1216.95	6.90	209.30	1.98	1223.91	44.87	4.66	0.00009	-0.026	1223.9	0
-580	1216.98	6.89	209.18	1.98	1223.93	44.86	4.66	0.00009	-0.026	1223.9	0
-870	1217.01	6.89	209.07	1.98	1223.96	44.85	4.66	0.00009	-0.026	1223.9	0
-1160	1217.04	6.89	208.96	1.98	1223.99	44.84	4.66	0.00009	-0.026	1223.9	0
-1450	1217.07	6.89	208.85	1.98	1224.01	44.83	4.66	0.00009	-0.026	1224.0	0
-1740	1217.09	6.88	208.74	1.98	1224.04	44.82	4.66	0.00009	-0.026	1224.0	0
-2030	1217.12	6.88	208.64	1.98	1224.06	44.81	4.66	0.00009	-0.026	1224.0	0
-2320	1217.15	6.88	208.53	1.99	1224.09	44.80	4.65	0.00009	-0.026	1224.0	0
-2610	1217.18	6.88	208.43	1.99	1224.12	44.79	4.65	0.00009	-0.026	1224.1	0
-2900	1217.21	6.87	208.33	1.99	1224.14	44.78	4.65	0.00009	-0.026	1224.1	0

Check 5:

Constants	
Q =379	cfs
g =32.20	ft/s^2
ρ =1.94	$slugs/ft^3$
ν =1.66E-05	ft^2/s

Top of Channel		
z =	1185.01	ft
b =	20	ft
y =	4.58	ft
m =	1.50	
s =	0.0001	
n =	0.02	
A =	123.15	ft^2
V =	3.08	ft/s
Re =	1.21E+06	Turbulent
Fr =	0.28	Subcritical
E =	4.73	ft

Δx = 210

x	z	y	A_c	v	E	E + z	Pw	Rh	S_f	$S_{f,ave}$* Δx	E + z	Δ(E+z)
0	1185.01	4.58	123.15	3.08	4.73	1189.74	36.52	3.37	0.0003	-	-	-
210	1184.99	4.68	126.43	3.00	4.82	1189.81	36.87	3.43	0.0003	-0.068	1189.81	0
420	1184.97	4.77	129.55	2.93	4.90	1189.87	37.20	3.48	0.0002	-0.063	1189.87	0
630	1184.95	4.86	132.52	2.86	4.98	1189.93	37.51	3.53	0.0002	-0.059	1189.93	0
840	1184.93	4.94	135.37	2.80	5.06	1189.99	37.81	3.58	0.0002	-0.055	1189.99	0
1050	1184.91	5.02	138.11	2.74	5.13	1190.04	38.09	3.63	0.0002	-0.052	1190.04	0
1260	1184.88	5.09	140.76	2.69	5.21	1190.09	38.36	3.67	0.0002	-0.049	1190.09	0
1470	1184.86	5.16	143.31	2.64	5.27	1190.14	38.62	3.71	0.0002	-0.047	1190.14	0
1680	1184.84	5.23	145.79	2.60	5.34	1190.18	38.87	3.75	0.0002	-0.044	1190.18	0
1890	1184.82	5.30	148.20	2.56	5.40	1190.22	39.12	3.79	0.0002	-0.042	1190.22	0
2100	1184.80	5.37	150.55	2.52	5.47	1190.27	39.35	3.83	0.0001	-0.040	1190.27	0

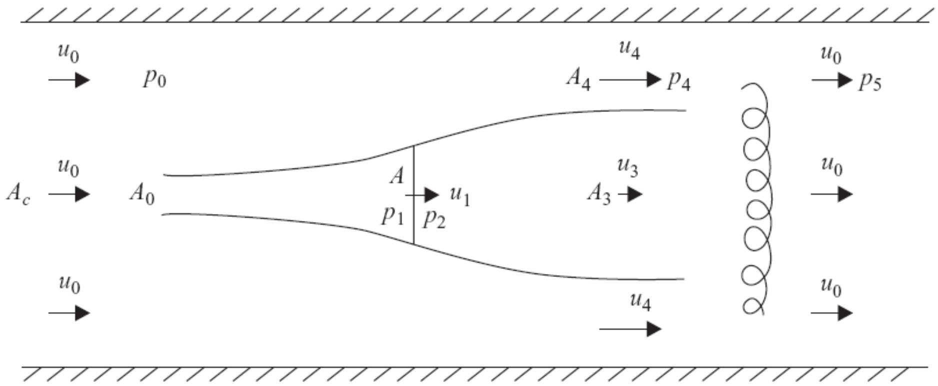
"Cemetery" Check:

Constants	
Q =316	cfs
g =32.20	ft/s^2
ρ =1.94	$slugs/ft^3$
ν =1.66E-05	ft^2/s

Top of Channel		
z =	1236.72	ft
b =	6	ft
y =	2.45	ft
m =	1.50	
s =	0.02048	
n =	0.013	
A =	23.72	ft^2
V =	13.32	ft/s
Re =	1.28E+06	Turbulent
Fr =	1.76	Supercritical
E =	5.21	ft

Δx = 147												
x	z	y	A_c	v	$v^2/2g$	E +z	Pw	Rh	S_f	$S_{f,ave}$* Δx	E + z	Δ(E+z)
0	1236	2.45	23.72	13.32	2.76	5.21	14.84	1.60	0.007	-	-	-
147	1233	2.03	18.41	17.17	4.58	3.61	13.33	1.38	0.015	1.60	3.61	0
293	1230	1.92	17.10	18.48	5.30	1.23	12.94	1.32	0.018	2.38	1.23	0
440	1227	1.88	16.62	19.01	5.61	-1.51	12.79	1.30	0.019	2.73	-1.51	0
586	1224	1.87	16.44	19.23	5.74	-4.40	12.73	1.29	0.020	2.89	-4.40	0
733	1221	1.86	16.36	19.32	5.79	-7.35	12.71	1.29	0.020	2.95	-7.35	0
879	1218	1.86	16.33	19.35	5.82	-10.33	12.70	1.29	0.020	2.98	-10.33	0
1026	1215	1.86	16.31	19.37	5.83	-13.32	12.70	1.29	0.020	2.99	-13.32	0
1172	1212	1.86	16.31	19.38	5.83	-16.32	12.69	1.28	0.020	3.00	-16.32	0
1319	1209	1.86	16.31	19.38	5.83	-19.32	12.69	1.28	0.020	3.00	-19.32	0
1465	1206	1.86	16.30	19.38	5.83	-22.32	12.69	1.28	0.020	3.00	-22.32	0

Appendix K: Garrett and Cummins Theory of Hydrokinetic Turbines in a Channel



Definition sketch of a single turbine in a channel [24]

The assumptions underpinning Garrett and Cummins' theory are valid only when the *Fr* and blockage ratio (ε) are relative low. The blockage ratio is the ratio of the cross-sectional area of the turbine to the cross-sectional area of the channel.

$$\varepsilon = A / A_c$$

Using the continuity equation, conservation of momentum, conservation of energy, and the Bernoulli equation along a streamline, the following equations are derived:

$$u_3 (u_4 - u_o) = \varepsilon\, u_1 (u_4 - u_3)$$

$$u_1 = u_3 (u_4 + u_3) / (u_4 + 2\, u_3 - u_o)$$

Assuming that $u_3 = u_o/3$, the reference power can be calculated using:

$$P_{ref} = A_c\, u_o\, ½\, (u_4 - u_o)\, (u_4 + 2\, u_3 - u_o)$$

The power available for generation can then be calculated with the following relation:

$$P / P_{ref} = u_1 / u_o$$

Appendix L: Hydrokinetic Evaluation for Check 2

Hydrokinetic Power Calculation for Original Channel Design

$\rho = 1000\ kg/m^3$
$y = 2.10\ m$
$b = 6.1\ m$
$A_c = 19.41\ m^2$
$v = 0.604\ m/s$
$\eta = 0.30$
$KE = ½\ \eta\ \rho\ A_c\ v^3\ 10^{-3} = ½\ (0.3)\ (1000\ kg/m^3)\ (19.41\ m^2)\ (0.604\ m/s)^3\ (10^{-3}) =$ **$\boxed{0.64\ kW}$**

Kinetic Power for Various Channel Widths

$Q = 414$ cfs
$\eta = 0.3$
Depths calculated by conservation of energy

$A_c = b * y$
$v = Q / A_c$
$KE = ½\ \eta\ \rho\ A_c\ v^3\ 10^{-3}$ (ρ, A_c, and v must be in SI units)

Channel Width (ft)	Depth (ft)	Area (sq-ft)	Velocity (ft/s)	KE (kW)
20	6.82	136.40	3.04	4.94
19	6.80	129.20	3.20	5.50
18	6.78	122.04	3.39	6.17
17	6.76	114.92	3.60	6.96
16	6.73	107.68	3.84	7.92
15	6.70	100.50	4.12	9.10
14	6.65	93.10	4.45	10.60
13	6.60	85.80	4.83	12.48
12	6.53	78.36	5.28	14.96
11	6.43	70.73	5.85	18.36
10	6.29	62.90	6.58	23.22
9	6.07	54.63	7.58	30.78
8	5.66	45.28	9.14	44.81
7.5	5.23	39.23	10.55	59.71

Calculation of the Critical Width

$Q = 414$ cfs
$y_o = 6.9$ ft
$b_o = 20$ ft
$m = 1.5$

$A_o = y_o (b_o + y_o m) = 209.42$ sq-ft
$v = Q / A_o = 1.98$ ft/s
$E_o = y_o + v_o^2/(2 g) = 6.96$ ft
$y_c = (2/3) E_c = 4.64$ ft
$v_c = \text{sqrt}(g y_c) = 12.23$ ft/s
$b_c = Q / (V_c y_c) = (414 \text{ cfs}) / (12.23 \text{ ft/s} * 4.64 \text{ ft}) = \boxed{\textbf{7.30 ft}}$

Difference in Critical Energy from Channel Energy and Calculation of Power from Available Energy

$Q = 414$ cfs
$E = 6.96$ ft
For a rectangular channel:
$y_c = ((Q/b)^2/g)^{1/3}$
$E_c = (3/2) y_c$
$P = 0.085 H_{L, Unlined} Q$ (English units)

Channel Width (ft)	Critical Energy (ft)	E - Ec (ft)	Total Power (kW)	Thermal Losses (kW)	Power Available for Generation (kW)
20	3.55	3.41	120.00	12.00	108.00
19	3.68	3.28	115.42	11.54	103.88
18	3.81	3.15	110.85	11.08	99.76
17	3.96	3.00	105.57	10.56	95.01
16	4.12	2.84	99.94	9.99	89.95
15	4.31	2.65	93.25	9.33	83.93
14	4.51	2.45	86.22	8.62	77.59
13	4.74	2.22	78.12	7.81	70.31
12	5.00	1.96	68.97	6.90	62.08
11	5.30	1.66	58.42	5.84	52.57
10	5.64	1.32	46.45	4.65	41.81
9	6.05	0.91	32.02	3.20	28.82
8	6.55	0.41	14.43	1.44	12.99
7.5	6.84	0.12	4.22	0.42	3.80

Appendix M: Installation Costs for Hydrokinetic and Conventional Hydropower

Hydrokinetic Turbines

According to Roy D. Dodson [31] the average cost to line a channel with Portland cement concrete is $55 per square meter. Converting this to English units, it is found that the cost is about $4.87 per square foot. For the case of lining a narrowed channel with concrete, the height of the channel would be the maximum calculated height of the water plus a 0.98 ft freeboard. The purpose of a freeboard is to provide a safety for the channel in the case of a surge. The length of the lined portion is dependent on the number of turbines in the water, as multiple turbines need extra space between them so that the wake from one turbine does not impede the operation of another. To do this at least 10 feet between each turbine is left. The total lined area of a channel would be:

Total area to be lined = [(base width) + 2 (depth + 0.98 ft)] x (length)

The following chart gives the estimated cost to line a channel for a given width and length, which is a function of the number of turbines placed in a given channel. The cost used for the hydrokinetic design also includes the cost of the turbine and its installation, which are specified in Appendix B.

Chart of civil work costs depending on channel width and number of turbines

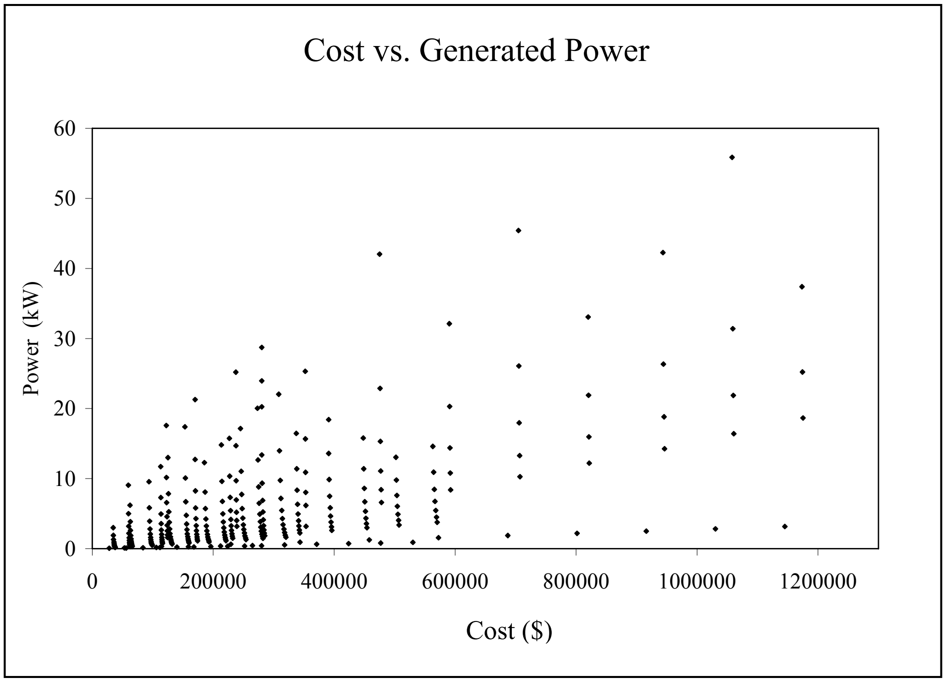
# of Turb-ines	Channel Widths (ft)											
	9	10	11	12	13	14	15	16	17	18	19	20
1	6,937	7,220	7,502	7,785	8,067	8,350	8,632	8,914	9,197	9,479	9,762	10,044
2	6,937	7,220	7,502	7,785	8,067	8,350	8,632	8,914	9,197	9,479	9,762	10,044
3	10,406	10,830	11,253	11,677	12,101	12,524	12,948	13,372	13,795	14,219	14,643	15,066
4	13,874	14,439	15,004	15,569	16,134	16,699	17,264	17,829	18,394	18,959	19,524	20,089
5	13,874	14,439	15,004	15,569	16,134	16,699	17,264	17,829	18,394	18,959	19,524	20,089
6	17,343	18,049	18,755	19,461	20,168	20,874	21,580	22,286	22,992	23,698	24,405	25,111
7	17,343	18,049	18,755	19,461	20,168	20,874	21,580	22,286	22,992	23,698	24,405	25,111
8	20,812	21,659	22,506	23,354	24,201	25,049	25,896	26,743	27,591	28,438	29,285	30,133
9	20,812	21,659	22,506	23,354	24,201	25,049	25,896	26,743	27,591	28,438	29,285	30,133
10	27,749	28,879	30,009	31,138	32,268	33,398	34,528	35,658	36,788	37,917	39,047	40,177

Conventional Hydropower

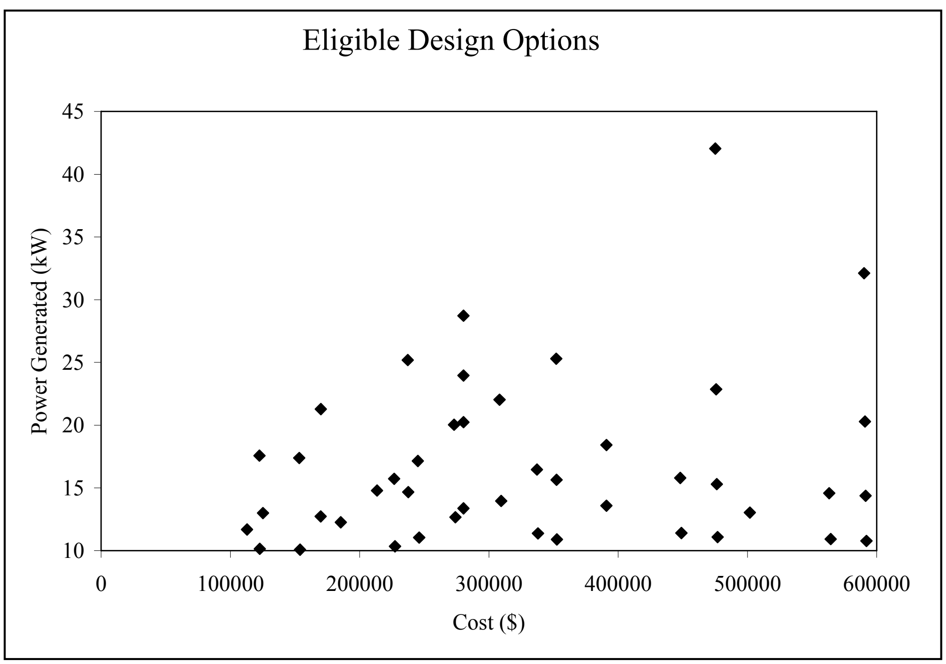
Based on the Wales ECO Centre [32], small scale conventional hydropower having a head of 60 ft or less is found to cost between $4,500/kW and $5,500/kW. In order to estimate the cost of a conventional hydropower system the expected power is calculated assuming an efficiency of 90%, and using this value and the average cost per kilowatt the expected cost of installing the design can be calculated.

Appendix N: All Hydrokinetic Cases for Check 2

<u>All design cases:</u>



<u>Eligible design cases: Cases generating more than 10 kW and costing less than $600,000 to install</u>



List of eligible cases for Check 2

Rank	Turbine Size (kW)	# of Turbines	Channel Width (ft)	Expected Power (kW)	Capital Costs ($US)	Cost per Kilowatt ($US/kW)
1.	25	1	13	18	122,500	6800
2.	10	5	11	21	170,000	8000
3.	5	5	9	17	153,000	9000
4.	25	2	14	25	237,000	9500
5.	5	4	9	13	125,000	9600
6.	10	2	10	12	113,000	9600
7.	10	9	12	29	280,000	9700
8.	25	4	15	42	475,000	11300
9.	10	8	12	24	280,000	11600
10.	25	1	14	10	123,000	12300
12.	10	3	11	13	170,000	13000
13.	5	9	10	20	273,000	13600
14.	10	7	12	20	280,500	14000
15.	25	3	15	25	352,000	14000
16.	5	10	10	22	308,000	14000
17.	5	8	10	17	245,000	14100
18.	10	4	11	16	226,500	14100
19.	5	7	10	15	213,500	14200
20.	5	6	10	12	185,500	15400
22.	5	5	10	10	154,000	15400
23.	25	2	15	15	235,100	15600
24.	25	5	16	32	590,000	18500
25.	10	6	12	16	337,000	21000
26.	25	4	16	23	476,000	21000
27.	10	5	12	13	280,000	21500
28.	10	9	13	18	391,000	21500
29.	5	9	11	13	274,000	21500
30.	10	4	12	10	227,000	22000
31.	5	10	11	14	309,500	22000
32.	5	8	11	11	246,000	23000
33.	25	3	16	16	352,000	23000
34.	10	8	13	16	448,000	28000
35.	10	7	12	14	391,000	28000
36.	25	5	17	20	591,000	28000
37.	10	6	13	11	338,000	30700
38.	25	4	17	15	476,000	31700
39.	25	3	17	11	353,000	32000
40.	10	9	14	13	502,000	38000
41.	10	10	14	15	563,000	38000
42.	10	8	14	11	449,000	40000
43.	25	5	18	14	591,500	42200
44.	25	4	18	11	477,000	43000
45.	10	10	15	11	564,000	51300
46.	25	5	19	11	592,000	53800

Appendix O: Conventional Hydropower Evaluations for Check 2

Power Evaluation

$\eta = 0.9$

$\rho = 1000\ kg/m^3$

$Q = 414\ cfs = 11.72\ m^3/s$

$H = 22.4\ ft = 6.83\ m$

$P = \eta\ \rho\ Q\ g\ H\ 10^{-3} = (0.9)(1000\ kg/m^3)(11.72\ m^3)(9.8\ m/s^2)(6.83\ m)\ (10^{-3}) = \boxed{\textbf{706.02 kW}}$

Cost Evaluation

(706.02 kW) * ($5,000/kW) = $\boxed{\textbf{\$3,530,100}}$

Appendix P: Hydrokinetic Evaluation of Check 5

Hydrokinetic Power Calculation for Original Channel Design

ρ = 1000 kg/m^3
A_c = 12.68 m^2
v = 0.84 m/s
η = 0.30
$KE = \frac{1}{2}\, \eta\, \rho\, A_c\, v^3\, 10^{-3} = \frac{1}{2}\,(0.3)\,(1000 \text{ kg/m}^3)\,(12.68 \text{ m}^2)\,(0.84 \text{ m/s})^3\,(10^{-3}) =$ $\boxed{\textbf{1.13 kW}}$

Kinetic Energy for Various Channel Widths

Q = 379 cfs
η = 0.3
Depths calculated by conservation of energy

$A_c = b * y$
$V = Q / A_c$
$KE = \frac{1}{2}\, \eta\, \rho\, A_c\, v^3$ (ρ, A_c, and v must be in SI units)

Channel Width (ft)	Depth (ft)	Area (sq-ft)	Velocity (ft/s)	KE (kW)
20	4.45	89.00	4.26	8.90
19	4.41	83.79	4.52	10.04
18	4.37	78.66	4.82	11.39
17	4.31	73.27	5.17	13.13
16	4.24	67.84	5.59	15.31
15	4.15	62.25	6.09	18.19
14	4.02	56.28	6.73	22.25
13	3.83	49.79	7.61	28.43
12	3.33	39.96	9.48	44.14

Determine the Critical Width

Q = 379 cfs
y_o = 4.58 ft
b_o = 20 ft
m = 1.5

$A_o = y_o\,(b_o + y_o\, m)$ = 123.07 sq-ft
$V = Q / A_o$ = 3.07 ft/s
$E_o = y_o + V_o^2/(2\, g)$ = 4.72 ft
$y_c = (2/3)\, E_c$ = 3.15 ft

$V_c = \text{sqrt}(g\ y_c) = 10.07$ ft/s
$b_c = Q / (V_c\ y_c) = (379 \text{ cfs}) / (10.07 \text{ ft/s} * 3.15 \text{ ft}) = \boxed{\textbf{11.95 ft}}$

<u>Difference in Critical Energy from Channel Energy and Calculation of Power from Available Energy</u>

Q = 379 cfs
E = 4.72 ft
For a rectangular channel:

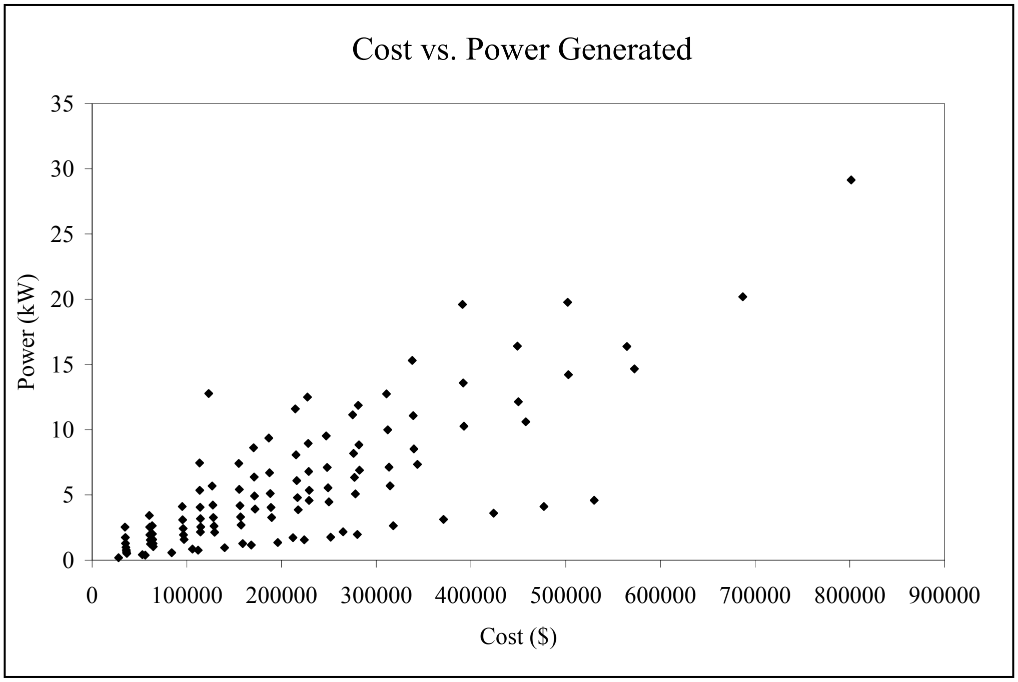
$y_c = ((Q/b)^2/g)^{1/3}$
$E_c = (3/2)\ y_c$

$P = 0.085\ H_{L,\ Unlined}\ Q$ (English units)

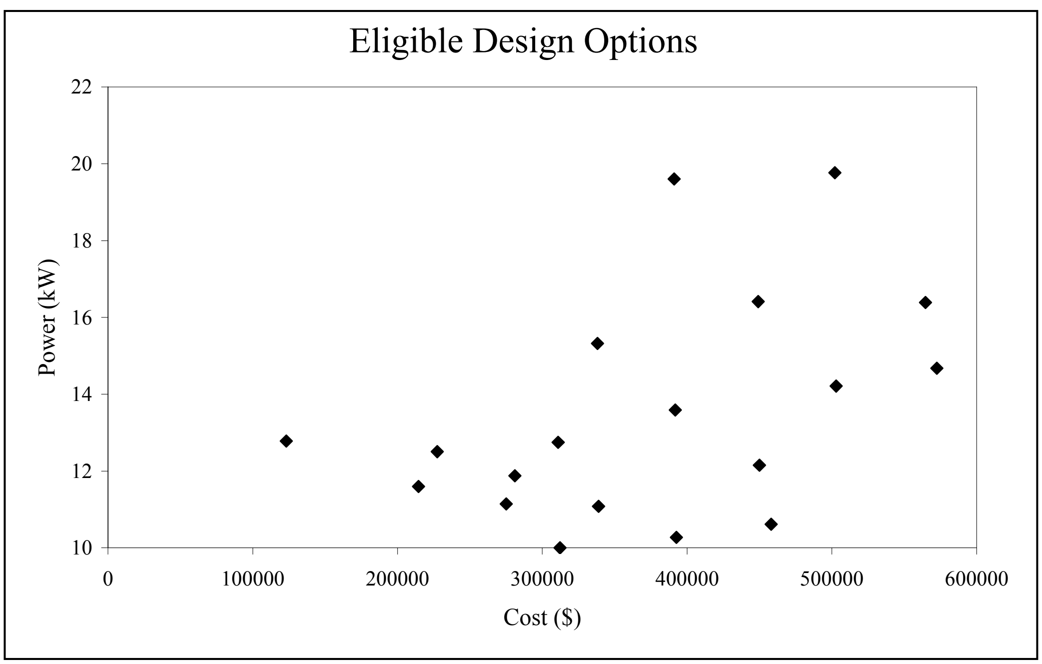
Channel Width (ft)	Critical Energy (ft)	E - Ec (ft)	Power (kW)	Thermal Losses (kW)	Power Generated (kW)
20	3.35	1.37	44.13	4.41	39.72
19	3.47	1.25	40.27	4.03	36.24
18	3.60	1.12	36.08	3.61	32.47
17	3.73	0.99	31.89	3.19	28.70
16	3.89	0.83	26.74	2.67	24.06
15	4.06	0.66	21.26	2.13	19.14
14	4.25	0.47	15.14	1.51	13.63
13	4.47	0.25	8.05	0.81	7.25
12	4.71	0.01	0.32	0.03	0.29

Appendix Q: All Hydrokinetic Cases for Check 5

<u>All design cases:</u>



<u>All eligible design cases:</u>



All eligible cases for Check 5

Rank	Rated Turbine Size (kW)	Number of Turbines	Channel Width (ft)	Expected Power Generation (kW)	Expected Capital Costs ($US)	Capital Cost per Kilowatt ($US/kW)
1.	25	1	20	13	123200	9500
2.	10	4	17	13	227400	17500
3.	5	7	16	12	214500	18000
4.	10	7	18	20	391100	20000
5.	10	6	18	15	338100	22500
6.	10	5	18	12	281100	23500
7.	5	10	17	13	310900	24000
8.	5	9	17	11	275200	25000
9.	10	9	19	20	502000	25100
10.	10	8	19	16	449000	28000
12.	10	7	19	14	391900	28000
13.	10	6	19	11	338900	30800
14.	5	10	18	10	312200	31200
15.	10	10	20	16	564600	35000
16.	10	9	20	14	503000	35900
17.	10	8	20	12	450000	37500
18.	10	7	20	10	392700	39300
19.	25	5	Orig*	15	572500	38100
20.	25	4	Orig*	11	458000	41600

* Orig: original channel design (unlined trapezoidal channel with a base width of 20 ft)

Appendix R: Conventional Hydropower Evaluation for Check 5

Power Evaluation

$\eta = 0.9$

$\rho = 1000 \text{ kg/m}^3$

$Q = 379 \text{ cfs} = 10.73 \text{ m}^3/\text{s}$

$H = 10.6 \text{ ft} = 3.23 \text{ m}$

$P = \eta\ \rho\ Q\ g\ H\ 10^{-3} = (0.9)\ (1000 \text{ kg/m}^3)\ (10.73 \text{ m}^3)\ (9.8 \text{ m/s}^2)\ (3.23 \text{ m})\ (10^{-3}) = \boxed{\textbf{305.68 kW}}$

Cost Evaluation

$(305.68 \text{ kW}) * (\$5{,}000) = \boxed{\textbf{\$1,528,413}}$

Appendix S: Friction Losses due to Steel Penstock for "Cemetery" Check

Flow Rate =	316	cfs
Gravity =	32.2	ft/s^2
Steel Pipe =	0.0001475	ft
Penstock =	1500	ft

Assumed Pipe Diameter	Pipe Velocity (ft/s)	Estimated (e/d)	Estimated Friction Factor (Moody Diagram)	Head Loss $f*L/D*(v^2/2g)$ (ft)
4	25.146	0.000037	0.01	36.85
5	16.094	0.000030	0.0095	11.47
6	11.176	0.000025	0.009	4.37
7	8.211	0.000021	0.0085	1.91

Appendix T: Conventional Hydropower Evaluation for "Cemetery" Check

<u>Option 1</u>

Power evaluation:

$\eta = 0.9$

$\rho = 1000\ kg/m^3$

$Q = 316\ cfs = 8.94\ m^3/s$

H = (total head) – (head loss of penstock) = 40.97 ft – 11.5 ft = 8.98 m

$P = \eta\ \rho\ Q\ g\ H\ 10^{-3} = (0.9)(1000\ kg/m^3)(10.65\ m^3)(9.8\ m/s^2)(8.98\ m)\ (10^{-3}) = \boxed{\textbf{843.5 kW}}$

Cost evaluation:

(843.5 kW) * ($5,000) = $\boxed{\textbf{\$4,217,600}}$

<u>Option 2</u>

Power evaluation:

$\eta = 0.9$

$\rho = 1000\ kg/m^3$

$Q = 316\ cfs = 8.94\ m^3/s$

H = 8.86 ft = 2.7 m

$P = \eta\ \rho\ Q\ g\ H\ 10^{-3} = (0.9)\ (1000\ kg/m^3)\ (8.94\ m^3)\ (9.8\ m/s^2)\ (2.7\ m)\ (10^{-3}) = \boxed{\textbf{212.90 kW}}$

Cost evaluation:

(212.90 kW) * ($5,000) = $\boxed{\textbf{\$1,064,500}}$