Study No. 12

Flow Restoration Action Plan for Flow-line or Penstock Shutoff – April 2006 Supplemental Report

Volume I - Public

Mystic Lake Hydroelectric Project FERC No. 2301

Mystic Lake, Montana

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April 1, 2006

Table of Contents

LIST OF TABLES

LIST OF FIGURES

30% VALVE CONSTRUCTION DRAWINGS FOUND IN:

STUDY No. 12 Flow Restoration Action Plan for Flow-line or Penstock Shutoff – April 2006 Supplemental Report, VOLUME II – CEII.

Section 1 - Introduction

1.1 Purpose

The purpose of this task is to develop a 30% design for a flow shutoff and bypass flow restoration system so that if the penstock or flow line is breached, the flow line and penstock can be isolated from the rock tunnel and water flow can be restored to the upper bypass channel to continue on below the Re-regulation Dam. This design and subsequent report provides an investigation of the alternatives, feasibility and conceptual design to provide flow restoration in the event of a penstock breach.

1.2 Project Personnel

The study was organized and managed by GEI Consultants, Inc. GEI provided engineering services for the study and prepared this report.

Key project team members include:

Project Manager	John Pizzimenti, PhD
Project Engineer	Doug Foss, P.E.
Civil/Hydraulic Engineer	Chad Masching, P.E.
Geotechnical Engineer	Marilyn Dodson, P.E.
Structural Engineer	Steve Morris, P.E.
Technical Reviewer	Brian Johnson, P.E.

1.3 Background and Project Understanding

The Mystic Lake Hydroelectric Project (hereafter referred to as the Project) provides no mechanism to reliably close off runaway flow conditions and restore bypass and flows below the Re-regulation Dam in the event that the flow line or penstocks are ruptured or breached by earthquakes, landslides, or other unanticipated incidents. Study No. 12 of the Mystic Final Study Plan (March 2005) called for an investigation of the alternatives, feasibility, and a conceptual design to provide flow restoration in the event of a penstock breach. Currently, the Project is capable of isolating the flow line by only one method, which is by closing the head gate at the extreme uppermost end of the flow line. The existing head gate is located on the northeast shore of Mystic Lake at the entrance to the 1005 ft rock tunnel. When the flow line head gate is closed, there are two methods available to release water to the stream channel between the dam and the powerhouse: by spills when the reservoir water surface is above the spillway crest; and by an existing small outlet at the Mystic Lake Dam. Reservoir spills offer an unreliable means to restore flow to the stream, as the reservoir is operated at levels below the spillway crest most of the time (average of 70% each year). The dam outlet is located near the base of the dam, providing gravity access to a small percentage of the reservoir volume. The use of this outlet as an effective means of flow restoration is questionable, as evidenced by the installation and use of the existing flow restoration valve located near the downstream end of the tunnel.

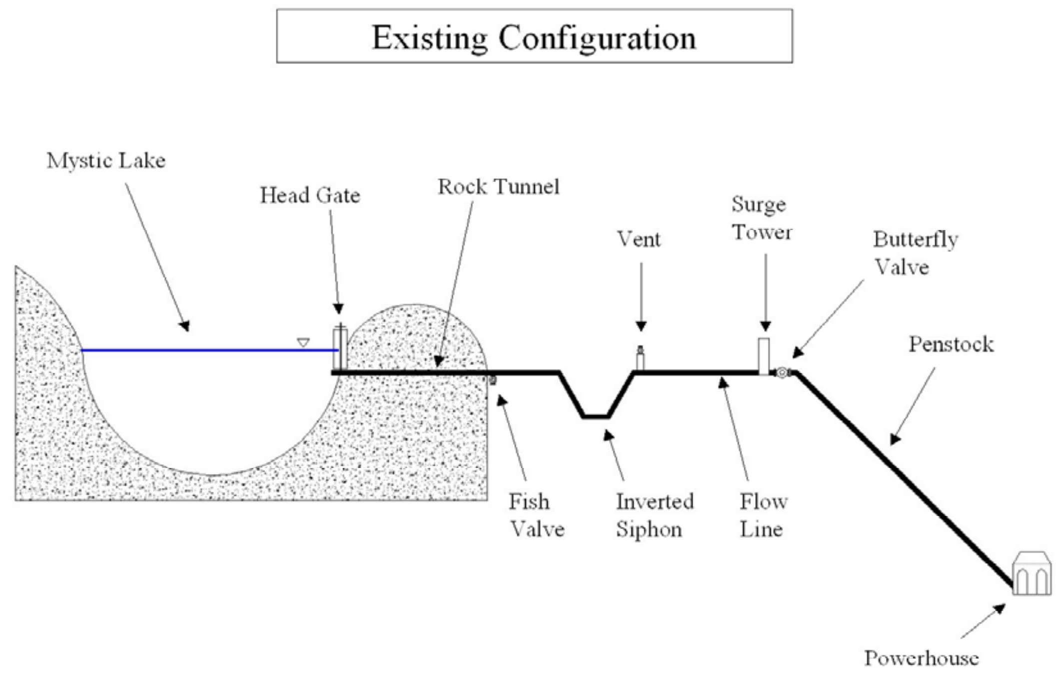

Figure 1.6-1.

Currently, the flow line and penstock system consists of a head gate at the inlet to a 1005 foot long rock tunnel leading from the lake to a 57-inch diameter steel flow line that runs for approximately three miles to a surge tower. The surge tower provides a mechanism to equalize momentum and absorb pressure variations from operational changes that cause any flow variation. Immediately downstream from the surge tower is a butterfly valve that is used to isolate the penstock from the flow line. At the butterfly valve, the flow line connects to the penstock; which carries water to the powerhouse (Figure 1.6-1).

As currently operated, the flow line structure is the only available means to provide flow augmentation into the bypass channel. The flow line structure is a single ten-inch diameter, remotely operated globe valve. This valve is capable of delivering a minimum of 3 cfs under low head conditions and a minimum of 10 cfs during summer months when the lake is near full pool. In the event that the flow line was breached due to some mechanical failure, the head gate would need to be closed and the current flow restoration valve would be ineffective at returning flows to the stream. Currently, the flow restoration at the dam is judged to be generally unreliable, a flow line breach would be expected to result in protracted interruptions of flow in and below the bypass channel.

Subsequently, a 30% design of a workable system to accommodate the need for a reliable means of isolating the flow line from the rock tunnel and restoring flow to West Rosebud Creek in the upper bypass channel and below the powerhouse was undertaken by PPL Montana. This report outlines the findings of that exercise. It has been the purpose of this task to identify workable and reliable configurations to enable flow to be controlled and directed properly under any conditions of weather, pool elevation, and in case of loss of power and/or telemetry with the power plant. This design will require special instrumentation and control systems to correctly and reliably conclude that a penstock or flow line rupture has occurred, and to react appropriately.

1.4 Site Visit

As part of the preliminary design review, two engineers from GEI Consultants, Inc. visited the site in the summer of 2005. The facility from the head gate to the high gradient penstock was examined for constructible and feasible options that were subsequently developed.

Section 2 - Design Criteria

2.1 Operational Criteria

Current regulatory criteria for minimum flows in West Rosebud Creek are the following:

- Minimum flow in the bypass reach must be 10 cfs from June through August and 3 cfs September through May

- Minimum flow in West Rosebud Creek below the Re-regulation Dam is 20 cfs, or lower when natural inflow conditions are less than 20 cfs

The Project is designed to provide diversion releases from the Mystic Lake flow line to West Rosebud Creek during normal and emergency conditions. The bypass valves will be sized to provide a minimum of 20 cfs below the Re-regulation Dam for all Mystic Lake water levels above 7612.5 ft amsl. In addition to meeting the minimum regulatory criteria below the Re-regulation Dam (20 cfs), the valves are capable of providing flows below the Re-regulation Dam similar to the average monthly measured flows below the Re-regulation Dam less the average Mystic Lake spillway contribution (see Table 3.2-1).

Installation of an isolation valve in the flow line will be required to isolate the conduit during a runaway pipeline condition. The isolation valve will be chosen to meet the following criteria:

- Normal operating velocity below 20 ft/sec

- Maximum system head loss of 5 ft or less

- Capable of automatic closure during a runaway pipeline condition

2.2 Constraints

Due to the fact that all of the components for this project must be flown to the construction site by some form of commercially available helicopter, component weight becomes a significant limiting design constraint in determining the proposed design configuration. The design team spent considerable effort weighing the pros and cons of using lighter weight components in larger quantities, versus smaller numbers of heavier components. Available space was also a significant design constraint for several reasons. First, the location where the new flow shutoff system must be sited is a small bench cut in the side of a steep rock mountain slope. As the structure becomes larger, the ability to maintain a reduced profile to protect it from rockfalls becomes increasingly difficult. A larger structure lacks the advantage of a compact profile where more falling rocks would pass over the structure rather than directly impact it. Also as structural columns become taller, and beams become longer, the required strength and therefore the required weight (and expense) grows quickly. For these reasons, there is a strong impetus to maintain as small a profile as is reasonably possible. This requirement and the logistics of weight and space minimization had a strong influence on the course of the design.

A second primary design requirement was for continuous adequacy of flow. There are two predominant flow constraints that were adhered to. One was the requirement for the bypass valve to be capable of delivering the minimum bypass reach flow rate of 3 - 10 cfs under all operational lake levels from 7612 ft to 7673.5 ft amsl. Secondly, the bypass valve must be capable of delivering a minimum flow of at least 20 cfs of flow at the minimum operational lake elevation of 7612 ft amsl. These two constraints alone present substantial challenges to the successful selection of one or more valves that will perform satisfactorily over a relatively wide range of heads and discharge requirements. Ultimately, the choice was made to use two separate bypass valves, one designed to throttle and operate on a near continuous basis, the other designed to open fully on occasion to provide adequate flows to the bypass channel and below the Re-regulation Dam in the case of an emergency; such as a flow line breach, but otherwise remain closed.

2.3 Main Shutoff Valve Configuration

Based on weight considerations, the desire for low head loss, high reliability, longevity, no operational requirement to throttle the valve (the main shutoff valve will always operate either fully open or fully closed), and the relatively infrequent need to actuate the valve, metal-seated ball valves emerged as attractive candidates. Selecting the largest practical valve ensures the lowest head loss, but this must be balanced against cost, complexity, reliability, and weight and space considerations. GEI examined the alternative of using two smaller valves in place of one larger valve, and also compared the cost of using smaller helicopters versus using the largest commercially available helicopter capable of lifting the largest single valve components it can handle. Valve diameters down to 36 inches were considered for single valve applications. The use of two 36-inch valves was also considered, but was subsequently rejected due to space limitations, added complexity, and the lack of appreciable cost savings when all the cost components were factored in.

Ultimately, we concluded that a single 42-inch diameter ball valve was found to provide an appropriate combination of cost, conservation of limited space, and functional reliability without introducing appreciable head loss.

2.4 Overhead Bridge Crane

The relatively high weight of the requisite shutoff valve requires delivering the valve to the construction site in pieces, and then performing the final assembly at the installation site. The design includes an overhead bridge crane capable of lifting and accurately placing a 15-ton load, to facilitate final valve assembly and installation. This overhead bridge crane installation is being designed to be removable, so that the choice of whether to take it down and store it rather than leave it permanently erected above the valve house can be decided at some future date.

Since the bridge crane will be used to lift valve components rolled into position using the service railroad, the bridge crane must pass directly above the service railroad in the vicinity of the location where the valves will be installed. Although a fully enclosed bridge crane would provide the best alternative for maintaining a permanent bridge crane at the site, height limitations for the valve house dictated an external bridge crane that would be exposed to the elements. The crane may also be used to help set structural members and timbers for the valve house. However, upon completion of the valve house, the bridge crane will not be able to move or remove hydraulic components without partial removal of the roof members and metal shell of the building. Provisions in the building design will allow for field dismantling of the roof by removing timbers and metal cross beams, and the sheet metal sides will be removable at critical locations to facilitate valve removal.

2.5 Rock Deflection Systems, Nets, Trips, and Barriers

Studies and analyses were undertaken to evaluate rockslide paths, expected potential energies and anticipated weights of potential rockslides based on the topography and geology in the area above the valve house. The use of steel nets, deflectors, and specially designed retaining walls are also being considered as preventive measures intended to shield the valve house as much as possible in an attempt to avoid it being directly impacted by falling rock. As described elsewhere in this report, the valve house is designed to deflect and to withstand limited rock impact forces imparted by falling rocks. A redundant series of steps have been laid to ensure that 1) the probability of a rockslide occurring directly over the new valve house is reduced, 2) the profile of the valve house is held low to decrease the probability of direct impacts, and 3) the valve house is designed to withstand limited direct hits from rock fall impacts.

2.6 Valve House

The size and strength of the valve house and the reinforcement of the roof is constrained by the load requirements of the overhead bridge crane, by clearance and access requirements of valve and equipment components housed inside the valve house, by the size and layout of the final valve selection, and the results of the rockfall analyses that were used to determine impact forces from falling rock. To the extent possible and as described elsewhere in this report, focus is being directed to design protective systems that will deflect falling debris away from the structure.

Electrical power is available at the existing valve house and will be provided in the new facility. The valve house will be insulated, heated and lighted as needed. Auxiliary power systems for the purpose of providing backup power to operate the valves and maintain controller power will be provided by battery backup systems in conjunction with a pressurized pneumatic or hydraulic reservoir capable of closing the shutoff valve (as well as opening the emergency bypass valve) under full head and runaway flow conditions.

2.7 Bypass Valves

The existing fish valve was designed to enable it to release a minimum of 3 cfs September through May at low lake levels, and a minimum of 10 cfs in summer months (June - August) when lake levels are typically much higher. The proposed flow restoration facility must, in addition to being capable of releasing sufficient bypass flows, be capable of releasing sufficient flows below the Re-regulation Dam at all operational lake elevations. Hence, a larger valve is needed to allow a minimum flow of 20 cfs to be released below the Re-regulation Dam at the minimum lake elevation, than would be needed for normal operation with a minimum required bypass flow of 3 cfs. The rock tunnel exit is at an elevation of 7,608 ft amsl. Based on existing drawings of the Mystic Lake facilities, the invert of the rock tunnel intake is at 7608.75 ft amsl. The tunnel itself slopes at 0.4 percent for approximately 1000 ft before it exits at the location of the bypass. Based on this information, it was assumed that the invert of the flow line at the bypass location was approximately at 7604.8 ft amsl. To facilitate crossing underneath the railroad tracks at the bypass location, the bypass valves will be constructed with centerlines below the flow line invert. At the minimum operational reservoir level of 7,612.5 ft amsl, there will be approximately between 10.5 and 11.5 ft of static head on the bypass valves. To provide minimum allowable flows below the Re-regulation Dam, any new bypass valve must deliver at least 20 cfs under the same minimum head conditions. The existing fish valve was also observed to experience cavitation under high head operating conditions and partially open settings. Hence, the constraints on flow release capacities and available head are quite variable, and require a range of capabilities to produce the wide range of flow conditions required during normal operations as well as emergencies.

2.8 Venting

A combination air\vacuum valve is included in the design on the downstream side of the flow line isolation valve to avoid excessive pressure differentials between the inside and the outside of the flow line during rapid isolation valve closures. The valve will also provide for the release of displaced air within the flow line as the conduit is refilled, and remove accumulated air bubbles that will collect downstream of the isolation valve due to the reduction in pipe diameter at the valve. The venting system is similar to the vent located immediately downstream of the existing inverted siphon.

2.9 Flow Line Filling

Provisions are included to facilitate slow controlled filling of the flow line to enable the flow line to be slowly refilled prior to fully opening the isolation valves. For this purpose, the design includes a small bypass valve and required plumbing to allow the flow line and penstock to be fully filled under controlled conditions prior to re-opening the main shutoff valve.

2.10 Instrumentation

The controls system and components designed to automate the detection of a flow line breach and to enhance metering of release flows for bypass flow augmentation is underway. The required instrumentation, controls, flow meters, power supplies, and other ancillary equipment will depend on the final configuration of the system and therefore, have not been evaluated in detail at this time.

2.11 Structural Analysis

Analyses have been conducted to assess the expected impact forces may be encountered due to rock impacts. The predominant structural strength constraints are being dictated by valve size and weight requirements, dimensions and exposure of the valve house and ancillary structures, the adequacy in resisting pressure and momentum forces encountered in closing the valves against runaway flow conditions, and the direct resistance and deflection of impacting rock from rockfall events.

2.12 Geological Analyses

Geologic analyses performed in support of the design included review of available geologic mapping and evaluation of rockfall potential.

2.13 Constructability

As indicated elsewhere in this report, care is being taken to ensure that the structure can be assembled using procedures that are cost effective, feasible, practical, and that reduce the time the plant must be off line to accomplish construction. We understand that various contractors may have alternative methods to effect the construction of this facility; we present this section to suggest one possible approach, and to demonstrate the feasibility of construction. There are several limiting constraints that must be considered during the construction phase of this project. These constraints are described below.

Due to the high cost of staging a heavy lift helicopter, and the limited availability and difficulty in scheduling these large and expensive aircraft, the components that must be flown to the site should be pre-delivered and ready to fly to the construction site prior to arrival of the heavy lift air crane. All of the valve transitions, connections, pedestals, thrust blocks and all other support equipment must be in place before the heavy components are delivered. All footings and foundations would be poured and the structural steel members would be delivered by medium lift helicopter (such as a Bell 204B). The service railroad trestle would have to be rebuilt and the tracks rerouted to provide additional clearance and be capable of supporting the weight of a fully assembled 15-ton valve. Upon completion of the rerouted tracks and trestle at the site of the existing valve house, the overhead bridge crane could be erected over the new valve house foundation prior to delivery of the valve components.

Once the bridge crane installation and trestle modifications are completed, the head gate to the flow line (located at the upstream end of the rock tunnel to Mystic Lake) could be closed, and the flow line would be drained and severed at the valve house. The footings, foundations, pedestals and thrust blocks would be formed up and poured. Flexible sleeve type couplings would be fitted to the ends of the severed flow line.

After the concrete work is completed, the various valve sub assemblies and structural steel would be flown to the site by a heavy lift helicopter (Sikorsky S64 Aircrane) and landed upon suitably designed service railroad carts. These components would be rolled into place, assembled, and erected using the overhead bridge crane. Each cart could be pushed uphill of the newly widened trestle as their components were removed and bolted to the valve under assembly. Once the main shutoff valve was fully assembled, it would be moved into position and bolted into place using the bridge crane.

All additional valve assemblies, transitions, and bypass valves would also be inserted using the overhead bridge crane. After the valve and piping components have been positioned and bolted into place, the flow line would be refilled and the plant brought back on line. This approach minimizes the amount of time the plant is off line by staging all of the necessary equipment and building as much of the system as possible prior to severing the flow line to perform the final installations.

The smaller valves actuators, control systems, and the structural steel for the valve house would be erected in a similar manner. After completion of the structures, the bridge crane could either be left in place (although exposed and vulnerable to slide damage), or removed and stored until needed in the future.

Section 3 – Hydraulic Operations and Design

3.1 Introduction

The primary reason for introducing a flow shutoff system into the flow line at the Mystic Lake facility is to enable effective closure of the flow line and rapid and controlled restoration of flow into the bypass channel and below the Re-regulation Dam in the event of a flow line breach. Consequently, a few constraints dictate how this should best be accomplished. These constraints include weight limitations, space for construction, the legal minimum flows required by the Project in the bypass and below the Re-regulation Dam throughout the year, and the fact that the Project operates over a wide range of discharges and pool elevations. The design has focused on balancing these constraints and provides greater available operational flexibility to respond to unanticipated events to maximize safety, and to quickly and automatically restore existing flows below the Re-regulation Dam to reasonable levels.

3.2 Bypass Valve Selection

A wide range of head conditions and required release flows are encountered in the normal operation of the Mystic Lake facility. The valves were sized based on an assumption that there would be 4 ft of head at minimum pool. Later, it was determined that there would be an additional 4 to 8 ft of head at minimum pool. Therefore, the ball valve may be slightly over-sized (Table 3.2-1). A somewhat smaller ball valve may be recommended as the design is refined.

To achieve the required bypass discharges for both normal and emergency operations under a wide range of head differentials, it is either necessary to select one valve that is generally operated at relatively high degrees of throttling most of the time and occasionally opened under emergency conditions, or select two valves to perform the two required functions. After evaluating the choices and determining the advantages and disadvantages of each (including the need to use emergency power systems to actuate the controls), the selection of two separate valves designed to perform two different functions was made. By selecting two different valves, we were able to size one valve to serve as a normal bypass valve that can be continuously throttled and that is also capable of dissipating high energies under high head conditions. This valve (formerly known as the fish valve) has a design requirement to be able to release between 3 and 10 cfs at any reservoir pool elevation, to operate effectively over a wide range of valve openings, and designed to be adjusted continuously by remote controlled actuation over many years of operation. Such a valve must be wear resistant, designed to avoid cavitation at high differential pressures, be able to accurately meter flows, and be able to reliably pass any sediment and entrained debris during service.

The second valve (the emergency bypass valve) was sized to deliver adequate flows below the Re-regulation Dam when opened completely. Since it is possible that no power or telemetry signals will be available if a flow line or penstock breach occurs due to a powerful earthquake or for some other reason that causes power and communication links between the valve and the remote control interface to be lost, this valve must deliver an appropriate discharge when fully opened, based only on the existing lake elevation at the time it is opened. Assuming this is a likely possibility, the valve must be sized such that when fully opened, it will deliver a discharge that attempts to match the throughput of the power plant to the extent possible.

Table 3.2-1 presents the average monthly flows below the Re-regulation Dam and average monthly spill over Mystic Lake Dam, the bypass flows (difference between the average monthly spill over Mystic Lake Dam and flow at the Re-regulation Dam), minimum flow requirements, the average monthly pool elevation, the projected maximum discharge of the bypass (16-inch valve) and emergency bypass valve (20-inch valve) under average monthly conditions, and combined discharge of both valves. The results of this design demonstrate the capacity of the maximum combined flow of both valves (last column) closely match the average flows below the Re-regulation Dam.

Table 3.2-1. Compass Bypass Curve of Valve Discharges. Bypass flows (cfs) reflect flows below the Re-regulation Dam minus spill at Mystic Lake Dam.

Month	Below Re-regulation Dam Average Monthly Flow (cfs)	Average Monthly Spill Over Mystic Dam (cfs)	Bypass Flows (cfs)	Required FERC Bypass Flow (Upper Weir) (cfs)	Required FERC (Below Re-regulation Dam) (cfs)	Average Mystic Lake Water Level (ft)	16" Control Valve Maximum Outflow (cfs)	20" Ball Valve Maximum Outflow (cfs)	Combined Valve Maximum Outflow (cfs)
Jan	66	5.7	60	3	20	7632.9	21	67	88
Feb	55	5.8	49	3	20	7626.5	19	60	79
Mar	48	5.0	43	3	20	7619.8	16	51	67
Apr	40	6.5	33	3	20	7614.8	14	42	56
May	73	15.4	58	3	20	7620.1	16	52	68
Jun	209	85.6	123	10	20	7652.3	27	87	114
Jul	335	173.0	162	10	20	7671.9	32	102	134
Aug	192	46.2	145	10	20	7671.7	32	102	134
Sep	118	17.6	100	3	20	7668.4	31	100	131
Oct	102	14.2	87	3	20	7662.0	29	94	123
Nov	97	9.0	88	3	20	7652.4	27	87	114
Dec	77	6.5	70	3	20	7642.4	24	78	102

3.3 Selection of Flow Line Isolation Valve

As previously discussed, site space constraints dictate use of a single flow line isolation valve. Several potential valve types were evaluated for this purpose, including ball valves, butterfly valves, and various cone valve configurations. A ball valve was selected because, when fully opened, it does not have moving parts within the flow line and can be closed under the severe flow conditions induced during runaway flows. Also, transient pressures, which may develop during instantaneous butterfly valve closure with stem failure, are not possible with ball valves.

Weight restrictions for components of the bypass system required a reduction in valve size from the 57-inch diameter of the flow line. GEI analyzed the effects of reducing the flow line diameter upstream and downstream of the flow line isolation valve on overall system head. We looked at 100-foot long flow line section without the bypass valve and with three different inline valve sizes. Because existing drawings show several different diameters of pipe at the transition from the rock tunnel to the flow line, we assumed the flow line was a 60-inch diameter steel pipe at the bypass location. Table 3.3-1 summarizes the system head losses for no valve, and for 36-, 42-, and 48-inch diameter isolation valve options. Table 3.3-2 summarizes velocities through the bypass configuration compared to the baseline 60-inch diameter assumed flow line.

Table 3.3-1. Flow vs. Head Loss in Flow Line

Flow (cfs)	System Head Loss (ft)			
	Baseline	36" Valve	42" Valve	48" Valve
50	0.0	0.4	0.2	0.1
100	0.1	1.7	0.8	0.3
120	0.2	2.4	1.1	0.5
140	0.2	3.3	1.5	0.7
160	0.3	4.4	1.9	0.9
180	0.4	5.5	2.4	1.1
200	0.5	6.8	3.0	1.3

Table 3.3-2. Flow vs. Velocity in Flow Line

Flow (cfs)	Velocity in Valve (ft/sec)			
	Baseline	36" Valve	42" Valve	48" Valve
50	2.5	7.1	5.2	4.0
100	5.1	14.1	10.4	8.0
120	6.1	17.0	12.5	9.5
140	7.1	19.8	14.6	11.1
160	8.1	22.6	16.6	12.7
180	9.2	25.5	18.7	14.3
200	10.2	28.3	20.8	15.9

The 42-inch diameter valve was chosen because it met the head loss design criteria with a maximum of 2.5 foot system head loss for the range of possible operational flows and velocities were maintained under 20 ft/sec up to almost 200 cfs.

3.4 Air/Vacuum Valve Design

During closure of the flow line isolation valve, a large negative pressure will develop in the conduit downstream of the shutoff valve. For larger diameter conduits with small t/D (diameter/thickness) ratios, collapse potential of the pipeline is probable as water exiting the system creates a vacuum pressure at the upstream end. The maximum recommended vacuum pressure for a 57-inch-diameter conduit with a 0.25-inch thickness is approximately 1.4 psi. Further, the 0.4 percent slope of the flow line allows a maximum gravity flow of the conduit of about 200 cfs, equating to 12,000 cfm of displaced air. Preliminarily, a 14-inch air vacuum valve was selected to meet the performance requirements.

Refilling the flow line and penstock after dewatering will require approximately 1.6 MG of water. Table 3.4-1 summarizes the conduit filling times associated with a variety of filling rates. We recommend a filling rate between 10 and 50 cfs. The selected air/vacuum valve will also accommodate these flows.

Table 3.4-1. Summary of Conduit Filling Rates vs. Fill Time

Pipe Filling Rate (cfs)	Fill Time (hr)
10	5.79
15	3.86
25	2.32
50	1.16
75	0.77
100	0.58
125	0.46
150	0.39

3.5 Bypass Valve Design

Two different valve types were chosen to meet the Project design criteria for providing discharges to West Rosebud Creek. For normal releases, a 16-inch globe style control valve was selected because it can be set to maintain constant discharges independent of reservoir water levels. The valve controls will be designed to compute flow rates based on differential pressure across the valve and operate pilot solenoids on the valve to regulate flow. The valve can provide 3 to 10 cfs normal discharge flows for all reservoir water levels to meet regulatory requirements but has a capacity of 32 cfs at full reservoir.

This valve has a minimum ‘cracking pressure’ of 5 psi (11.5 ft static head) at the valve centerline, which is required to open the valve. Currently, the centerline of this valve is set at 7601.0 ft amsl to accommodate reservoir discharges down to a reservoir water surface elevation of 7612.5 ft amsl. Debris along the bottom of the flow line could cause operational problems for this valve, so the flow line will be tapped from the side for the bypass. A 16-

inch diameter knife gate valve will be installed upstream of the control valve to isolate the valve for maintenance.

A 20-inch diameter metal seated ball valve with hydraulic actuator was chosen to provide emergency bypass flows of at least 20 cfs during the range of reservoir levels identified in the design criteria. Fully opened, this valve has a capacity of up to 110 cfs at full reservoir. At this flow rate, the velocity will approach 50 ft/sec. All piping and fittings for this valve should be stainless steel to accommodate these relatively high velocities. This valve is not designed for accurately metering flows while partially closed but may be throttled for normal creek return flows when the reservoir is below 7612.5 ft amsl. A 20-inch diameter knife gate isolation valve will be installed upstream of this valve.

Table 3.5-1 summarizes the discharge capacities of the two bypass valves individually and the combined discharge potential of both valves.

Table 3.5-1. Bypass Valve Capacities vs. Reservoir Head

Reservoir Water Level (ft)	**Fully Opened Valve Discharge (cfs)**		
	16-in Globe	**20-in Ball**	**Combined**
7611	-	36.8	36.8
7612	-	38.8	38.8
7612.5	12.8	39.7	52.5
7615	14.2	44.2	58.4
7620	16.5	52.0	68.5
7625	18.5	58.8	77.3
7630	20.4	64.9	85.2
7635	22.1	70.4	92.5
7640	23.6	75.6	99.2
7645	25.1	80.4	105.5
7650	26.5	84.9	111.4
7655	27.8	89.2	117.0
7660	29.1	93.4	122.4
7665	30.3	97.3	127.6
7670	31.4	101.1	132.5
7671	31.5	101.8	133.4
7673.5	32.2	103.6	135.8

Section 4 – Rockfall Evaluation

4.1 Introduction

GEI conducted a preliminary rockfall evaluation to investigate the feasibility of protecting the new valves from rockfall events. This preliminary evaluation was based on photographs of the site, USGS topographic mapping, and computer simulation of potential rockfall events. No site visit or geologic mapping was conducted in support of this evaluation.

4.2 Site Conditions

The topography of the site is relatively steep, mountainous terrain. The elevation of the study area ranges from approximately 7,000 to 8,000 ft amsl. The northeast-facing slope above the outlet tunnel has an average angle varying from 30 to 60 degrees from horizontal with an approximately 20-foot near-vertical cut immediately above the outlet tunnel. Outcropping granitic intrusive rocks composed of quartz monzonite and aplites are exposed across the entire slope (MBMG, 2001). A few lower-sloped areas contain some light vegetation and small trees.

Rockfall events have occurred near the existing pipeline at the proposed bypass valve location, as evidenced by a dented rail. The rock size and source area are not precisely known.

4.3 Rockfall Hazard

Some subjective evaluation of potential source areas was necessary to establish probable rock trajectories and rock sizes likely to be produced by the source area. Rockfall initiating from various outcrop locations were considered in our analysis for areas within the projected draw angle that appear to be topographically possible. Naturally occurring rockfall activity on the slope appears to be possible from any outcrop location.

Detailed studies of the existing rock slope and accumulated debris with respect to statistically probable rock sizes and the probability of rockfall events were not performed during this project phase. While rocks up to 10 ft in diameter are in the streambed below the structure, most of the rocks, which appear to be present on the slope above the structure, are 2 to 5 ft in diameter. Based on these observations, rockfall analyses included rocks ranging from 1 foot to 10 ft in diameter.

4.4 Rockfall Simulation

To estimate the probabilistic trajectories of falling rocks, the critical profile of the site was modeled using the Colorado Rockfall Simulation program, Version 4.0 (CRSP). CRSP is a widely accepted program for use in evaluating rockfall behavior. Information provided by the program includes rock bounce height, rock velocity, and kinetic energy. The model was developed using published 2-foot topographic base maps.

One profile was chosen to represent likely critical trajectory. 1-, 2-, 3-, 5-, 7-, and 10-foot – diameter rocks were analyzed for this profile (Tables 4.4-1 through 4.4-6). The starting zones for the rocks allowed for gravity initiation, which we believe is the likely method of the rocks that initiate from the outcropping gneiss. Slope characteristics were determined based on photographs from a distance of the entire site and the topographic base maps. Additional refinement was conducted on these profiles, reflective of our experience in rockfall analysis. Analysis points were placed at critical locations on the profile to determine design criteria for mitigation at a favorable location above the valve location. A summary of the CRSP results is provided below. Tables 4.4-1 through 4.4-6 summarize the bounce heights and impact energies for each of the analysis points.

Analyses indicate that in general, bounce heights, velocities, and consequently impact energies, will be large in the area of interest uphill from the valve. Impact energies of approximately 330,000 to 1,870,000 foot-pounds are possible at some locations on the slope for a 3- and 5-foot-diameter rock, respectively. At locations where mitigation is likely to be deployed in the slopes above critical structures, the potential impact energies for rocks sized from 1 to 10 ft in diameter range from approximately 1,800 to 13,300,000 foot-pounds.

Table 4.4-1. Rockfall simulation with 1-ft rock.

Analysis Point	Source Area	Velocity (ft/sec)	Bounce Height (ft)	Energy (ft-lbs)
1.Rockfall Shed (upslope)	A	Average: 70.0	Average: 4.4	Average: 8,575
		Maximum: 86.5	Maximum: 12.1	Maximum: 12,249
2.Rockfall Shed (downslope)	A	Average: 65.0	Average: 13.6	Average: 7,539
		Maximum: 90.6	Maximum: 18.9	Maximum: 13,375
1.Rockfall Shed (upslope)	B	Average: 64.9	Average: 4.2	Average: 7,296
		Maximum: 86.6	Maximum: 12.2	Maximum: 12,300
2.Rockfall Shed (downslope)	B	Average: 62.0	Average: 13.6	Average: 6,785
		Maximum: 82.8	Maximum: 19.5	Maximum: 11,112
1.Rockfall Shed (upslope)	C	Average: 56.3	Average: 3.2	Average: 5,532
		Maximum:66.3	Maximum: 8.5	Maximum: 7,201
2.Rockfall Shed (downslope)	C	Average: 55.1	Average: 13.1	Average: 5,373
		Maximum: 71.8	Maximum: 17.2	Maximum: 8,030
1.Rockfall Shed (upslope)	D	Average: 50.3	Average: 2.1	Average: 4,411
		Maximum: 63.5	Maximum: 7.3	Maximum: 6,395
2.Rockfall Shed (downslope)	D	Average: 48.6	Average: 13.2	Average: 4,235
		Maximum: 62.8	Maximum: 16.9	Maximum: 6,319
1.Rockfall Shed (upslope)	E	Average: 30.4	Average: 1.0	Average: 1,760
		Maximum: 46.4	Maximum: 4.6	Maximum: 3,717
2.Rockfall Shed (downslope)	E	Average: 35.3	Average: 12.3	Average: 2,250
		Maximum: 49.5	Maximum: 15.11	Maximum: 4,247

Table 4.4-2. Rockfall simulation with 2-ft rock.

Analysis Point	Source Area	Velocity (ft/sec)	Bounce Height (ft)	Energy (ft-lbs)
1.Rockfall Shed (upslope)	A	Average: 69.3	Average: 4.41	Average: 66,496
		Maximum: 86.3	Maximum: 10.9	Maximum: 100,758
2.Rockfall Shed (downslope)	A	Average: 64.6	Average: 13.7	Average: 59,476
		Maximum: 83.8	Maximum: 19.9	Maximum: 90,812
1.Rockfall Shed (upslope)	B	Average: 64.4	Average: 4.1	Average: 57,580
		Maximum: 79.3	Maximum: 12.5	Maximum: 83,831
2.Rockfall Shed (downslope)	B	Average: 61.4	Average: 13.2	Average: 53,655
		Maximum: 82.1	Maximum: 18.5	Maximum: 86,364
1.Rockfall Shed (upslope)	C	Average: 58.0	Average: 3.3	Average: 46,576
		Maximum: 73.5	Maximum: 8.7	Maximum: 67,111
2.Rockfall Shed (downslope)	C	Average: 55.0	Average: 13.2	Average: 43,224
		Maximum: 71.2	Maximum: 16.7	Maximum: 66,775
1.Rockfall Shed (upslope)	D	Average: 51.0	Average: 2.0	Average: 35,808
		Maximum: 61.3	Maximum: 7.2	Maximum: 47,724
2.Rockfall Shed (downslope)	D	Average: 49.1	Average: 13.2	Average: 34,787
		Maximum: 62.3	Maximum: 16.8	Maximum: 50,272
1.Rockfall Shed (upslope)	E	Average: 30.6	Average: 0.8	Average: 14,244
		Maximum: 46.7	Maximum: 4.0	Maximum: 30,599
2.Rockfall Shed (downslope)	E	Average: 35.8	Average: 12.3	Average: 18,375
		Maximum: 49.5	Maximum: 15.5	Maximum: 34,035

Table 4.4-3. Rockfall simulation with 3-ft rock

Analysis Point	Source Area	Velocity (ft/sec)	Bounce Height (ft)	Energy (ft-lbs)
1.Rockfall Shed (upslope)	A	Average: 70.6	Average: 4.5	Average: 231,102
		Maximum: 82.2	Maximum: 11.8	Maximum: 329,659
2.Rockfall Shed (downslope)	A	Average: 64.4	Average: 13.7	Average: 200,683
		Maximum: 89.6	Maximum: 20.3	Maximum: 346,229
1.Rockfall Shed (upslope)	B	Average: 65.5	Average: 4.1	Average: 199,852
		Maximum: 81.2	Maximum: 12.4	Maximum: 281,765
2.Rockfall Shed (downslope)	B	Average: 60.8	Average: 13.8	Average: 177,533
		Maximum: 84.9	Maximum: 19.1	Maximum: 302,977
1.Rockfall Shed (upslope)	C	Average: 57.7	Average: 3.5	Average: 155,561
		Maximum: 73.2	Maximum: 10.3	Maximum: 221,761
2.Rockfall Shed (downslope)	C	Average: 55.1	Average: 13.2	Average: 145,329
		Maximum: 76.5	Maximum: 16.5	Maximum: 247,218
1.Rockfall Shed (upslope)	D	Average: 49.9	Average: 2.3	Average: 116,469
		Maximum: 66.5	Maximum: 7.4	Maximum: 185,148
2.Rockfall Shed (downslope)	D	Average: 48.8	Average: 13.2	Average: 113,512
		Maximum: 64.3	Maximum: 16.3	Maximum: 175,457
1.Rockfall Shed (upslope)	E	Average: 30.5	Average: 1.1	Average: 47,987
		Maximum: 47.7	Maximum: 4.7	Maximum: 100,403
2.Rockfall Shed (downslope)	E	Average: 34.9	Average: 12.5	Average: 59,637
		Maximum: 49.0	Maximum: 15.4	Maximum: 112,991

Table 4.4-4. Rockfall simulation with 5-ft rock.

Analysis Point	Source Area	Velocity (ft/sec)	Bounce Height (ft)	Energy (ft-lbs)
1.Rockfall Shed (upslope)	A	Average: 71.0	Average: 4.1	Average: 1,094,106
		Maximum: 94.3	Maximum: 12.4	Maximum: 1,788,279
2.Rockfall Shed (downslope)	A	Average: 64.8	Average: 14.2	Average: 943,297
		Maximum: 87.0	Maximum: 20.6	Maximum: 1,640,984
1.Rockfall Shed (upslope)	B	Average: 66.0	Average: 4.1	Average: 941,508
		Maximum: 83.6	Maximum: 9.2	Maximum: 1,416,893
2.Rockfall Shed (downslope)	B	Average: 61.4	Average: 13.4	Average: 841,297
		Maximum: 83.4	Maximum: 18.5	Maximum: 1,375,229
1.Rockfall Shed (upslope)	C	Average: 57.8	Average: 3.1	Average: 724,695
		Maximum: 72.4	Maximum: 9.7	Maximum: 1,010,216
2.Rockfall Shed (downslope)	C	Average: 55.7	Average: 13.2	Average: 687,217
		Maximum: 70.8	Maximum: 17.4	Maximum: 988,791
1.Rockfall Shed (upslope)	D	Average: 50.3	Average: 2.9	Average: 547,911
		Maximum: 63.4	Maximum: 6.7	Maximum: 792,908
2.Rockfall Shed (downslope)	D	Average: 49.2	Average: 13.8	Average: 536,491
		Maximum: 64.0	Maximum: 16.2	Maximum: 825,027
1.Rockfall Shed (upslope)	E	Average: 31.2	Average: 1.2	Average: 226,289
		Maximum: 50.1	Maximum: 4.8	Maximum: 477,625
2.Rockfall Shed (downslope)	E	Average: 35.2	Average: 12.5	Average: 277,584
		Maximum: 52.0	Maximum: 15.4	Maximum: 536,486

Table 4.4-5. Rockfall simulation with 7-ft rock.

Analysis Point	Source Area	Velocity (ft/sec)	Bounce Height (ft)	Energy (ft-lbs)
1.Rockfall Shed (upslope)	A	Average: 69.9	Average: 3.9	Average: 2,931,926
		Maximum: 90.6	Maximum: 16.8	Maximum: 4,589,590
2.Rockfall Shed (downslope)	A	Average: 66.2	Average: 13.8	Average: 2,672,571
		Maximum: 90.3	Maximum: 22.4	Maximum: 4,557,362
1.Rockfall Shed (upslope)	B	Average: 65.0	Average: 3.8	Average: 2,510,758
		Maximum: 79.9	Maximum: 10.5	Maximum: 3,717,563
2.Rockfall Shed (downslope)	B	Average: 60.2	Average: 13.6	Average: 2,209,303
		Maximum: 79.5	Maximum: 17.6	Maximum: 3,503,117
1.Rockfall Shed (upslope)	C	Average: 57.3	Average: 3.3	Average: 1,960,958
		Maximum: 71.1	Maximum: 9.0	Maximum: 2,745,674
2.Rockfall Shed (downslope)	C	Average: 55.1	Average: 13.4	Average: 1,856,217
		Maximum: 70.6	Maximum: 17.2	Maximum: 2,721,414
1.Rockfall Shed (upslope)	D	Average: 50.3	Average: 2.8	Average: 1,506,146
		Maximum: 63.4	Maximum: 6.9	Maximum: 2,193,485
2.Rockfall Shed (downslope)	D	Average: 64.8	Average: 13.1	Average: 1,473,408
		Maximum: 49.1	Maximum: 16.8	Maximum: 2,331,840
1.Rockfall Shed (upslope)	E	Average: 31.5	Average: 1.0	Average: 638,155
		Maximum: 47.9	Maximum: 5.1	Maximum: 1,229,824
2.Rockfall Shed (downslope)	E	Average: 35.1	Average: 12.7	Average: 761,400
		Maximum: 48.6	Maximum: 15.2	Maximum: 1,349,416

Table 4.4-6. Rockfall simulation with 10-ft rock.

Analysis Point	Source Area	Velocity (ft/sec)	Bounce Height (ft)	Energy (ft-lbs)
1.Rockfall Shed (upslope)	A	Average: 69.6	Average: 4.1	Average: 8,507,255
		Maximum: 90.8	Maximum: 13.0	Maximum: 13,320,174
2.Rockfall Shed (downslope)	A	Average: 66.0	Average: 13.7	Average: 7,752,237
		Maximum: 84.9	Maximum: 18.8	Maximum: 11,669,093
1.Rockfall Shed (upslope)	B	Average: 64.7	Average: 3.5	Average: 7,334,247
		Maximum: 82.8	Maximum: 11.2	Maximum: 11,458,649
2.Rockfall Shed (downslope)	B	Average: 61.1	Average: 13.5	Average: 6,669,151
		Maximum: 82.7	Maximum: 18.1	Maximum: 11,062,797
1.Rockfall Shed (upslope)	C	Average: 58.4	Average: 3.3	Average: 5,856,600
		Maximum: 73.7	Maximum: 9.3	Maximum: 8,464,427
2.Rockfall Shed (downslope)	C	Average: 55.1	Average: 13.2	Average: 5,378,038
		Maximum: 70.6	Maximum: 16.6	Maximum: 7,991,620
1.Rockfall Shed (upslope)	D	Average: 51.0	Average: 2.2	Average: 4,509,576
		Maximum: 63.6	Maximum: 6.9	Maximum: 6,678,434
2.Rockfall Shed (downslope)	D	Average: 49.4	Average: 13.2	Average: 4,338,222
		Maximum: 64.05	Maximum: 16.3	Maximum: 6,451,238
1.Rockfall Shed (upslope)	E	Average: 31.7	Average: 1.0	Average: 1,882,190
		Maximum: 49.9	Maximum: 3.7	Maximum: 3,924,296
2.Rockfall Shed (downslope)	E	Average: 35.4	Average: 12.6	Average: 2,264,431
		Maximum: 48.9	Maximum: 15.3	Maximum: 4,149,550

4.5 Rockfall Mitigation

Rockfall mitigation is generally divided into two categories, upslope stabilization and down slope protection. Upslope stabilization occurs at the source area and includes the following methods: scaling and trimming, rock bolts, cable lashing, anchored mesh and nets, shotcrete, and buttresses. Because of the narrow shape and possible limited size of the source area, these methods of mitigation for this site may be feasible and will be investigated in more detail with a site visit to evaluate upslope stability and identify potential threats.

Down slope protection methods may also be an appropriate means of mitigation. These include ditches or berms, catch fences and walls, deflectors, and rockfall catchment sheds. These methods vary in their effectiveness based on their ability to stop or deflect rocks with differing levels of impact energy. At this preliminary level of investigation, a heavily reinforced valve house was evaluated as a means to protect the bypass valve.

4.6 Rockfall Conclusions and Recommendations

Based on observations of site photographs and the results of the CRSP analysis, the existing pipeline is located within the area that would likely be affected by rockfall. The probability of impacting the pipeline is likely and mitigation should be considered for this site.

The choice of mitigation systems should be based on an evaluation of the cost and benefits related to each of the proposed solutions. One of the primary criteria in the choice of a system is the impact energy (size and velocity) of the rock for which it is designed.

The valve house shown in the 30% design drawings is designed to contain average 1-and 2-foot diameter rocks falling from the full height of the slope, and for average 3-foot diameter rocks falling from mid-slope. The structure will be able to resist impacts from larger rocks falling shorter distances as well. It is unclear at this time exactly what represents a typical rockfall event at the site, but a wide range of potential impact forces were evaluated.

The evaluation included simulation of up to 10-foot-diameter rocks. Rock fragments greater than those evaluated, as well as comprising multiple rocks, are possible at this location. The impact energy of very large rocks may exceed the yield strength of the valve house, but the primary intent of this structure is to protect the valves and ancillary equipment from a rock fall impact and ensure that they continue to function. Complete protection from all rockfall hazards was not judged to be practical. The proposed design is intended to provide reasonably reliable protection to the flow line shutoff and stream restoration facilities.

Section 5 - Structural Considerations

5.1 General Structural Considerations

The proposed protective structure to be built around the valves is a structural frame consisting of a series of sloped steel wide flange beams that will be supported by steel columns on the downslope side and be anchored into the rock face on the upslope side. Intermediate cross beams will be placed between these main beams. Timbers placed over the main and cross beams will enclose the structure and provide additional protection from falling rocks. The sides of the structure will be covered with sheet metal to facilitate heating the structure.

The protective structure is designed to withstand limited impact loads from rock slides. These limitations include:

- The pipeline will be protected against rocks of sizes and sources stated in Section 4.
- The structure will be designed to withstand impact loads from the average, but not the maximum, bounce heights associated with the falling rocks.
- The columns and the cross beams near the columns will not be designed to resist the significantly larger impact loads expected at the down-slope end of the structure. Rocks impacting at this location are not expected to have a trajectory that will intersect with the pipeline and valves.
- Yielding of the structural members should be expected for impacts greater than the design loads. Once completed, the protective structure will need to be inspected on a regular basis. Damaged structural members should be repaired or replaced as soon as possible after they are discovered.

The approximate dimensions of the building are 45 ft long by 22 ft wide. The building will have an interior ceiling height of approximately 12.5 ft near the railroad tracks that will slope upward towards the rock face at about 25 degrees.

Section 6 References

Montana Bureau of Mines and Geology (MBMG), 2001, *Preliminary Geologic Map of the Red Lodge 30”x60” Quadrangle, South-Central Montana*, Open File Report MBMG 423.