Renovating Aging Penstocks: Analyzing Replacement Alternatives for Cost and Energy Generation

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ABSTRACT

The Bulls Bridge hydroelectric development, located in the towns of Kent and New Milford in Litchfield County, Connecticut and currently owned by FirstLight Power Resources (FLPR), is one of five hydropower developments forming the Housatonic River Hydroelectric Project. The development was constructed in 1903 with a 13 foot diameter steel penstock feeding six units, and in 1912 an additional 8 foot diameter steel penstock was constructed to provide additional flow.

By 2013, the two penstocks required annual dewatering for inspection and repairs of leakage. The penstocks were exhibiting deformation and corrosion at the saddle supports, particularly along the 13 foot diameter penstock. A 2014 alternatives analysis considered four options to rehabilitate the penstocks: (1) replace the existing 13 foot diameter penstock with a new 13 foot diameter steel penstock; (2) replace both penstocks with either a 14 foot by 14 foot concrete penstock or a 15 foot diameter steel penstock; (3) replace both penstocks with two 10 foot diameter Fiberglass Reinforced Polymer (FRP) pipes; or (4) rehabilitate the existing penstocks.

Based on the findings of the alternatives analysis and experience from previous penstock rehabilitations, FLPR selected the two 10 foot diameter FRP penstock alternative (alternative number 3) due to its low construction cost and minimal change in energy generation. The design of the new FRP penstock involved several key components including aligning the new pipe to join with the existing manifold, designing three thrust blocks, and designing connections to the existing manifold.

Construction of the new FRP penstock began in September 2015 and was completed in January 2016. The new penstock consists of two sections of FRP pipe that transition to the steel manifold at the powerhouse. Installation on a slope and under a Connecticut state highway bridge created difficulties during construction. The FRP penstocks are partially buried and do not have any cradle supports. The construction also included replacement of the penstock intake gates and repairs to the surge risers.

INTRODUCTION

The Bulls Bridge hydroelectric development (Development) is one of five hydropower developments on the Housatonic River that together form the Housatonic River Hydroelectric Project. The Development is located in the towns of Kent and New Milford in Litchfield County, Connecticut.

The Development is operated as a run-of-river facility and consists of two dam structures, a power canal, an intake structure, two intake gates, two penstocks that convey water from the intake structure approximately 425 feet downhill to the powerhouse, the powerhouse manifold which distributes flow from the penstocks to the units, two 100 foot high surge tanks on the manifold, and a powerhouse that houses six identical double runner horizontal-shaft Francis turbines.
The Development was constructed in 1903 with only one 13 foot diameter penstock to convey water from the intake structure to the powerhouse manifold. In 1912, an 8 foot diameter penstock was added into the water conveyance system to provide additional flow to the units. The 8 foot penstock was installed adjacent to the 13 foot penstock and connected to the upstream 8 foot diameter vertical surge tank.

Recent inspections by FLPR and maintenance concerns led to the need to develop an overall repair or replacement strategy to keep the penstock operational. The areas of primary concern with the steel penstocks included:

- The majority of the 13 foot diameter penstock rivet seams showed continued leakage;
- The upstream section of the 13 foot diameter penstock supported only by the continuous 2.5 foot wide invert concrete support was experiencing localized deformation that was the primary cause of the persistent shell rivet seam leakage;
- Both penstocks showed accelerated corrosion in the area where the penstocks’ shells contacted the concrete saddles, which caused holes to form on the shell at the top of the concrete saddles.

In 2013, Gomez and Sullivan Engineers, D.P.C. (Gomez and Sullivan) was contracted by FLPR to perform an alternatives analysis of various repair options. A report summarizing the alternatives analyses was submitted in July of 2014, and Gomez and Sullivan was subsequently retained to provide design services for the repairs selected by FLPR. The objective of this paper is to review the alternatives analysis and the design and construction of the selected alternative.

ALTERNATIVES ANALYSIS

The alternatives analysis included an assessment of eliminating the surge tank, replacing the manifold, evaluating potential penstock material choices, developing alternative replacement configurations, and reviewing rehabilitation options for the existing penstocks.

**Elimination of Surge Tanks and Replacement of Manifold**

A 2004 Penstock Conditions Assessment Report indicated the surge risers could be removed to reduce future maintenance costs but also indicated the need for additional analysis. Gomez and Sullivan performed this analysis as part of the alternatives analysis, and it included: a transient analysis to estimate the transient pressures in the system without surge risers; stress calculations for the turbine casings, manifold, and penstocks to assess their ability to resist the transient pressures.
pressures; an evaluation of impacts to inertial stability of the system; and the costs associated with removal versus maintenance of the surge risers.

**Transient Analysis**

The transient analysis was performed using the Allievi Water Hammer Charts provided in the Hydroelectric Handbook, Second Edition (Creager and Justin, 1950). The computations were performed for the original penstock configuration and assumed that no surge tanks are present during a full load rejection of all of the units, which was assumed to be the worst case scenario. The analysis did not consider the reflection of waves from branch pipes (i.e. the manifold) as is common for a hydroelectric project; further consideration of this interaction through the manifold is best suited for a computational fluid dynamics (CFD) model of a transient simulation model. The headlosses were computed from the intake to approximately 400 feet downstream of the intake, as this is just upstream of the manifold.

The analysis was performed assuming the maximum reservoir operating level (i.e., Elevation 356 foot Connecticut Light and Power - CL&P Datum) for the headwater, and a tailwater elevation of 245.5 feet. A sensitivity analysis of the tailwater elevation suggested that the tailwater had an insignificant impact (of about one foot) on the rise in head due to transients. Using these assumptions and a factor of safety of 1.5 (an additional 50%) to account for the complexity of the manifold, the transient analysis indicated an increase in head due to transients with the surge risers removed of approximately 27 feet in the 13 foot diameter penstock and 36 feet in the 8 foot diameter penstock.

It was noted that due to the complexity of the system with the interaction between the two penstocks through the manifold, the transient pressures of the existing configuration (i.e., with the surge risers present) could not be easily obtained using hand calculations. Further, the surge risers may act as a pressure relief due to their limited height (i.e., the risers do not go to the full height of the anticipated surge pressures), thus producing less pressure rise than suggested by the analysis performed. In order to better evaluate the function of the current surge risers, it was noted that pressure measurements during a full load rejection should be obtained. However, the interaction of the penstocks through the manifold and the pressure relief from limited surge riser height were not considered in this analysis.

Figure 2: Manifold and surge risers at the Bulls Bridge powerhouse
**Inertial Stability**

To investigate the impact the surge risers have on inertial stability of the turbines a simplified analysis was performed. First, the impact of the risers was evaluated. Second, the overall inertial stability of the powerhouse units was evaluated. An analysis based on the Thoma stability criteria (Zipparro and Hasen, 1993) was used to determine whether the surge tanks were effective in adding to the inertial stability of the system. Computations following this criteria indicated the surge risers do not have a significant effect on the inertial stability of the units. Thus, it was deemed appropriate to ignore the effect of the surge risers when evaluating the inertial stability of the units while noting that the true effect of the surge risers may not be adequately assessed by this stability criteria, due to the complex nature of the manifold and surge riser configuration.

The inertial stability of units at the Bulls Bridge Powerhouse was next evaluated by computing the ratio of the machine starting time \(T_m\) to the water column starting time \(T_w\) using formulas provided by the U.S. Bureau of Reclamation (USBR). Some of the turbine and generator properties used for input values in these formulas were based on additional USBR formulas rather than plant records. It was recognized that these additional formulas are based on experience with modern hydroelectric generating equipment and may not be applicable to equipment of the age of that at the Bulls Bridge development. However, it was determined that these formulas were adequate for an initial analysis, and any further analysis for a potential final design would use values obtained from plant records, the manufacturer, or direct measurement.

The computed stability index value of 5.8 was greater than the criteria (2.0), thus the plant governing stability was considered satisfactory according to the criteria used. Since the analysis of inertial stability ignored the surge tanks, it suggested they could be removed without significant impacts to the inertial stability of the system.

**Penstock/Manifold Condition – Observations**

In support of the project, thickness measurements and existing conditions of the turbine casings, existing manifold, and both penstocks were recorded. The measurements indicated that the 13 foot penstock upstream of the manifold and the entire 8 foot penstock were generally in good condition. However, while the majority of the 13 foot diameter pipe was in good condition, there were several isolated locations in poor condition particularly where the pipe met the edges of the concrete saddles. At these locations, the pipe appeared to have multiple patches to address persistent corrosion problems.

The manifold appeared to be in good condition with little variation in the interior and exterior appearance. The only other areas of concern were within 50 feet of the intake structure where there were some pinholes found in the steel. The 8 foot diameter penstock appeared to be in better condition than the 13 foot diameter since it was supported farther off the ground and carried smaller loads than the 13 foot penstock while being better supported by the existing concrete saddles.
Visual observations of the 13 foot diameter penstock indicated significant localized penstock deformation and corrosion at the supports of the penstock. A 2004 Penstock Condition Assessment Report noted that the “13 foot diameter pipe is supported by a continuous concrete cradle under only the bottom 22 degrees (2.5 foot) of the invert, which is only 20% of the normal 120 degree invert saddle support.” The small size of the cradle and localized deformation indicated that the concrete supports were insufficient. Further, there appeared to be accelerated stress induced corrosion at these locations causing frequent maintenance issues.

The surge risers were observed from within the manifold from below to provide a visual assessment of the condition of the gunite liners. The gunite appeared mostly intact and solid, however there was some delamination noted near the bottom of the risers. Two butterfly valves within the manifold were also inspected. One valve at the end of the 8 foot penstock before it connects to the surge riser appeared to be in good condition. The other valve between the 13 foot penstock and the 8 foot surge riser appeared heavily eroded along the edges and would not create a complete seal if closed. Both butterfly valves were secured in the open position and not used during normal operations.

**Penstock, Manifold, and Turbine Casings – Stress Calculations**

Stress calculations for the penstocks, manifold and turbine casings based on the thickness measurements were performed to investigate the possibility of shell rupture. The hoop stresses in the base material were calculated according to the ASCE Manuals and Reports on Engineering Practice No. 79 Steel Penstocks, Second Edition (ASCE Manual of Practice No. 79). The calculated hoop stresses, with a factor of safety of 1.5 included, were compared to the allowable material stress to determine the stress ratio at the reading location. Stress ratios under a value of 1.0 suggest sufficient factors of safety for the section. The penstock material was assumed to be a low carbon steel comparable to A36 steel with tensile strength of 52 ksi and yield strength of 26 ksi. For all the locations thickness measurements were taken on the penstocks and manifold, the base material had stress ratios under 1.0 and therefore within the factor of safety.
Hoop stresses were also calculated for the longitudinal joints along the penstocks and the manifold. The stresses were calculated for three different joint arrangements in the manifold, the 13 foot diameter penstock, and the 8 foot diameter penstock. For each arrangement, the hoop stresses were calculated using the same method as for the base material but with an additional joint efficiency factor applied. The stress ratios for the joints in the 13 foot and 8 foot penstocks were all under 1.0 and therefore within the factor of safety. However, the stress ratios for several of the joint locations within the manifold were greater than 1.0, indicating the potential for lower factors of safety (on the order of 1.2 or higher when considering full surge). It should be noted that these calculations included the full surge pressures from the transient analysis assuming no surge tanks with an added factor of safety of 1.5 on the surge pressures. These surge pressures may be overly conservative as the existing surge tanks may limit the total pressure rise in the manifold. Further, existing operations have not caused failure of the manifold and the strength of the manifold has been sufficient over the life of the project.

Longitudinal stresses were also calculated for the manifold spans and penstock sections per the ASCE Steel Penstocks manual. For the longitudinal stress calculations, the beam action in the manifold and penstock was generally on the order of 2-4% of the allowable stress of the material. The calculations indicated that the hoop stresses govern the strength of the penstock and manifold and that longitudinal stresses are not as significant.

Following recent leakage observed in some of the turbine casings, thickness measurements on the casings were taken. Using the turbine casing thickness measurements, hoop stresses in the base material and joints were calculated using the same methods as for the penstocks. The stress ratios for the base material were all under 1.0 and within the factor of safety for the turbine casings. The calculations suggest sufficient factors of safety for the turbine casings.

Based on the observations, stress calculations, transient analysis, and inertial stability analysis it was confirmed that the penstocks where in need of repair or replacement. Additionally, it was concluded that the existing manifold and surge risers were acceptable for continued operations. It was assumed the existing manifold and surge risers would remain for all the penstock replacement configurations.

**Replacement Configurations**

Through consultation with FLPR, conceptual replacement designs were prepared for four major alternative configurations. Each of these configurations are discussed in further detail below.

**Alternative 1. Existing configuration**

The first alternative proposed to replace the existing 13 foot diameter penstock with a new 400 foot long, 13 foot diameter steel penstock in the same location. The new steel pipe sections would extend from the existing upstream gate to the manifold at the powerhouse. Existing and new concrete foundations with new steel saddles would support the new penstock. New rock anchors would be drilled through the existing concrete foundations and into bedrock. Based on the field observations and stress calculations, the existing 8 foot diameter penstock and manifold would remain in service.
Alternative 2. One new penstock
The second alternative proposed to replace both the existing 13 foot and 8 foot diameter penstocks with either a 400 foot long, 14 foot by 14 foot (inside dimension) concrete box or a 400 foot long, 15 foot diameter steel penstock. The new conduit would be constructed where the existing 13 foot penstock was located. The concrete box would utilize the existing concrete foundations for the 13 foot penstock. New rock anchors would be drilled through the existing concrete foundations and into bedrock. The 15 foot steel pipe would be buried in fill to the mid-point of the pipe.

Alternative 3. Two new penstocks
The third alternative proposed to replace both the existing 13 foot and 8 foot diameter penstocks with two 10 foot diameter FRP penstocks. The northern pipe would be about 375 foot long and the southern pipe about 315 foot. Both new penstocks would be constructed in the location of the existing penstocks and would be backfilled to the springline of the pipe. Each penstock would have a new concrete and steel transition at the downstream connection to the manifold. Preliminary discussions with Hobas Pipe USA (Hobas) confirmed the proposed construction details and limits on pipe size for transportation. Although a 10.5 foot (126 inch) diameter pipe is made by Hobas, the cost increase for the marginal increase in pipe size was more significant than the corresponding savings in headloss reduction.

Alternative 4. Rehabilitation of penstocks and foundations
The fourth alternative proposed the rehabilitation of the 13 foot diameter steel penstock in place and enlarging its foundations. This plan would provide new concrete supports staggered between the existing supports and an enhanced design providing greater bearing area to avoid the punching problem observed at the existing supports. Next the existing supports would be removed and new plates bent to the approximate profile of the penstock and extending approximately 6-in beyond the limits of the old supports would be welded in place on the exterior of the penstock. To provide an additional measure of security, clean well graded gravel or graded crushed stone would be placed up to the springline of the pipe between the new concrete supports.

Watertight internal liners that could be sprayed-on at the deteriorated surface areas of the penstocks as well as thicker liners to be applied at the areas with pitting and rivet leakage were investigated. The external liners would be patched or replaced as well.

Common Repair Items
Inspection of the interior of the manifold and lower areas of the risers and analysis of the horizontal riveted joints in the manifold resulted in the addition of items of work that are common to all alternatives. The gunite lining in the risers in the areas where delamination was noted would be blasted clean to remove the gunite lining, and an interior coating would be applied that would extend beyond the limits of the gunite lining to remain in place. In addition, it was assumed that targeted replacement of existing rivets and connections at horizontal riveted seams would be required for all the alternatives. Due to the existing condition of the headgates, replacement of these gates was also included for all alternatives. This work included changes to the existing gate slots to accommodate new gates and construction of a bulkhead.
**Penstock Materials**

Several penstock materials were evaluated to determine the most practical and cost efficient material for replacement or rehabilitation at the site, if the penstocks were to be replaced. The evaluation included a review of steel, fiberglass, concrete and High Density Polyethylene (HDPE). The advantages and disadvantages that were considered for each material are listed below.

*Steel*

Steel pipe has been produced since the 1800’s and used for hydroelectric applications for about as long. Steel pipe advantages include its great tensile strength, durability and long life (100 years or longer), cost effectiveness to purchase, install and maintain, high carrying capacity, ductility, adaptability, reliability and resiliency to water hammer. Available interior and exterior coatings and cathodic protection can extend the pipe life even further. Disadvantages include more time for installation than some of the other pipe materials due to the welding, potential corrosion issues if not properly coated or protected, and more exact trenching and cradle requirements.

*Fiberglass*

Fiberglass Reinforced Polymer (FRP) pipe has been produced for more than 40 years and has been used in hydropower applications since the early 1980’s. The advantages of FRP pipe include its high pipe stiffness, corrosion resistance, long-term flow performance, light weight for easier installation, low maintenance costs, and the ability for it to be manufactured in diameters large enough for hydropower applications. Disadvantages include the higher cost of the pipe, UV degradation, fabrication and installation of fittings, location of fabricators which can lead to high freight charges, and quality monitoring of the pipe. Further, each manufacturer’s material properties are proprietary, and production capabilities such as span lengths, diameters, etc. vary between manufacturers, so a detailed design requires significant input from each manufacturer.

*Concrete*

Concrete for conduits has been available since the 1800’s and reinforced conduits since the early 1900’s. Concrete conduit advantages are its high compressive strength, durability and longevity (100 years or longer), availability, and available sizes. Disadvantages include more time required for installation than some pipes due to the cast-in-place method, and a higher cost than some of the other pipe materials.

*HDPE*

HDPE piping has gained ground in the penstock and water piping industry. Sizes of HDPE range from very small to a maximum of about 5 feet in diameter. The advantages of HDPE include flexible pipe that adapts to rugged conditions, reduced installation time due to heat-fused pipe connections, smooth interior for minimal headlosses, cost-effectiveness, chemical/corrosion resistance, lightweight, and long term strength and ductility. Disadvantages include lower pressure ratings for larger diameter pipes than for similar sized pipes of other material, and limited larger size diameters.
Economic Considerations

Opinion of Probable Construction Cost

Opinions of probable construction costs (OPCCs) were prepared for each of the proposed alternatives to identify capital costs required for each alternative. Conceptual sketches of each selected alternative were developed at a sufficient level of detail to communicate the proposed improvements and to enable development of an OPCC. The OPCC was developed at a budgetary level using a combination of vendor quotes, R.S. Means Construction Cost Data, industry standard cost data, and available final costs from comparable projects as appropriate. The OPCC included itemized costs for final design and permitting, mobilization/demobilization, access and water handling, erosion and sediment control, proposed work, and construction phase services. For a preliminary design stage, a 25% contingency was carried. The following are the OPCCs for the selected alternatives.

Table 1: Opinion of Probable Construction Costs for Penstock Alternatives

<table>
<thead>
<tr>
<th>Alternative</th>
<th>OPCC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alternative 1 – New 13 foot Dia. Steel Penstock</td>
<td>$3,194,000</td>
</tr>
<tr>
<td>Alternative 2A – 14 foot x 14 foot Concrete Box</td>
<td>$5,344,000</td>
</tr>
<tr>
<td>Alternative 2B – New 15 foot Dia. Steel Penstock</td>
<td>$3,401,000</td>
</tr>
<tr>
<td>Alternative 3 – Two 10 foot FRP Penstocks</td>
<td>$3,020,000</td>
</tr>
<tr>
<td>Alternative 4 – Rehabilitate Existing 13 foot Penstock</td>
<td>$3,978,000</td>
</tr>
</tbody>
</table>

Budgetary estimates for removal of the surge risers and manifold and their replacement with a new manifold were also carried out at an initial level. However, since the existing manifold was functional and its condition appeared sufficient for its intended use, the cost of replacing the existing manifold was not warranted based on nominal increases in energy for more efficient hydraulic conditions.

Energy Analysis

An energy analysis spreadsheet was utilized to compute the Annual Net Generation for the site based on an annual flow duration curve. Using the flow, net head (feet) and turbine efficiency for a given alternative, the station generation was computed. The annual total generation was decreased by 6% to account for scheduled and unscheduled operations and maintenance and for transformer losses. Therefore, the annual net generation computed represents 94% of the estimated annual total generation.

The flow available for generation considered a 200 cfs minimum flow as indicated in the November 2004 Supporting Technical Information Document (STID). The powerhouse contains six identical Francis turbines with a total station flow capacity of 1,250 cfs. Each unit was assumed to have a maximum capacity of approximately 208 cfs. The minimum flow capacity of each unit was assumed to be 40% of the maximum unit capacity (i.e. approximately 83 cfs), which is typical for Francis units. The turbine efficiency was estimated to be 62%, based on the nameplate rating stated in the STID and losses associated with the generator. The model utilizes a constant headwater elevation of 353 CL&P Datum, as the STID indicates that the canal intakes are
automated to maintain the headwater at the powerhouse at this level. The estimated tailwater, flow, and headlosses at the site were calculated for the analysis as detail below.

A tailwater curve was estimated as one was not available for the site. This rating curve was based on available USGS gage measurements of gage height and flow and on flow and elevation information obtained from the FEMA Flood Insurance Study (FIS) for New Milford Township. The nearest USGS Gage is located approximately 0.4 miles downstream of the Bulls Bridge Powerhouse, and there are no major tributaries entering the Housatonic River between the gage and the powerhouse.

The energy analysis was performed using an annual flow duration curve based on data available from USGS gages near the Development. The USGS gages on the Housatonic River were evaluated for proximity to the project site and number of years of available data. From this, two gages were identified as the most appropriate for the development of a flow duration curve. Of the two gages, USGS gage 01200500 at Gaylordsville, CT was selected for use in the energy analysis due to its proximity to the site and available historical data. In addition, flow from gage 01200000 Ten Mile River near Gaylordsville enters the Housatonic River in between the canal intake and the powerhouse and was used to adjust flows at gage 01200500. The site has an estimated drainage area of 784 square miles, and the adjusted Gaylordsville gage has a drainage area of 793 square miles (i.e. 101% of the site drainage area). The flow at the adjusted gage was prorated based on the ratio of the drainage areas for the annual flow duration curve at the Development.

Headlosses were estimated for each alternative from the intake to approximately 400 feet downstream of the intake. Headlosses for the manifold were not estimated because they remain essentially the same for each alternative. This approach overestimated the net annual generation, but had no impact on the alternative comparison. Variables considered in the computation of headloss for each alternative included trashrack losses, friction losses, and minor losses (i.e. entrance, vertical and horizontal bends, expansion/contraction transitions, head gates, and tees) as appropriate.

The energy analysis indicated alternatives 2A, 2B, and 3 were expected to provide marginally less annual net generation than the existing condition. Additionally, it appeared that Alternative 1 would provide a slight increase in annual net generation. The following table is a direct comparison of the impact of the headlosses on the energy generation for each alternative.

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Annual Net Generation at Site (GWh/yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Existing</td>
<td>32.88</td>
</tr>
<tr>
<td>1</td>
<td>32.93</td>
</tr>
<tr>
<td>2A</td>
<td>32.84</td>
</tr>
<tr>
<td>2B</td>
<td>32.84</td>
</tr>
<tr>
<td>3</td>
<td>32.81</td>
</tr>
<tr>
<td>4</td>
<td>32.91</td>
</tr>
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</table>
RECOMMENDATIONS

Gomez and Sullivan provided several recommendations to FLPR based on measurements and analysis of existing conditions, the OPCCs of the alternatives, and the energy analyses of the alternatives.

Existing Surge Risers and Manifold

The first recommendation given in the alternatives analysis was to maintain and repair the surge risers and manifold as required. The transient analysis, inertial stability, and economic analysis indicated that the gains in energy production do not offset the cost of a new manifold. Also, based on past performance and observed existing conditions it was Gomez and Sullivan’s opinion that the existing manifold and risers, with proper maintenance, should continue to perform for 30 years. Economic analysis included in the OPCCs indicated the net present value for the maintenance of these structures for the next 30 years should be less than replacement.

The recommended repairs to the surge risers included removing the deteriorated gunite lining, coating the area of removal with a lining to help slow future deterioration, and inspecting the remaining gunite on a periodic basis to check its integrity.

In addition, Gomez and Sullivan recommended obtaining actual transient pressure readings during a full load rejection to confirm and calculate factors of safety in the manifold. Alternatively, a CFD model or a more complex transient analysis was suggested if FLPR desired to remove the existing surge risers. A thorough inspection of the rivets and seams and replacement of any deteriorated rivets as well as repairs to prevent further leakage from the areas previously noted on the turbine casings were also recommended.

Penstock

Based on observations, calculations, energy analysis, and the OPCCs, the two most beneficial alternatives appeared to be Alternative 3 (replacing both penstocks with two new 10 foot diameter FRP penstocks) and Alternative 1 (replacing the 13 foot diameter penstock in kind with a new 13 foot diameter steel penstock and retrofit/replacement of the foundations). Alternative 3 had the lowest total OPCC, while Alternative 1 had the highest estimated net generation. The lower cost of Alternative 3 outweighed the potential gains in energy generation of Alternative 1, thus Alternative 3 was recommended as the most cost effective alternative for penstock replacement at Bulls Bridge.

DESIGN

A summary of the penstock replacement alternatives, OPCCs, energy analysis, and recommendations was submitted in a report to FLPR dated July 2014. Based on that report and experience with past penstock rehabilitation projects, FLPR selected to proceed with the design of Alternative 3: replace the existing penstocks with two 10 foot diameter FRP penstocks. The design of the new penstocks included aligning the new pipe with the existing manifold and the design of thrust blocks, transition sections to the manifold, and new intake gates.
The locations of the various design elements were based on two surveys. The first survey of the project site and surrounding area was performed in 2011 and was developed with photogrammetric methods from color aerial photographs. The second survey was performed in support of the penstock rehabilitation and was compiled by a land surveyor. This survey included more detailed information about the locations of the existing penstocks and site structures.

**Penstock**

The design of the two 10 foot diameter penstocks was based on the pressure class required for the design head and discussion with Hobas. Other FRP pipe suppliers were researched, but Hobas was chosen due to FLPR’s past experience with them, cost, and availability of materials. The design head was computed as the difference between maximum reservoir operating level (Elevation 356 foot CL&P Datum) and the manifold centerline (Elevation 257.9 foot CL&P Datum). The required pressure class for the penstock was then determined using the design head.

During the design, it was concluded that the most efficient method to transition from the intake to the new 10 foot diameter pipes was to extend the new pipes to the start of the intake and remove or fill concrete in the intake block as required around the new pipe. For the transition to the manifold, two steel reducers were proposed that would then be welded with a butt strap connection to the existing manifold pipe.

The intake and the reducers provided the points between which the FRP pipe was aligned. A single bend in the alignments was added approximately 150 feet downstream of the intake to avoid excavation of bedrock near the existing ground surface. The alignments were also restricted by a bridge the penstocks pass under. To avoid the lower limit of the bridge, the centerline elevation at the intakes was lower which required additional concrete removal of the intake block.

Since penstock saddles had caused issues with the existing steel penstocks in the past (deformation and localized deterioration) and the existing ground surface was relatively close to the penstocks, the existing saddles were removed and the new FRP penstocks were supported by fill extending to just above the springline of the pipe. The fill was used to support the pipe from the downstream side of the intake block to retaining walls located at the two thrust blocks at the transition pieces. The depth of fill was designed to be two feet above the springline of the FRP pipe to provide adequate friction resistance to prevent sliding movement on the sloped installation surface. Three ring girders were designed according to ASCE Manual of Practice No. 79 to support the transition sections between the thrust blocks and the connections to the manifold where there was no fill.
The new FRP penstocks were supplied by Hobas in approximately 20 foot segments. The segments are connected by couplings, with special closure couplings for the final segment installed to provide a seal. A total of thirty-two segments comprised the new penstocks, with four of the segments acting as closure pieces. Four closure segments were provided so each section of pipe between the thrust blocks could be closed separately. The FRP pipe was connected to the steel pipe with couplings that allowed for minor axial movement. An expansion joint was also included in the design for the 8 foot diameter steel pipe connection to the manifold because axial movement from thrust loads in the manifold and thermal movement was expected. The 13 foot diameter pipe did not have a separate expansion joint since the manifold section was restrained by existing concrete supports and the coupling connection to the FRP pipe allowed for axial thermal movement.

**Thrust Blocks**

Three thrust blocks were designed to hold the penstock in place and counteract thrust loads created by flow. One is located midspan at the bend in the alignment 150 feet downstream from the intake for each pipe, and two more are located at the transitions to the steel manifold. All three thrust blocks were designed to withstand overturning due to hydrostatic forces, hydrodynamic forces, and backfill soil pressures (due to uneven levels created by the slope of the hillside the penstock descends).

All the thrust blocks were designed to bear on bedrock. Since no data was available for the depth from the existing ground surface to bedrock several depths were designed for based on expected depths determined by field observations (locations with visible bedrock). To confirm the design, at least one boring at the location of each proposed thrust block was scheduled to occur during construction, and anchors into bedrock were planned if the measured depths exceeded the design depths. The thrust block located 150 feet downstream of the intake was initially designed as two separate, adjacent thrust blocks. However, this design was revised during construction as a single thrust block to provide additional stability.

**Penstock Support Frame**

Adjacent to the powerhouse, an existing steel frame supported the 8 foot diameter penstock between the manifold and an existing thrust block. The frame was constructed with built-up steel I-shaped columns and steel angles. Inspections of the frame noted that many of the members showed signs of deterioration with some spots rusted through the full thickness of the members. Repairs to these members were proposed as part of the penstock rehabilitation.

To determine the extent of the repairs and possible partial or full replacement of the frame, a finite element model of the frame was created and analyzed. The model

*Figure 5: Support frame under the 8 foot diameter penstock*
accounted for loads from environmental factors (wind, snow, and seismic), axial loads from thermal movement of the pipe, and thrust loads from the manifold and new reducer section. Based on the loads input and the requirements of the AISC Steel Construction Manual, the model indicated that the design of the existing frame was acceptable to resist the loads with sufficient factors of safety. Since the existing members were sufficient but in poor condition, in-kind replacement of the deteriorated frame members was proposed.

**Bulkheads and Head Gates**

During the investigation of the existing conditions, it was noted that the two gates located at the intake structure were not functional and in need of replacement. One of the gates could not be operated and was stuck in the open position. The other gate could be operated but did not form a seal along the upper edge when closed and water could flow over the top of the gate. Instead of operating the gates, FLPR used the gates to the power canal located next to the dam spillway to stop flow; flow can then be drained through the powerhouse and gates along the canal.

Full replacement of both intake gates was planned as part of the design. This included removal of the existing gates and operators and installation of new gates with actuators, gate slots to fit the new gates, and bulkheads to create a seal with the top of the gate when the gate is closed. The new gates were designed by Steel-Fab, Inc. (out of Fitchburg, MA) and the bulkheads were designed by Gomez and Sullivan. The bulkheads were designed as steel member frames with skin plates and were designed to resist hydrostatic and winter ice loads and to provide a full seal with a closed gate during maximum reservoir operating level.

**CONSTRUCTION**

The final design was submitted to the Federal Energy Regulation Commission (FERC) in March 2015. FERC approved the project noting that if the design changed due to the results of the borings the revised design would need to be resubmitted. In June 2015, Bancroft Contracting Corporation (Bancroft) was awarded the construction contract, and construction started in September 2015.

Once the existing pipe was removed from the proposed boring locations, five borings were performed: two at the midspan thrust block, one at the transition thrust block to the 13 foot diameter manifold pipe, and two at the transition thrust block to the 8 foot diameter manifold pipe. Based on the borings, the bedrock elevation at the midspan thrust block and the transition thrust block to the 8 foot diameter manifold pipe was lower than expected. Increasing the depth to bedrock did not require a significant change to the design of the midspan thrust block, and combining the originally planned two separate sections into one section provided the needed stability. However, the 8 foot pipe transition thrust block required an additional eighteen anchors drilled into bedrock to stabilize the thrust block with the corrected depth to bedrock. This revised design was submitted to FERC and again approved.

During the design phase, several locations for an access road to the penstock were investigated. Due to large amounts of vegetation and trees, the potential locations for this access road where limited; therefore, a path passing through the existing substation and behind the powerhouse was
proposed. After temporary access through the substation for construction was approved by the owner of the substation, the access road was aligned and constructed by the contractor.

Once the design of the thrust blocks was finalized, installation of the pipe and construction of the thrust blocks began. After the removal of existing steel and concrete was completed, the first segments of FRP pipe were installed at the intakes. Installation of pipe segments continued downstream from the intake. Once the four thrust blocks were constructed, pipe segments were installed out from those points as well to the planned closure piece locations. One challenge during the installation of the pipe was fitting the segments underneath the existing bridge over the penstocks. Since the pipe could not be lowered directly into place at that location, the contractor devised a rig to move the pipe up the slope and into place. The rig was composed of a horizontal beam attached to a telescoping column set on pneumatic tires; one end of the rig was then lifted by a backhoe, driven through the pipe, and the column was extended to lift the pipe and move it.

After individual pipe segments were placed, drain pipes were installed parallel to the penstocks with risers for maintenance access every one hundred feet. An electrical conduit was also installed parallel to the penstocks from the intake gatehouse to the powerhouse to allow remote operation of the new gates. Fill was placed over the drain pipes and conduits up to two feet above the springline.

The final items installed were the replacement members for the support frame of the 8 foot diameter penstock, the intake gates, and the bulkheads. Once the final items were installed an initial test of the system with water at the normal operating level was performed to check for leaks. Only two minor leaks at joints on the steel transition sections were noted and subsequently repaired. In January 2016, construction was completed, the penstock and powerhouse were watered up, and operation resumed.
REFERENCES


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Mr. Sawyer is a civil/structural engineer with Gomez and Sullivan Engineers, D.P.C. with experience in dam stability and design of steel and concrete structures for dams. His recent work has included designs of trash racks, penstocks, and several types of fish passage structures such as fish lifts, fish ladders, fish barriers, and plunge pools. Mr. Sawyer received a B.S. in Civil Engineering from the University of New Hampshire and is currently a registered Engineer-In-Training.