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Technical Memorandum No. QY-2016-8311-1

Truckee Canal Corrective Action Study

Newlands Project, Nevada
Mid-Pacific Region

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U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
Denver, Colorado

July 2017

Mission Statements

The U.S. Department of the Interior protects America's natural resources and heritage, honors our cultures and tribal communities, and supplies the energy to power our future.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

Cover photo: Derby Diversion Dam

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BUREAU OF RECLAMATION
Technical Service Center, Denver, Colorado
Geotechnical Engineering Group, 85831100

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Newlands Project, Nevada
Mid-Pacific Region

P.E.
Prepared: Chris Ellis, P.E.
Civil Engineer, Water Conveyance Group 85831100

6/21/17
Date

Prepared: Michelle Norris
Civil Engineer, Water Conveyance Group 85814000

6/21/2017
Date

Prepared: James Anders
Economist, Economics, Planning and Technical Communications 85827000

6.21.17
Date

Checked: Paul Craig, P.E.
Civil Engineer, Geotechnical Engineering Group 1 85831400

6/23/17
Date

Technical Approval: Ryan J. Woodruff, P.E.
Group Manager, Geotechnical Engineering Group 1 85831100

6/21/17
Date

Peer Review: Tim Brown, P.E.
Group Manager, Civil Structures Group 85815000

6/21/17
Date

REVISIONS					
Date	Description	Prepared	Checked	Technical Approval	Peer Review

Project Team Concurrence

This Technical Memorandum (TM) summarizes findings from a risk reduction analysis meeting that was held in May 2016 at the Bureau of Reclamation's (Reclamation) Technical Service Center (TSC) in Denver, Colorado. A Risk Estimating Team (RET) was convened to evaluate the potential risk reduction provided by the corrective action study alternatives described herein. Method for estimating and portraying the risk estimates specifically for the Truckee Canal and guidelines for decision making were described in a report titled: *Proposed Risk Analysis Process for the Truckee Canal, Decision Document and Technical Report of Findings*, dated July 2014 [4]. The risk reduction analysis utilized the methods outlined in the above document.

Members of the RET at the TSC accept the technical findings and conclusions presented in this TM. The signatures below indicate that area office and regional RET members and management have reviewed this report and concur with the findings presented herein.

Lee H. Berget, Lahontan Basin Area Office	Terri A. Edwards, Lahontan Basin Area Office
Jack Worsley, Lahontan Basin Area Office	Nadira Kabir, Lahontan Basin Area Office
7/10/7 Steven Melavic, Mid-Pacific Region MARK MORBERG, ACTING RE	7/10/17 Fred Holtz, Mid-Pacific Regional Office Holz

Abbreviations and Acronyms

1D	one-dimensional
2D	two-dimensional
AF	Acre-feet
AFP	annualized failure probability
ATP	Ability to pay
BEP	backwards erosion piping
CAS	Corrective Action Study
CLSM	controlled low strength concrete
CLP	Carson Lake and Pasture
CMP	corrugated metal pipe
CPT	cone penetration test
DEC	Design, Estimating, and Construction
DV	depth of flooding times flood velocity
EANB	Equivalent Annual Net Benefit
EIA	US Energy Information Administration
FBA	Farm budget analysis
ft	feet
ft^3/s	cubic feet per second (also abbreviated cfs)
FWR	Fallon Reservation Wetlands
GIS	Geographic Information System
GWh	Gigawatt hours
HEC-RAS	Hydrologic Engineering Centers River Analysis System
ICE	Intercontinental Exchange
IE	internal erosion
LBAO	Lahontan Basin Area Office
mi^2	square miles
M&I	Municipal and industrial
MP	Mid-Pacific (Region)
mp	milepost
MWh	Megawatt hours
NAVD88	North American Vertical Datum 1988
NFI	Net farm income
NOAA	National Oceanic and Atmospheric Administration
NSP	Nevada State Parks
NV	Nevada
O&M	operation and maintenance
OCAP	Operating Criteria and Procedures
PAR	population at risk

PFM	Potential Failure Mode
P&Gs	Economic and Environmental Principles and Guidelines for Water and Related Land Resources Implementation Studies
POA	Period of Analysis
PSHA	probabilistic seismic hazard analysis
Reclamation	Bureau of Reclamation
RET	risk estimating team
RPA	Revealed Preference Approach
SH	State highway
SOP	Standard Operating Procedures
SPT	standard penetration test
SWMP	Storm Water Management Plan
TC	Truckee Canal
TCID	Truckee-Carson Irrigation District
TM	Technical Memorandum
TSC	Technical Service Center
UNCE	Nevada Cooperate Extension
USCS	Unified Soil Classification System
USFWS	U.S. Fish & Wildlife Service
WTP	Willingness to pay
W&W	Wetlands and wildlife

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Technical Memorandum No. QY-2016-8311-1

Truckee Canal Updated Risk Analysis

Executive Summary

Purpose

The purpose of this Technical Memorandum (TM) is to document corrective action alternatives developed to address the identified risks posed by the Truckee Canal. Findings from a 2014 risk analysis study are summarized in a report titled; "*Updated Risk Analysis – Truckee Canal, Issue Evaluation and Technical Report of Findings*", dated June, 2015 [1][1]. Corrective action alternatives have been developed to improve the canal and its infrastructure to safely convey diversions from the Truckee River to Lahontan Reservoir to support the Newlands Project water needs.

Background

The Truckee Canal was constructed between 1903 and 1905 as part of the Newlands Project and is among the Bureau of Reclamation's (Reclamation) oldest structures. Water is diverted into the Truckee Canal from the Truckee River at the Derby Diversion Dam, about 20 miles east of Reno, Nevada (NV). The canal is about 31 miles long and discharges into Lahontan Reservoir, at the left abutment of Lahontan Dam. Truckee Canal flows are used to deliver irrigation water to agricultural lands along the length of the canal and to supplement inflows into Lahontan Reservoir from the Carson River for irrigation in the Carson Division.

On January 5, 2008, a portion of the canal embankment failed, causing flooding and property damage within Fernley, NV. Following the failure, numerous studies were completed to evaluate the cause, to evaluate the risk of future failures, and to develop feasible alternatives for improving the safety of the Truckee Canal. The failure was believed to result from internal erosion, which resulted from animal burrows in the canal embankment, combined with a rapid increase in the canal diversions in order to capture storm floodwaters in the Truckee River. Following the canal failure, a flow restriction of 350 ft^3/s was established through a court order. Stage-level restrictions were established by Reclamation in June 2009 at four staff gauge locations within the Fernley Reach, corresponding to the 350-ft^3/s flow rate. The term "stage level," as used in this report, is defined as the "unchecked" water surface level along the length of the

[1] Bracketed numbers indicate a numbered reference at the end of this report.

canal for a given inflow from the Derby Diversion Dam. Checking is defined as artificially increasing the water surface within the canal for a given flow rate to improve irrigation deliveries. This stage-level restriction was intended as a short-term (1 to 5 years) restriction until permanent structural improvements could be made to the Truckee Canal.

In 2014 the Lahontan Basin Area Office (LBAO) requested the Technical Service Center (TSC) to update the baseline risk estimates that were made following the January 2008 Truckee Canal failure and then use the findings to assist in developing an updated flow/stage level restriction and long-term risk reduction plan. Guidelines generally adopted for dam safety evaluations were used to define regions on the matrix (Figure II-1) that indicate tolerable long-term risk levels, tolerable short-term risk levels, and unacceptable risk levels for potential failure modes that can occur during normal operations. Current design standards were used to define the regions on the matrix for tolerable risks for hydrologic and seismic potential failure modes. Depending on where the risk estimates are plotted on the risk matrix, **recommended** actions from this study were developed to either: (1) restrict the flow in the canal to achieve tolerable risk levels, or (2) implement corrective actions to lower the risks if project water needs require canal flows that result in risks above the tolerable risk levels.

The 2014 risk analysis results were used to identify subreaches which resulted in tolerable short-term and unacceptable risk levels at the 350 and 600 ft^3/s vegetated stage level. At the 350 ft^3/s vegetated stage level about 6 miles of canal resulted in tolerable short-term and unacceptable risk levels. At the 600 ft^3/s vegetated stage level an additional 5.8 miles of canal resulted in tolerable short-term and unacceptable risk levels (see Figure ES-1). These areas generally include the approach to Tunnel 3 where the canal is unlined above the railroad, the entire Fernley Reach, and select subreaches in the Lahontan Reach with adverse geometry and higher consequences. The subreaches which were identified as having elevated risks and additional subreaches that have been identified as having excessive seepage losses were addressed by this CAS. Additional subreaches in the Lahontan Reach that were identified as having elevated risks from hydrologic potential failure modes have also been addressed by this CAS.

The updated risk analysis results have been used to develop a revised flow/stage level restriction for the Truckee Canal. The revised flow/stage level restriction was fully implemented December 2016 by the Mid-Pacific Region and documented in an October 2016 letter to the Truckee Canal Irrigation District (TCID). In general the revision removes the 350 ft^3/flow restriction at the Wadsworth Gauge and modifies the stage-level restriction levels in the Fernley Reach. Additional staff gauges have been installed by LBAO and TCID to monitor the revised stage-level restriction. The stage level was established by limiting the seepage gradient and providing improved freeboard to lower the risk of the key potential failure modes. The revised stage-level restriction has a peak operating range of about 300 to 540 ft^3/s. The peak operating range represents an unchecked canal and the expected peak flows given seasonal vegetation effects. [redacted]

Risk Reduction Alternatives

Ten CAS alternatives have been developed to address the PFMs that pose the highest risk to the Truckee Canal (see Table ES-2). The CAS alternatives can be grouped into three categories; 1) linear canal improvements to reduce the likelihood of an internal erosion failure (Alternatives 1 through 5), 2) improvements to inline hydraulic control structures to minimize the potential for ice or debris jams and to improve operational control (Alternative 6), and 3) flood protection features to prevent a rapid stage-level rise or canal bank overtopping (Alternatives 7 through 10). The alternatives have been used in combination to create risk reduction plans to address the key PFMs and to achieve the desired risk reduction.

The linear canal improvement alternatives have been developed to facilitate phased implementation. The initial phases would be constructed in areas with the highest risks (i.e. adverse embankment geometry, highest observance of embankment flaws, and highest consequence levels). Subsequent phases would be constructed in areas with the next highest risk levels. The LBAO requested the phased implementation plan be developed to incrementally improve the canal to safely convey flows with a peak operating range of 600 to 900 ft^3/s (maximum canal diversions modeled during the 1997 Final Adjusted Operating Criteria and Procedures (OCAP) Study). A two-phase implementation plan has been developed to achieve this. Figure ES-1 shows the areas to be improved as part of Phase I (flows ranging from 350 to 600 ft^3/s) and then Phase II (flows ranging from 600 to 900 ft^3/s). Table ES-1 provides a summary of the stage-level restrictions and seasonal peak operating ranges that would be in place over time.

While the CAS has developed a phased implementation plan to incrementally improve the canal to safely convey flows with a peak operating range of 600 to 900 ft^3/s, a water reliability and financial feasibility analysis has been completed as part of the CAS to determine whether the Phase II buildout is justified to sustain project water demand and meet the long-term Newlands Project water needs. The financial feasibility study is discussed further below and in detail in Section XI. The flow/stage level that will meet the long-term Newlands Project water needs will be developed further during the feasibility-level design, and the final determination will be completed based on the Record of Decision following the Final environmental impact study (EIS). The EIS is being completed by a private consultant to the LBAO. The EIS is expected to be complete by 2019.

Table ES-1.—Summary of Stage-level Restrictions and Associated Peak Operating Ranges

Conveyance Increments Considered	
Increment	Peak Operating Range (ft^3/s) (vegetated – unvegetated flows)
1 (Short-term Risk Managed Restriction)	300 to 540
2 (Phase I improvement)	350 to 600
3 (Phase II improvement)	600 to 900

Alternative 6 includes replacement of the existing check structures with new structures having wider gate bays and weir openings to minimize the potential for ice and debris jams at the approach. The new checks will have automated gates to improve operational control and provide the ability to isolate the canal in the event of a future canal failure.

Flood control features being considered include drainage crossings, gated wasteways, passive spillways, increasing the canal's capacity in select areas, and detention ponds. These alternatives will be located near the largest drainage basins that cross the canal. The flood control features will be used in combination with the linear canal improvement alternatives to lower the risk of internal erosion through the upper portion of the embankment and prevent overtopping from the design storm event (100-year, 6-hour thunderstorm) [19]. Modification to the Hazen Gauge structure is also being considered to reduce sedimentation in the Lahontan Reach and to improve conveyance capacity to Lahontan Reservoir. The LBAO has hired an independent consultant to review/update the hydrologic loadings. Findings from this study will be incorporated during the feasibility-level study.

Risk Reduction Analysis

A risk reduction analysis meeting was held at the TSC the week of May 2, 2016, to evaluate how the CAS alternatives would lower the risks posed by the controlling PFMs. The CAS alternatives were developed to address internal erosion through the embankment and to minimize the potential for flood inflows that might cause a rapid stage-level rise or overtopping. The risk reduction analysis evaluated to what degree each of the CAS alternatives addressed these key potential failure modes.

The risk reduction analysis indicated Alternatives 1, 2 and 4 lower risks to remote and low levels and result in tolerable long-term risk levels for all consequence levels identified at the Truckee Canal. Alternatives 3 and 5 were estimated to provide 2 orders of magnitude risk reduction (i.e. a very high baseline risk estimate would become a moderate risk estimate). In areas with a consequence Level 3, this would result in tolerable short-term risk levels and not achieve the desired long-term risk levels. Success of Alternative 5 (embankment reconstruction) would be directly related to O&M practices (i.e. burrowing animal control program and vegetation management). Should these practices be ignored, the risks of internal erosion could return to the current high levels. Alternatives 3 and 5 are not appropriate for the Fernley Reach, but should be considered in the Lahontan Reach where consequence levels are lower.

The linear canal embankment improvement measures (Alternatives 1 through 4) that include lining or a cutoff wall will also serve to cutoff embankment cracks should a strong earthquake occur in the area. Limiting the flows through the cracks would allow more time for TCID to intervene and stop a seismic related failure.

The hydrologic protective features have been developed to either reduce or eliminate runoff flows to the canal, or discharge the inflows at select locations. Combinations of hydrologic protective features were identified such that the stage level is not allowed to rise more than 1-foot above the 600 ft^3/s vegetated stage level in those areas not improved by the linear embankment improvements (i.e. lining, cutoff wall, or reconstructed embankment). Where the linear embankment improvements are implemented, the flood protection features were designed so that at least 1-foot of canal bank freeboard is maintained. Further study of the hydrologic loadings will allow the feasibility-level design team(s) to accurately size the features and appropriately locate them along the canal.

Replacement of the existing check structures with structures having enlarged gate openings and side weirs will reduce the potential for ice jams at these locations. The new check structures will have automated gates which will allow for improved operational control and can be used to limit outflows in the event of a future breach.

A combination of check structure replacements and both the linear embankment improvement and hydrologic protective feature alternatives will be required to address the key PFMs at the Truckee Canal. Viable combinations of the risk reduction alternatives have been developed. These “risk reduction alternative plans” are further discussed below.

Risk Reduction Plans

Combinations of the CAS risk reduction alternatives have been identified to develop a series of viable "risk reduction alternative plans" to address both internal erosion and hydrologic risks. Factors such as the individual alternative risk reduction, constructability, cost/benefit analysis and combinability of the alternatives to achieve the desired long-term risk reduction and canal conveyance capacity were considered when developing the risk reduction alternative plans. Improvements to operational control, consequence reduction and efficiency were also considered.

The LBAO has indicated the risk reduction alternative plans will be implemented over time as funding becomes available. [redacted] Reclamation will actively manage risk over the extended time to complete construction through annual risk inspections to determine changes over time and the impact on risk management outcomes. Every five years the risk analysis will be updated and result in new short-term risk objectives until the project has completed the long-term solution to risk on the Truckee Canal.

Each of the risk reduction alternative plans include replacement of the check structures to improve operational control and modifications to the Lahontan Reach to reduce sedimentation effects and increase the canal's capacity. In general these activities occur early in each of the identified risk reduction alternative plans.

Five viable risk reduction alternatives plans were developed for consideration by decision makers. Table ES-3 summarizes the recommended risk reduction alternative plans and likely phased implementation order. Table ES-3 also summarizes the risk reduction plan components, expected implementation years and total plan costs.

Hydrologic Analysis

A range of peak operating flow scenarios were simulated using a RiverWare model developed by Reclamation with demand data from the *Truckee Basin Study* [2] updated to represent the estimated current demand, to estimate how each of the risk reduction alternative plans compared to the *modeled historical water supply reliability* in the Newlands Project. For the purpose of the CAS, the modeled historical water supply reliability scenario was developed using the above RiverWare model and current demand data, and is defined as the approximate level of service current Newlands Project water users would have experienced if the historical hydrology from 1901 through 2000 was to be repeated and if the canal was operated with the current operating parameters (e.g., OCAP, TROA, etc) and under canal condition assumptions of the 1997 OCAP modeling (i.e., Truckee Canal capacity of 900 cfs). The modeled historical water supply reliability scenario indicates that water users in the Newlands Project (both Truckee and Carson Divisions) would have historically received at least 95 percent of their modeled water demands in 91 years out of the 100 evaluated (i.e., 9 water short years out of a 100). Nine scenarios ranging from 0 ft^3/s to 900 ft^3/s (maximum canal diversions modeled during the 1997 Final Adjusted OCAP Study) were considered in the analysis. Results of the analysis indicate implementation of Phase I (peak operating flows greater than 540 ft^3/s) would provide a similar level of reliability (9 water short years) when compared to the historical water supply reliability scenario. To achieve 540 ft^3/s with only the Phase I improvements, an enhanced aquatic vegetation control plan would be required. However, modeled scenarios with higher peak operating flows (above ~ 350 ft^3/s) would provide a greater reduction in the water supply shortages to water users within the Newlands Project. This assumes an aquatic vegetation control program is in place to maintain a year round peak operating flow of at least 540 ft^3/s. Above a peak operating flow of about 540 ft^3/s the average annual Truckee Canal diversions and total Carson Division shortages are similar. Further, should the canal be improved to safely convey higher flows during the winter/spring runoff season, Lahontan Reservoir targets will be achieved earlier, and less water will be taken from the Truckee River later in the irrigation season.

A comparison of fully lined versus unlined alternatives indicates water savings through seepage loss reduction along the Truckee Canal is small, on an average, over the period of analysis in comparison to the amount of water typically available in the within the Newlands Project. However, it is acknowledged that canal seepage losses can be a significant percentage of the Truckee River flows below the Derby Diversion Dam during the summer and fall months. The Phase II improvements and whether the increased cost of canal lining is financially feasible/justified is further discussed below.

Economic and Financial Feasibility Analyses

An economic and financial feasibility analysis was completed as part of this CAS to answer:

- **What peak operating flow range provides a similar level of economic benefits as compared to historical conditions? (i.e. is Phase I sufficient to meet the modeled historical reliability, or is Phase I plus Phase II needed?)**
- **Are the higher costs for the full geomembrane/concrete cover liner (i.e. seepage reduction/efficiency improvement alternatives) financially justified?**
- **Does TCID have the ability to repay the government based on the plan for LBAO to make loan installments of one million dollars per year to make the structural improvements?**

The economic analysis considered the benefits of irrigation water in the Carson and Truckee Divisions, Municipal and industrial (M&I) uses, wildlife and wetlands water supply, hydropower, and recreational benefits. Results of the hydrologic analysis discussed above were used to estimate the hydropower and recreation benefits provided by Truckee Canal under the various modeled scenarios by comparing them against the 0 ft^3/s without canal scenario and to evaluate the economic impacts resulting from a change in water supply under the various modeled scenarios relative to the historical reliability scenario. This comparison helps evaluate the benefits assigned to the no-action alternative (140 ft^3/s). Results of this analysis indicated a negligible economic impact (i.e., reduction in the avoided costs/lost project benefits) when the canal is improved to have a flow of at least 540 ft^3/s (Phase I with an enhanced aquatic vegetation control program). <u>The analysis indicated Phase II (peak operating range of 600 to 900 ft^3/s) is not economically justified for any of the alternatives/risk reduction plans being considered</u>.

The construction costs for the full geomembrane/concrete cover alternatives are about twice the costs for the synthetic sheet pile or partial liner alternatives. The economic analysis indicated the construction costs for the full geomembrane/concrete cover alternatives exceed the estimated benefits generated (negative net benefits) and the benefit cost ratio is below one; therefore, the full geomembrane/concrete cover alternatives are not economically justified. <u>The full geomembrane/concrete cover alternative may be suitable for use in select areas with known high seepage losses, but is not financially justified for risk reduction in the larger treatment areas</u>.

An ability-to-pay (ATP) study was completed to assess TCID's ability to generate revenue in order to pay for current Project costs and any additional costs attributable to the proposed risk reduction plans. In general, a district's annual average ATP is calculated by subtracting the estimated annual district-level expenses from the sum of the estimated annual district-level payment capacity and non-operating income. [redacted]

[redacted] The ATP study utilized the reported irrigated acreage, water usage and assessment data from 2015, and held them constant during the 50-year period of analysis. The analysis assumed that Phase I of either Risk Reduction Plan No. 3 or 5 would be implemented and provide a level of water supply reliability similar to the modeled historical water supply reliability (i.e. no unusual water shortages/restrictions would occur). The results of this ATP study indicate that, over the period of analysis, the TCID has the financial capability of paying all existing annual expenses associated with operating the Newlands Project and would be able to repay the Government's loan installments [redacted] Details of the ATP study are provided in Section XI.

Recommended Risk Reduction Plans for Feasibility-level Development

Results of the CAS have been used to identify two risk reduction plans for feasibility level development. These include:

- Risk Reduction Plan No. 3: Replace the check structures, Phase I and Phase II embankment cutoff wall, and two new upslope detention ponds.

- Risk Reduction Plan No. 5: Replace the check structures, Phase I and Phase II partial geomembrane/concrete cover liner system, and one new wasteway and drainage channel in the Lahontan Reach.

These two risk reduction plans provide the required risk reduction and cost about 20 to 60 million dollars less than the other risk reduction plans. Should the ongoing hydrologic loading analysis indicate the flood loadings are lower, then the costs for the hydrologic protective features could be reduced. Should only Phase I or a portion of Phase II provide adequate canal flows to meet the modeled historical water supply reliability, then the total project costs can further be reduced.

The geomembrane/concrete lining system is about twice the cost of the other linear canal embankment improvement alternatives. As a result, the total project costs for Plan Nos. 1 and 2 are much higher. Lining the Lahontan Reach with a geomembrane/concrete cover system (Plan No. 2) could be used to reduce seepage losses, improve efficiency, and convey flood flows to Lahontan Reservoir. A cost comparison of lining the Lahontan Reach and the other hydrologic protective features (wasteways and detention ponds) indicates lining the Lahontan Reach is not a cost effective approach to address the hydrologic risks. A cost benefit analysis of lining the Lahontan Reach could be further evaluated during the feasibility-level study if Project efficiency enhancements are considered.

Plan No. 4 includes extension of the synthetic sheet pile cutoff wall through the Lahontan Reach. Improving the embankment with a cutoff wall alone does not meet the flood routing criteria. A wasteway near the downstream end of the Lahontan Reach is required for this risk reduction plan. As a result, the total project cost is higher than the costs of Plan Nos. 3 and 5.

It is expected the risk reduction features, or combination thereof, that were used to develop risk reduction plan Nos. 3 and 5 will change during the feasibility-level study. Some of the risk reduction features evaluated during this CAS, not included in risk reduction plan Nos. 3 and 5 may provide added value and reduce costs and should not be excluded for future consideration.

Some of the proposed risk reduction features require additional right-of-way and/or land acquisition (i.e. detention ponds and discharge channels). Reclamation should communicate these preliminary plans with stakeholders and property owners to avoid future development of these areas. The detention/infiltration pond concept may allow for financial partnering with the City of Fernley. The detention pond near the U.S. Alt. 95 canal crossing will improve public safety and potentially allow the City of Fernley to exercise their surface water rights and supplement their aquifer recharge program.

The hydrologic, economic and financial feasibility analysis have indicated construction of Phase II is not needed to meet the modeled historical water supply reliability and is not economically justified. However, decision makers should consider the residual risks in the Phase II areas that might not be improved. Without the Phase II improvements, there would remain unimproved segments that would continue to be vulnerable to internal erosion during flood induced stage level rise. Additionally, the limited conveyance capacity in those areas not improved would limit the ability to convey flood inflows through the Fernley Reach for discharge further downstream leading to potential for overtopping. Should decision makers choose to only implement Phase I of the linear canal embankment improvements, then additional hydrologic protective features will be required in the Fernley Reach (not presently included in risk reduction plan No. 3 and 5). This might include a combination of a detention pond at pour point No. 8 and a wasteway near the Farm District Road Seep.

The proposed risk reduction plans presented herein include structural improvements to only those subreaches that were identified as having elevated risks at the flow / stage-levels being considered. The risk reduction plans do not preclude incidents or failures from occurring in areas where the structural improvements are not made. The areas outside of the phased risk reduction plans areas should continue to be inspected and monitored closely.

Canal Efficiency Modifications

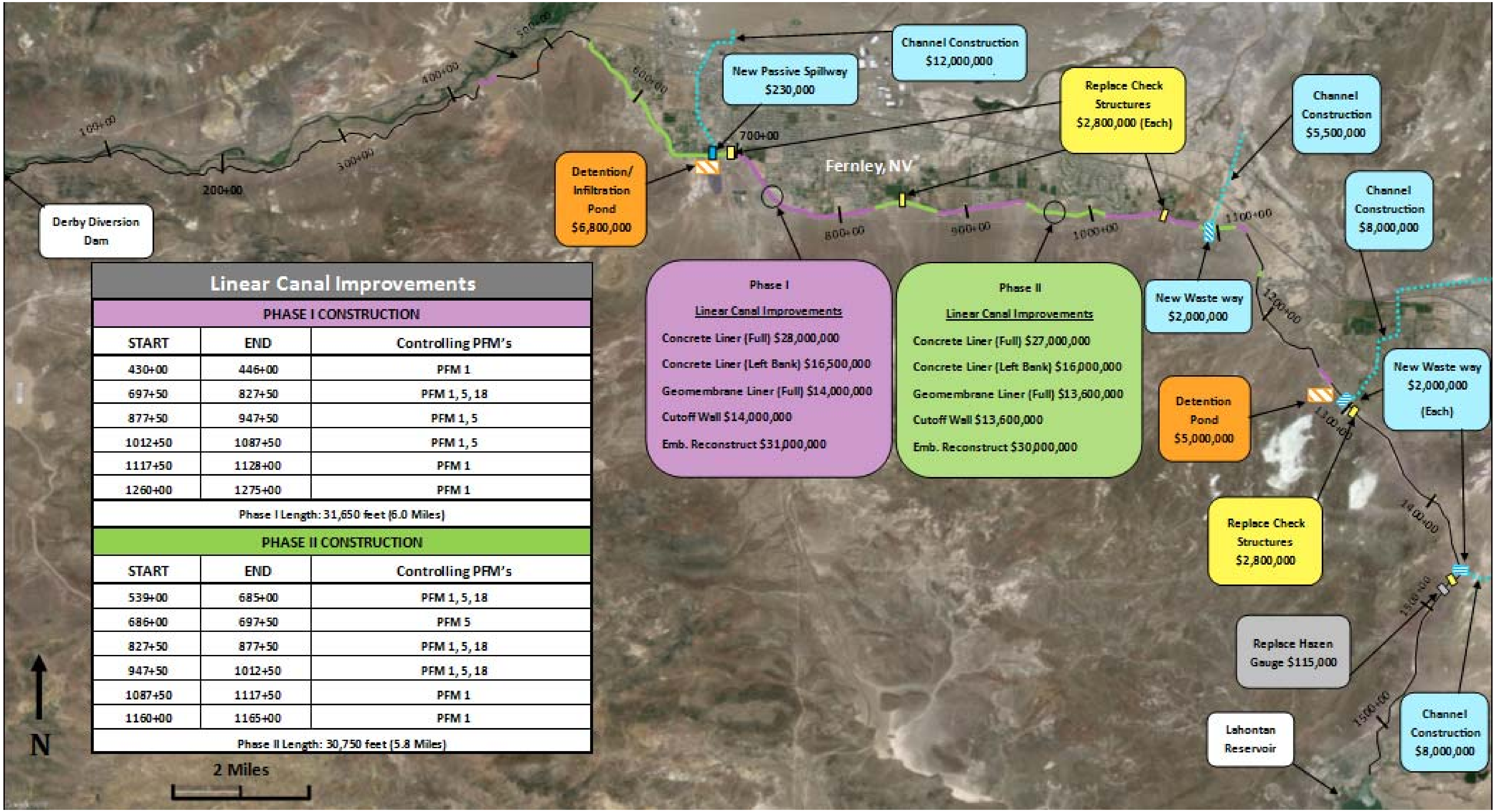
The feasibility-level study should continue to incorporate design features or operations controls that improve efficiency and minimize seepage losses. During the feasibility-level study the Risk Reduction Plan Nos. 3 and 5 should be considered with the expanded use of Alternative 3 (geomembrane/soil-cover lining) in the Lahontan Reach. Both Risk Reduction Plan Nos. 3 and 5 include geomembrane/soil-cover lining at the Steam Pad and Red Barn Seep areas. The use of geomembrane/soil cover lining throughout the remainder of the Lahontan Reach will further reduce seepage losses and reduce internal erosion risks. The addition of about 10 miles of geomembrane/soil-cover lining the Lahontan Reach would add about $23,000,000 to the total project costs for Risk Reduction Plan Nos. 3 and 5 listed in Table ES-3. Justification for addition of the geomembrane/soil-cover lining in the Lahontan Reach would need to be evaluated during the feasibility-level study.

Table ES-2.—Summary of Corrective Action Alternatives

Alternative	Description	Objectives	PFMs Addressed	Cost per linear foot ($/ft)	Phase I Cost ($)	Phase II Cost ($)	Comments
Alt. 1 – Geomembrane/Concrete Cover Canal Lining (Full Canal Prism)	Fully line existing canal prism with a geomembrane/concrete lining in areas with elevated IE risks.	Cutoff embankment flaws, minimize seepage losses, improve conveyance capacity	PFM1, PFM5, PFM11, PFM10, PFM18	885	28,000,000	27,000,000	Reshaping and improvement to existing soils required to support the new lining, lined prism sized for long-term canal flow rate, long-term maintenance required, must be installed during canal outage, will provide improved efficiency
Alt. 2 - Geomembrane/Concrete Cover Canal Lining (Left Canal Bank Only)	Line the left embankment slope and half of the invert with a geomembrane/concrete lining in areas with elevated IE risks.	Cutoff embankment flaws, minimize seepage losses, improve conveyance capacity	PFM1, PFM5, PFM11, PFM10, PFM18	520	16,500,000	16,000,000	Reshaping and improvement to existing soils required to support the new lining, lined prism sized for long-term canal flow rate, long-term maintenance required, must be installed during canal outage, will provide improved efficiency
Alt. 3 – Geomembrane/Soil Cover Canal Lining (Full Canal Prism)	Fully line existing canal prism with a geomembrane/soil cover lining in areas with elevated IE risks.	Cutoff embankment flaws, minimize seepage losses,	PFM1, PFM5, PFM11, PFM18	440	14,000,000	13,600,000	Reshaping and improvement to existing soils required to support the new lining, no improvement to aquatic vegetation affects, will provide improved efficiency
Alt. 4 - Embankment Cutoff Wall	Install synthetic sheet pile wall in left embankment to a depth of 15 feet in areas with elevated IE risks	Cutoff embankment flaws, minimize embankment seepage	PFM1, PFM5, PFM11, PFM18	440	14,000,000	13,600,000	No long-term maintenance required, can be installed during canal operations, minimal impacts to seepage losses or efficiency
Alt. 5 - Embankment Reconstruction	Reconstruct the left embankment in the areas with elevated IE risks	Remove embankment flaws, replace with a well compacted embankment	PFM1, PFM5, PFM11	980	31,000,000	30,000,000	Extensive long-term maintenance, burrowing animal control plan must be effective, constructed during a canal outage, armoring may be required, no reduction to seepage
Alt. 6 - Check Structure Replacement	Replace the existing check structures with automated gates configured to better pass wintertime flows	Utilize automated gates in the event of a future failure to isolate the canal, include side weir openings large enough to minimize potential for ice jams	PFM5	NA	11,200,000 (2,800,000 each)		Will provide improved operational control and management of the restricted stage-level(s), improved ability to use checks to isolate reaches of the canal to minimize releases should a failure occur
Alt. 7 - Drainage Crossings and Channels	Add drainage crossing and conveyance channels at largest pour points to canal	Minimize runoff to canal at largest pour points	PFM10, PFM11	NA	480,000 each (plus channel costs)		Conveyance of flood flows north of the canal will be challenging due to lack of right-of-way

Summary of Corrective Action Alternatives, cont'd

Alternative	Description	Objectives	PFMs Addressed	Cost per linear foot ($/ft)	Phase I Cost ($)	Phase II Cost ($)	Comments
Alt. 8 - New Gated Wasteway(s)	Add wasteways to allow for releases during a flood event or a canal failure	Release water from the canal to prevent elevated stage level and/or embankment overtopping, minimize volume released through a breach section	PFM10, PFM11	NA	2,000,000 each (plus channel costs)		Conveyance of flood flows north of the canal will be challenging due to lack of right-of-way
Alt. 9 - New Passive Spillway(s)	Add passive spillways and conveyance channels to make releases during a flood event	Release water from the canal to prevent elevated stage level and/or embankment overtopping	PFM10, PFM11	NA	230,000 each (plus channel costs)		Conveyance of flood flows north of the canal will be challenging due to lack of right-of-way
Alt. 10 -Detention/Infiltration Pond(s)	Add detention pond(s) at largest pour points	Attenuate flood flows to the canal, detention ponds could be used supplement groundwater recharge	PFM10, PFM11	NA	6,800,000 at Pour Point 8 (5,000,000 elsewhere)		Land acquisition could be challenging, pond should be below grade to not raise the flood risks of an embankment failure, ponds will require cleaning to remove sediment



START	END	Controlling PFM's
PHASE I CONSTRUCTION		
430+00	446+00	PFM 1
697+50	827+50	PFM 1, 5, 18
877+50	947+50	PFM 1, 5
1012+50	1087+50	PFM 1, 5
1117+50	1128+00	PFM 1
1260+00	1275+00	PFM 1
Phase I Length: 31,650 feet (6.0 Miles)		
PHASE II CONSTRUCTION		
539+00	685+00	PFM 1, 5, 18
686+00	697+50	PFM 5
827+50	877+50	PFM 1, 5, 18
947+50	1012+50	PFM 1, 5, 18
1087+50	1117+50	PFM 1
1160+00	1165+00	PFM 1
Phase II Length: 30,750 feet (5.8 Miles)		

Figure ES-1.—CAS Risk Reduction Features included in the Risk Reduction Plans and the Appraisal-level Costs

Table ES-3.—Risk Reduction Plan Summary

Risk Reduction Plan	Activity (Activity cost) (Implementation years)							Risk Reduction Plan Total Cost
1	Replace Check Structures and Hazen Gauge **($11,200,000) (year 2019)**	Phase I Canal Lining, Geomembrane/concrete cover (full prism) **($28,000,000) (years 2020 to 2049)**	Phase II Canal Lining, Geomembrane/concrete cover (full prism) **($27,000,000) (years 2050 to 2079)**	Hazen Wasteway and Drainage Channel **($10,000,000) (year 2088)**	-	-	-	**$76,200,000**
2	Replace Check Structures and Hazen Gauge **($11,200,000) (year 2019)**	Phase I Canal Lining, Geomembrane/concrete cover (full prism) **($28,000,000) (years 2020 to 2049)**	Phase II Canal Lining, Geomembrane/concrete cover (full prism) **($27,000,000) (years 2050 to 2079)**	Phase III Canal Lining, Geomembrane/concrete cover (full prism) **($23,300,000) (years 2080 to 2104)**	Phase IV Canal Lining, Geomembrane/concrete cover (full prism) **($23,300,000) (years 2105 to 2129)**	-	-	**$112,900,000**
3	Replace Check Structures and Hazen Gauge **($11,200,000) (year 2019)**	Phase I Embankment Cutoff Wall **($14,000,000) (years 2020 to 2034)**	Pour Point No. 8 Detention Pond **($6,800,000) (year 2041)**	Pour Point No. 16/19 Detention Pond **($5,000,000) (year 2046)**	Phase II Embankment Cutoff Wall **($14,000,000) (years 2047 to 2061)**	-	-	**$51,000,000**
4	Replace Check Structures and Hazen Gauge **($11,200,000) (year 2019)**	Phase I Embankment Cutoff Wall **($14,000,000) (years 2020 to 2034)**	Phase II Embankment Cutoff Wall **($14,000,000) (years 2035 to 2049)**	Phase III Embankment Cutoff Wall **($9,300,000) (years 2050 to 2059)**	Phase IV Embankment Cutoff Wall **($14,000,000) (years 2060 to 2074)**	Rock Ditch Wasteway and Drainage Channel **($10,000,000) (year 2084)**	-	**$72,500,000**
5	Replace Check Structures and Hazen Gauge **($11,200,000) (year 2019)**	Phase I Canal Lining, Geomembrane/concrete cover (partial prism) **($16,500,000) (years 2020 to 2036)**	Phase II Canal Lining, Geomembrane/concrete cover (partial prism) **($16,000,000) (years 2037 to 2052)**	Hazen Wasteway and Drainage Channel **($10,000,000) (year 2061)**	-	-	-	**$53,700,000**

Notes: [1]Costs for replacement of the Hazen Gauge are not included in the risk reduction plans. [2]Costs for adding geomembrane/soil cover lining in the Lahontan Reach will add about $23M to plan Nos. 1, 3 and 5.

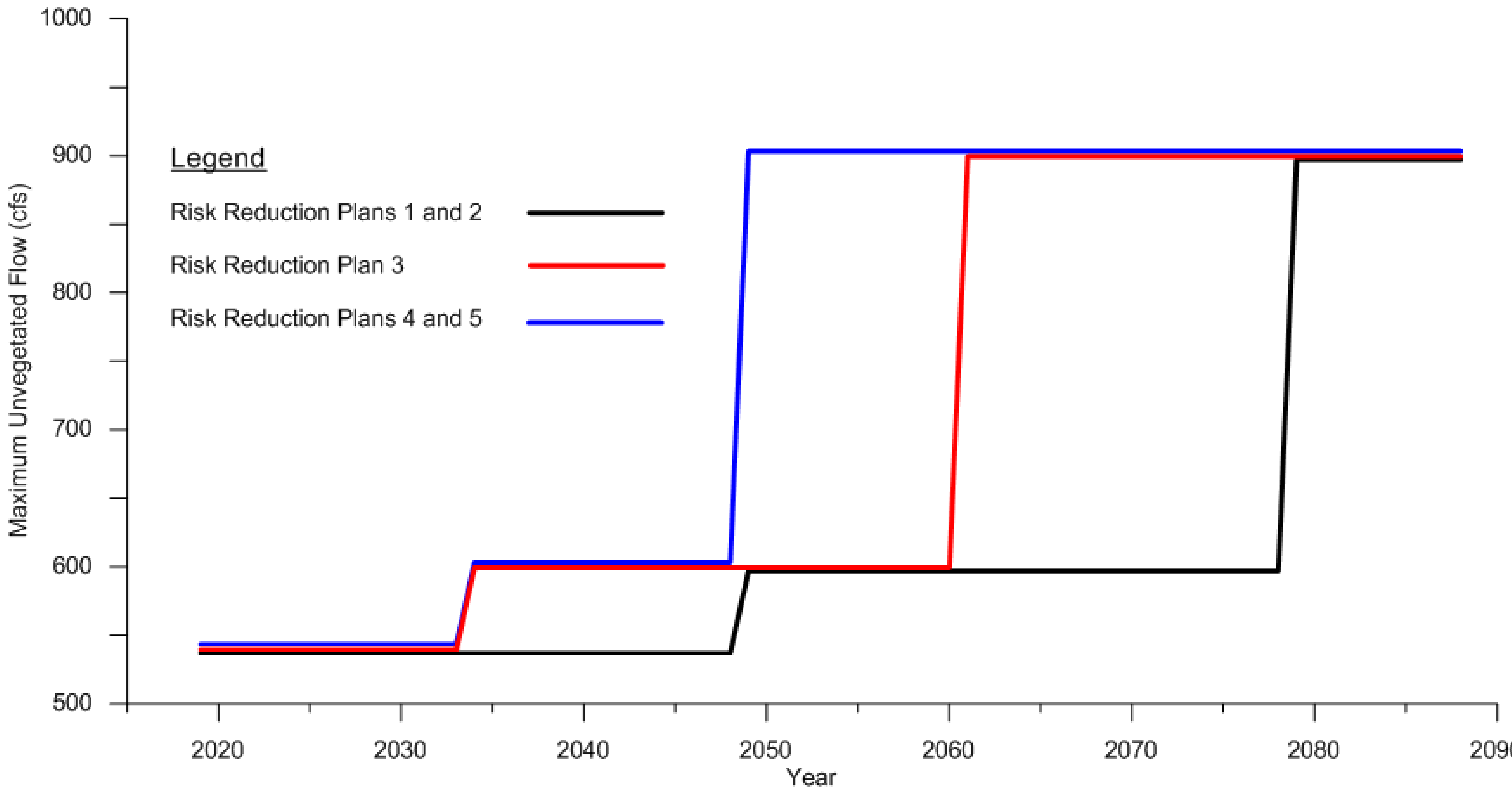


Figure ES-2 — Peak Operating Flow over Time for the Risk Reduction Plans

Technical Memorandum No. QY-2016-8311-1

Truckee Canal Corrective Action Study

I. Introduction

A. Purpose

The purpose of this Technical Memorandum (TM) is to document corrective action alternatives developed to address the identified risks posed by the Truckee Canal. Findings from a 2014 risk analysis study are summarized in a report titled; "*Updated Risk Analysis – Truckee Canal, Issue Evaluation and Technical Report of Findings*", dated June, 2015 [1]. Corrective action alternatives have been developed to improve the canal and its infrastructure to safely convey diversions from the Truckee River to Lahontan Reservoir to support the Newlands Project water needs.

B. Scope

The Bureau of Reclamation Lahontan Basin Area Office (LBAO) requested the Bureau of Reclamation Technical Service Center (TSC) complete a corrective action study (CAS) to identify potential corrective actions to lower risks posed by the Truckee Canal and to minimize the potential for a future failure. The TSC's scope of work for the CAS is outlined in a project management plan (PMP) dated August 18, 2015. Task to be completed as part of the CAS include:

1. Develop corrective actions alternatives to improve the canal and its infrastructure so it can safely convey flows and better support the Newlands Project water needs.
2. Develop corrective actions alternatives to address the potential failure modes which result in tolerable short-term and unacceptable risk levels at the flow/stage levels being considered.
3. Develop phased risk reduction alternative plan(s) that outline how the corrective action alternatives should best be implemented over time as funding becomes available. The phased risk reduction alternative plan(s) will be developed to incrementally increase the canal flow/stage level from the current restriction which results in a peak operating range of 300 to 540 ft^3/s up to a peak operating range of 600 to 900 ft^3/s. Following the implementation of the safety improvements, regulatory requirements outlined in the Truckee River Operating Agreement (TROA) and the 1997 final Operating Criteria and Procedures (OCAP) will continue to govern flows in the Truckee Canal.

4. Coordinate the corrective actions alternative development with an environmental impact study (EIS) being prepared concurrently by a consultant to the LBAO.
5. Support the LBAO in completion of water supply reliability modeling that reflects the corrective actions alternatives and how they might impact the efficiency of the canal.
6. Complete an economic and financial feasibility analysis on the phased risk reduction alternative plan(s). The financial feasibility analysis will identify whether the phased risk reduction plans meet the Newlands Project demand/historic reliability and discuss whether Phase II of the risk reduction plans are justified. The financial feasibility analysis will also evaluate TCID's ability to repay loan installments from Reclamation for the safety improvement measures.
7. Select at least two phased risk reduction alternative plan(s) to be carried forward to feasibility-level design.

II. Background

A. Project Description

The Truckee Canal was constructed between 1903 and 1905 as part of the Newlands Project and is among the Bureau of Reclamation's oldest structures. Water is diverted into the Truckee Canal from the Truckee River at the Derby Diversion Dam, about 20 miles east of Reno, Nevada. The canal is about 31 miles long and discharges into Lahontan Reservoir, at the left abutment of Lahontan Dam. Truckee Canal flows are used to deliver irrigation water to agricultural lands along the length of the canal (Truckee Division) and to supplement inflows into Lahontan Reservoir from the Carson River for irrigation in the Carson Division. All care, operation, and maintenance responsibilities for the Newlands Project, including the Truckee Canal, were transferred to the Truckee-Carson Irrigation District (TCID) by Reclamation on December 31, 1926. The canal is divided into three reaches: (1) the Derby Reach (approximately 10.3 canal-miles from Derby Diversion Dam to the city of Fernley, NV); (2) the Fernley Reach (approximately 11 canal-miles within the city of Fernley, NV); and (3) the Lahontan Reach (approximately 9.7 canal-miles from the city of Fernley to Lahontan Reservoir). The original design capacity of the canal was about 1,200 cubic feet per second (ft^3/s). Canal flows ranged from about 300 to about 900 ft^3/s prior to January 2008.

B. January 2008 Canal Failure

On January 5, 2008, a portion of the canal embankment failed, causing flooding and property damage within Fernley, NV. Following the failure, numerous studies were completed to evaluate the cause, to evaluate the risk of future failures, and to develop feasible alternatives for improving the safety of the Truckee Canal. The failure was believed to result from internal erosion, which resulted from animal burrows in the canal embankment, combined with a rapid increase in the canal diversions in order to capture storm floodwaters in the Truckee River. Since the canal failure, a flow restriction of 350 ft^3/s has been established through a court order to limit the risk of future canal failures. Stage-level restrictions were established by Reclamation in June 2009 at four staff gauge locations within the Fernley Reach, corresponding to the 350-ft^3/s flow rate, which was based on a 2008 Hydrologic Engineering Centers River Analysis System (HEC-RAS) model for the Truckee Canal. The term "stage level," as used in this report, is defined as the "unchecked" water surface level along the length of the canal for a given inflow from the Derby Diversion Dam. The word "checking," as used in this report, is defined as artificially increasing the water surface within the canal for a given flow rate to improve irrigation deliveries. This stage-level restriction was intended as a short-term (1 to 5 years) restriction until permanent structural improvements could be made to the Truckee Canal.

C. Risk Analyses from 2008 to 2010.

From 2008 through 2010, four risk analysis studies were completed to evaluate a range of canal stage levels and different loading conditions [3]. At the conclusion of the initial risk analysis meeting, requests for additional field exploration data and additional engineering analyses were made. Risk analyses were updated as this information was made available.

The risk analyses evaluated stage levels representing 250-, 350-, and 600-ft^3/s unchecked flow rates to capture the range of expected canal diversions to support the Newlands Project operations. Loading conditions that were evaluated included static normal operations, seismic loads, and hydrologic loads. The canal was divided into subreaches based on geologic conditions, lined versus unlined canal, consequences resulting from failure of the canal, and the ability to isolate segments with operational controls. The subreaches were primarily assigned in the Derby and Lahontan Reaches. The 2008-2010 risk estimating teams (RET) judged the Fernley Reach was similar along its left embankment length, and risk estimates made for this portion of the canal were used to represent the risks for the Fernley Reach as a whole. Separate risk estimates were made for the right embankment sections, at the Farm District Road seep area, and where improvements to the canal were already implemented.

The Potential Failure Modes (PFMs) with the highest identified risks were estimated to result from internal erosion through the left embankment, overtopping from ice jams, overtopping from flood inflows, and increased risk of internal erosion during flooding without overtopping. [redacted] Findings from the risk analyses were used to evaluate future operation of the canal and to identify areas along the canal where corrective actions should be taken to reduce the risk of future canal failure.

D. 2011 Corrective Action Study

In 2010 and 2011, the TSC completed a CAS, and a number of structural improvements alternatives were developed to address the key PFMs for a range of canal stage levels and to achieve different levels of risk reduction. Findings from these studies were summarized in *Summary of Final Baseline Risk Estimates and Evaluation of Risk Reduction for Proposed Corrective Action Alternatives, Truckee Canal Issue Evaluation Report of Findings*, April 2011 [3].

Corrective action alternatives included decommissioning of the canal, canal lining, construction of a cutoff wall through the left embankment, embankment reconstruction, placement of the canal in a gravity pipe system, installation of additional wasteways and drainage crossings, replacement of the turnout structures, and replacement of existing check structures. Estimated field costs ranged from about $30 million to $55 million, depending on the long-term canal flow rate needed for the Newlands Project, degree of risk reduction required, and combination of the above corrective actions. The length of treatment included the entire Fernley Reach and thousands of feet in the Derby and Lahontan Reaches. The only actions that have been taken from the 2011 CAS include replacement of the turnouts and removal of stock water lines in the Fernley Reach.

E. 2014 Updated Risk Analysis Findings

In 2013, the LBAO requested the TSC update the baseline risk estimates that were made following the January 2008 Truckee Canal failure and then use the findings to assist in developing an updated flow/stage level restriction and identify areas along the canal requiring structural improvements to lower the long-term risks to be addressed by this CAS. Currently, Reclamation does not have an agency-wide, risk-based methodology for evaluation of canals. A risk analysis process was developed specifically for the Truckee Canal following the 2008 canal failure and was updated by the 2014 risk estimating team (RET) to support development of a revised flow/stage level restriction and to identify areas along the canal needing improvement. The updated risk analysis process is outlined in a Technical

Memorandum, titled *Proposed Risk Analysis Process for the Truckee Canal, Decision Document and Technical Memorandum*, July 2014 [4]. For portraying and interpreting the risk analysis results, a risk matrix was developed and includes the consequence level on the x-axis and the risk likelihood on the y-axis. Guidelines generally adopted for dam safety evaluations were used to define regions on the matrix that indicate tolerable long-term risk levels, tolerable short-term risk levels, and unacceptable risk levels. Depending on where the risk estimates are plotted on the risk matrix, recommended actions were developed to either: (1) restrict the flow in the canal to achieve tolerable risk levels, or (2) implement corrective actions to lower the risks if project water needs require canal flows that result in risks above the tolerable risk levels. The risk portrayal matrix is included as figure II-1.

FAILURE LIKELIHOOD	CONSEQUENCES OF FAILURE LEVEL 0	LEVEL 1	LEVEL 2	LEVEL 3	LEVEL 4
VERY HIGH	Increased monitoring is an appropriate risk management activity, additional risk reduction needed to maintain agency credibility	[1]Short-term risk reduction required (within 6 months). Funding commitment and planning for [2]long term structural risk reduction required within 12 months. Full implementation of long term structural risk reduction required within 5 years. Needed to maintain agency credibility	Implement Canal Stage-Level Restriction to minimize short-term risks and Complete Long-term Structural Risk Reduction Measures if needed when Project Flow Rates result in PFMs plotting in this box	Implement Canal Stage-Level Restriction to minimize short-term risks and Complete Long-term Structural Risk Reduction Measures if needed when Project Flow Rates result in PFMs plotting in this box	Implement Canal Stage-Level Restriction to minimize short-term risks and Complete Long-term Structural Risk Reduction Measures if needed when Project Flow Rates result in PFMs plotting in this box
HIGH	Current monitoring schedule is an appropriate risk management activity, continue current annual maintenance improvements	Increased monitoring is an appropriate risk management activity, additional risk reduction through annual maintenance may be necessary	[1]Short-term risk reduction required (within 6 months). Funding commitment and planning for [2]long term structural risk reduction required within 12 months. Full implementation of long term structural risk reduction required within 5 years.	Implement Canal Stage-Level Restriction to minimize short-term risks and Complete Long-term Structural Risk Reduction Measures if needed when Project Flow Rates result in PFMs plotting in this box	Implement Canal Stage-Level Restriction to minimize short-term risks and Complete Long-term Structural Risk Reduction Measures if needed when Project Flow Rates result in PFMs plotting in this box
MODERATE	No risk reduction action needed, continue current monitoring and annual maintenance improvements	Current monitoring schedule is an appropriate risk management activity, continue current annual maintenance improvements	Increased monitoring is an appropriate risk management activity, additional risk reduction through annual maintenance may be necessary	[1]Short-term risk reduction required (within 6 months). [2]Appropriateness of Long-term Structural Risk Reduction to be evaluated within 12 months	Implement Canal Stage-Level Restriction to minimize short-term risks and Complete Long-term Structural Risk Reduction Measures if needed when Project Flow Rates result in PFMs plotting in this box
LOW	No risk reduction action needed, continue current monitoring and annual maintenance improvements	No risk reduction action needed, continue current monitoring and annual maintenance improvements	Current monitoring schedule is an appropriate risk management activity, continue current annual maintenance improvements	Increased monitoring is an appropriate risk management activity, additional risk reduction through annual maintenance may be necessary	[1]Short-term risk reduction required (within 6 months). [2]Appropriateness of Long-term Structural Risk Reduction to be evaluated within 12 months
REMOTE	No risk reduction action needed, continue current monitoring and annual maintenance improvements	No risk reduction action needed, continue current monitoring and annual maintenance improvements	No risk reduction action needed, continue current monitoring and annual maintenance improvements	No risk reduction action needed, continue current monitoring and annual maintenance improvements	No risk reduction action needed, continue current monitoring and annual maintenance improvements

Tolerable long-term risk levels

Tolerable short-term risk levels

Unacceptable short-term and long-term risk levels

Long-term flow/stage level restriction

Risk managed short-term flow/stage level restriction

Notes: [1] Short-term risk reduction measures may include enhanced vegetation and rodent control, automation of existing structures, increased monitoring, implementation of warning systems, stockpiling of liner materials, stockpiling of berm and filter materials, emergency readiness, additional engineering studies, and more detailed risk estimates to better understand the risks, etc.

[2] Long-term risk reduction measures may include structural improvements to the canal, additional automated isolation structures and wasteways, etc., to maintain the long-term safety of the Truckee Canal.

Figure II-1.—Risk matrix including recommended actions for the Truckee Canal

[redacted]

[redacted]

[redacted]

- [redacted]

  [redacted]

- [redacted]

  [redacted]

- [redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

5. Summary of Risks

The risks associated with the failure of the Truckee Canal were determined by combining the likelihood of failure and the consequence level for each of the 22 PFMs considered. The 2014 risk analysis results indicate that the highest risks are from the following PFMs:

- PFM1 – Internal Erosion through the Embankment
- PFM5 – Ice Jams Leading to Internal Erosion or Overtopping
- PFM10 – Embankment overtopping during hydrologic loadings
- PFM11 – Flooding leading to stage-level rise and internal erosion
- PFM18 – Seismic Deformation and Cracking Leads to Internal Erosion

The RET identified moderate to high uncertainty in the engineering analysis leading to the high hydrologic risk estimates. Additional hydrologic studies and a contributing sub-basin investigation program were completed in 2015/2016 to update the hydrologic loadings [6]. The updated hydrologic hazard study findings were considered in a subsequent risk analysis meeting held in May 2016 with similar RET members. The revised hydrologic loadings information provided to the RET indicated a high to very high likelihood of a flood related failure when the canal is operated above the 350 ft^3/s stage level, consistent with the 2014 risk analysis study [1] findings. The remaining PFMs considered by the RET resulted in tolerable long-term risk level estimates at each of the stage levels evaluated and do not require a flow/stage level restriction or other risk reduction measures at this time. A summary of the primary potential failure modes that contribute to the risk of the canal along with initial risk reduction actions is shown in Table II-1.

Figure II-2 through II-4 present the subreaches which resulted in unacceptable and short-term tolerable risk levels at the 600 ft^3/s vegetated stage level. The subreaches highlighted on Figures II-2 through II-4 will require structural risk reduction measures to safely convey flows up to the vegetated stage level. These areas generally include the approach to Tunnel 3 where the canal is unlined above the railroad, the entire Fernley Reach, and select subreaches in the Lahontan Reach with adverse geometry and higher consequences.

Subreaches which are known to have had poor historic performance and/or excessive seepage losses have been identified and are also addressed by this CAS. [redacted]

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[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

F. 2015 Value Planning Study

A value planning (VP) study was held in Carson City, Nevada in June 2015 [7]. The VP study was held to develop a preliminary list of viable canal improvement alternatives to be considered during the CAS. The VP study consisted of team members from the LBAO, Mid-Pacific (MP) Region, the TSC, TCID, the City of Fernley, EIS contractor representatives, the Pyramid Lake Paiute Indian Tribe, and the Fallon Paiute Shoshone Indian Tribe. The VP study brainstorming phase centered around safety improvements to the canal but also measures to improve canal efficiency and reduce water usage. The VP study team listed over 100 proposals to address the above issues. The team selected 8 of the most viable proposals to be further developed. These include:

- Proposal 1 - Canal Lining (full and partial)
- Proposal 2 - Embankment Cutoff Wall
- Proposal 3 - Add Emergency Wasteway(s)
- Proposal 4 - Construct Retention/Detention Ponds
- Proposal 5 - Construct Automated Check Structures
- Proposal 6 - Construct Effluent Pipeline from Reno to the Truckee Canal Headworks
- Proposal 7 - Focused Water Right Purchases
- Proposal 8 - More and Accurate Water Measurement Devises

Proposals 1 through 5 were developed to address canal safety issues and have been included for further development as part of this CAS. Other proposals or components of proposals not presented by the VP team have been considered by this CAS team. Proposals 6 through 8 provide means for alternative water supplies and efficiency improvements. These proposals are outside of the scope of work for this CAS, but should be considered as part of the EIS.

III. Design Data and Engineering Analysis

A. Survey Topography

An aerial survey of the canal and the City of Fernley was completed March 21, 2008. The survey was flown at a scale of 1”=300’ with a reported vertical accuracy of +/- 0.5 ft. The survey data was used to develop 1-foot contours for a zone 300 feet either side of the canal alignment and within the City of Fernley. The survey data is in the State Plane Coordinate System having a horizontal datum of North American Datum (NAD) 1983 State Plane Zone Nevada West in U.S. survey feet and vertical datum of North American Vertical Datum (NAVD) 1988 in U.S. survey feet.

When the aerial survey was flown the majority of the canal was dry, however; an estimated 50 ft^3/sec was being diverted from Derby Dam to the Gilpin Wasteway. This resulted in the upstream 7.8 miles of the Truckee Canal between Derby Dam and the Gilpin Wasteway being partially inundated. Photogrammetry is only valid for topography above the water surface. A site visit was held shortly after the aerial survey to measure the flow depths which were then used to manually adjust the cross sections in this area. Flow depths ranged from about 1 to 1.5 feet.

In 2012 and 2014 TCID completed invert cleaning and sediment removal from the canal in the Fernley and Lahontan Reaches. The excavations lowered the canal invert from about 6 inches up to 4 feet. To reflect these changes ground surveys were completed by the MP Region's surveyors in August 2014 and January 2015. The TSC used the ground survey information to modify the March 2008 contour data within the canal prism. The modified contour data was used in the CAS when measuring required earthwork quantities and when completing hydraulic modeling.

B. Geologic Investigations and Foundation Observations

Geologic investigations were completed along the length of the Truckee Canal, including surface mapping, drill holes, and cone penetration test (CPT) soundings. The Fernley Reach was investigated first. Drill holes were completed August 2009 through September 2009, with some additional drill holes completed in April 2010. CPT investigation in the Fernley Reach was completed in October 2009. Drill holes and/or CPTs were completed approximately every 500 feet along the crest of the left embankment. A total of 36 drill holes and 116 CPTs were advanced in the Fernley Reach.

Drill holes in the Lahontan and Derby Reaches were completed March through May 2010. A total of 11 drill holes in the Derby Reach and 13 drill holes in the Lahontan Reach were completed during this period. CPTs in the Lahontan and Derby Reaches were completed in May 2010. A total of 39 CPTs in the Derby Reach and 29 CPTs in the Lahontan Reach were completed during this period.

Geologic data packages were prepared by the MP Geology group, including the individual drill hole and CPT logs, surface observations and geologic mapping, and geologic subsurface profiles and cross sections [8]. Additional geologic observations were made during the Fernley Reach turnout replacements in 2012. The observations were documented by the MP geology group [9]. Findings from the available geologic investigations were the basis for the CAS alternatives described herein.

C. Hydrology

The Truckee Canal crosses about 20 natural drainages with contributing basins ranging from less than 1 square mile to nearly 45 square miles for a total of 100 square miles. The drainages collect precipitation from the areas south and west of the canal alignment. Due, in part, to the arid environment, the original construction did not include any drainage crossings, and the drainages discharge directly into the canal. These inflows accumulating in the Fernley and Lahontan Reaches of the canal, where there are no wasteways, have the potential to result in overtopping and are a source of canal sediment.

There has only been one known failure at the Truckee Canal attributed to runoff. In 1919 rapid snow melt caused large runoff from pour point No. 8 near the SH-95 bridge in the Fernley Reach. There is no evidence suggesting that the canal was overtopped but, rather, the opposite canal bank was eroded from the inflows. There have been no other reported hydrologic events in the 110 years of operation that threatened overtopping.

The TSC's Flood Hydrology Group completed an initial hydrologic hazard analysis (HHA) study in support of the risk analysis studies following the 2008 canal failure. The initial study, completed in 2010, indicated the potential for storms with a return period of about 25 years to result in inflows that exceed the canal's capacity. The 2010 study was updated in 2012 by collecting additional topography data, refining the precipitation-frequency curves, developing both one dimensional (1-D) and two dimensional (2-D) runoff models, and then routing the potential inflows for a range of initial canal diversions at the onset of the storm. The critical storm identified in the 2012 study was the wintertime 48-hour general storm. This was based on the assumption that runoff volume would be a greater threat to the canal than peak discharge. Cumulative inflows from the 25-year event exceeded the freeboard storage volume, had inflows that were nearly double the canal's capacity and indicated a very high likelihood of failure by overtopping.

The HHA was updated in 2016 [6]. Work included a field infiltration program, refining the sub-basin soils data with latest geologic mapping information, and calibrating the 1-D and 2-D runoff models to a significant rain event in July, 2013 (a thunderstorm).

The 2016 HHA study evaluated both short-duration thunderstorm and long-duration general storms. Rainfall totals representing a 100-year event were selected in the analysis. The study evaluated the estimated 6-, 24- and 48-hour, 100-year rainfall events. Rainfall totals estimates were 1.4-, 2.6- and 3.6-inches respectively. A front loaded mass curve was selected to model the 6-hour thunderstorm type rainfall event and a two-thirds distribution mass curve was selected to model the 24- and 36-hour general storm events. Twenty pour points from the contributing basins were identified in the study. Pour Point No. 8, located in the Fernley Reach has the largest contributing basin (about 45 square miles). A field infiltration program completed in the contributing sub-basins and calibration to a July 2013 thunderstorm event were used to develop revised overland flow models. A 1-D model was developed for the contributing sub-basins. Loss values in the 1-D model were calibrated to outflows during the July 2013 event. A 2-D model was developed for the basins with alluvial fans to better understand the overland flow characteristics and infiltration. Loss values assigned to the 2-D model were selected from the field infiltration program and calibration to the July 2013 event. Findings from the overland flow modeling study indicated the 6-hour thunderstorm event produced the largest peak runoff values. While the general storm events have the potential to produce greater runoff volume, the cumulative runoff rate from the 6-hour event is about 4 times the canal's capacity.

Flood routings [6] were completed with a range of initial canal flows at the onset of the flooding. Two pour point inflow scenarios were selected: 1) inflows from all 20 pour points along the canal, and 2) inflows from pour points 5 through 20 only, which assumes the existing wasteways and passive spillways in the Derby Reach offset the inflows from pour points 1 through 4. Results of the flood routing study indicate stage-level rise approaching the embankment crest and/or embankment overtopping is likely in the Fernley and Lahontan Reaches of the canal when diversions at the onset of the flooding are 350 ft^3/s or greater.

An updated hydrologic risk analysis meeting was held in May 2016 considering the information presented in the 2016 HHA report. Findings from the risk analysis meeting indicated a high to very high likelihood of overtopping in the Fernley and Lahontan Reaches. Areas in the Lahontan Reach with the least amount of freeboard and where sedimentation and vegetation cause a stage-level rise were judged to have the highest likelihood of overtopping. While the critical storm type had changed from the previously completed hydrologic risk analyses, the findings were similar and indicate hydrologic protective features are needed to protect the Fernley and Lahontan Reaches from flood inflows.

The 2016 HHA used an aerial reduction factor to account for variations in the rainfall totals throughout the contributing basins. It was also assumed that each of the contributing basins would be impacted at the same time. Since the critical design storm is a thunderstorm, the 2016 RET judged it was unlikely that the thunderstorm would be of sufficient size to impact all of the drainages, and if it did there would be some time lag as it traveled along the south side of the canal. While there remains uncertainty in the 2016 HHA findings, the RET judged the latest study does a good job at estimating the potential runoff from the individual drainages but recommended that further work be done to understand the aerial extent and expected travel patterns for the design storm event.

For this CAS the LBAO recommended the 2016 HHA results be used for sizing the hydrologic protective features. Table III-1 summarizes the peak discharge inflows and runoff volumes for the 6-hour thunderstorm event to be used during the CAS. The LBAO plans to contract with a local engineering firm and staff from NOAA's Meteorologic Group in Reno, Nevada to further refine the hydrologic loadings. This study will then be used to refine the hydrologic protective features designs during the feasibility-level design phase.

Table III-1. – Peak flows and storm volumes into the canal for the 6-hour local storm

Pour Point	Model	Peak Flow (ft^3/s)	Total Volume (ac-ft)
1	HEC-1	246	22
2	HEC-1	213	16
3	HEC-1	519	53
4	HEC-1	645	51
5	HEC-1	522	51
6	HEC-1	42	2
7	None	0	0
8	SRH2D	667	67
9	SRH2D	14	1
10	SRH2D	32	1
11	SRH2D	0	0
12	SRH2D	66	3
13	SRH2D	40	2
14	SRH2D	18	1
15	None	0	0
16	HEC-1	341	44
17	HEC-1	165	16
18	HEC-1	245	26
19	HEC-1	633	106
20	HEC-1	471	59

D. Seismology

A site-specific seismic hazard analysis was performed for the Truckee Canal in 2010 [10]. The analysis was performed to identify seismic sources, develop probabilistic seismic loadings, and analyze potential surface displacements. Seismic loads for the canal are relatively high due to the close proximity to the

Pyramid Lake Fault, which is thought to intersect the canal in the western portion of the Fernley Reach. The estimated ground motions are highest at the intersection of the canal and the Pyramid Lake Fault, and they decrease as the distance from the Pyramid Lake Fault increases. Table III-2 shows the peak horizontal acceleration for selected return periods at five sites along the canal. Site 102 is just downstream from Tunnel 3, Site 100 is at TC-1, Site 103 is at the Ricci Lane Seep, Site 101 is at the Tedford Bridge, and Site 104 is at TC-12.

Table III-2.—Seismic Loadings for the Truckee Canal [10]

Record Period (years)	102	100	103	101	104	Lahontan Dam
1,000	0.31	0.34	0.32	0.27	0.23	0.24
2,000	0.44	0.49	0.46	0.35	0.33	0.32
5,000	0.64	0.76	0.72	0.49	0.45	0.44
10,000	0.80	0.95	0.91	0.62	0.57	0.54
20,000	0.95	1.15	1.10	0.74	0.73	0.66
50,000	1.20	1.45	1.38	0.95	0.93	0.82

When evaluating existing and new structures (non-embankment), seismic loadings will be limited to the approximately 500-yr return period ground motions (10% probability of exceedance in a 50-yr period). This earthquake loading criteria for structures is consistent with design of new canal structures [11]. When evaluating the canal embankments and foundations, seismic loadings will be limited to the 2,500-yr return period ground motions. This return period corresponds to a 2% probability of exceedance in a 50-yr period and is based on design code ASCE 7 [12], currently being adopted in Reclamation's Design Standard No. 3 [11]. From the information in Table III-2, the peak horizontal accelerations near the center of the Fernley Reach for the 500 and 2,500-year events are about 0.2 and 0.5 g respectively.

IV. Design Criteria

The LBAO has requested the TSC develop corrective action alternatives to improve the Truckee Canal and its infrastructure to safely convey diversions from the Truckee River to Lahontan Reservoir to support the Newlands Project long-term water needs. The LBAO has also requested the alternatives be developed considering improvements to canal operations, conveyance efficiency and minimize water losses.

A meeting was held November 19, 2015 with the LBAO, TSC and MP decision makers to identify the desired long-term canal flow capacity and the hydrologic and seismic loading conditions for which the CAS alternatives will be designed. The selected design criteria and loading conditions are described below.

A. Canal Conveyance Capacity

The LBAO has indicated the CAS repair alternatives are to be developed to allow the Truckee Canal to safely convey flows to meet project demand. While the canal would be improved to safely convey water demand needs, diversion restrictions will continue to control diversions from the Truckee River until the final repairs for public safety are completed. Improving the safe conveyance capacity will allow TCID improved operational flexibility and provide added protection from flood inflows and icing conditions.

Truckee Canal repairs will be implemented over time, starting in the highest risk areas identified in the 2015 Truckee Canal Risk Analysis [1], to allow incremental conveyance capacity increases. The CAS implementation plan will consider the following incremental conveyance capacities and the associated repairs:

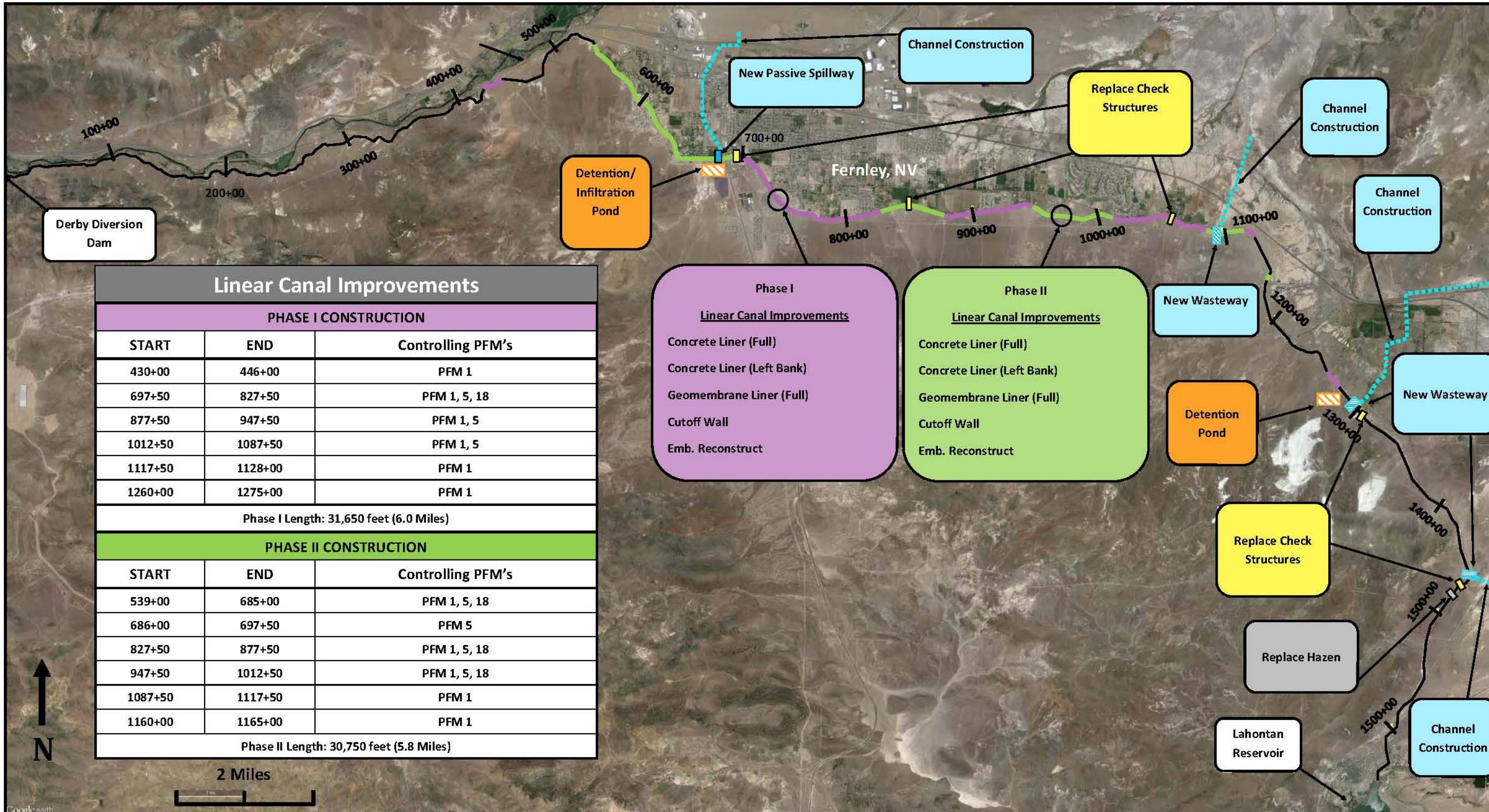
Conveyance Increments Considered	
Increment	Peak Operating Range (ft^3/s) (vegetated – unvegetated flows)
1 (Short-term Risk Managed Restriction)	300 to 540
2 (Phase I improvement)	350 to 600
3 (Phase II improvement)	600 to 900

Each incremental improvement will allow TCID improved operational flexibility to meet water demand in the Newlands Project; however, repairs will be limited to those necessary to meet project demand and to safely convey flood flows as discussed below. If that level of repair can be reached with the completion of increment 1 and/or 2, then increment 3 may not be considered.

The LBAO has requested the CAS alternatives allow for a phased implementation plan. As funding becomes available components of the phased implementation plan would be constructed to incrementally increase the safe conveyance capacity. Phase I of the implementation plan would address those subreaches resulting in unacceptable and tolerable short-term risk levels while operating in the peak operating range of 350 ft^3/s to 600 ft^3/s stage level (about 31,650 feet of canal). Phase II of the implementation plan would address those remaining subreaches resulting in unacceptable and tolerable short-term risk levels while operating up to the Newlands Project demand stage level with a peak operating range of 600 ft^3/s to 900 ft^3/s (estimated to be an additional 30,750 feet of canal).

While the CAS has developed a phased implementation plan to incrementally improve the canal to safely convey flows with a peak operating range of 600 to 900 ft^3/s, a water reliability and financial feasibility analysis was completed as part of the CAS to determine whether the Phase II buildout is justified to sustain project water demand and meet the long-term Newlands Project water needs. The financial feasibility study is discussed further below and in detail in Section XI. The flow/stage level that will meet the long-term Newlands Project water needs will be developed further during the feasibility-level design stage, and the final determination will be completed based on the Record of Decision following the Final EIS.

Figure IV-1 shows the subreaches of the canal to be improved as part of Phase I and II respectively.



Linear Canal Improvements

START	END	Controlling PFM's
PHASE I CONSTRUCTION		
430+00	446+00	PFM 1
697+50	827+50	PFM 1, 5, 18
877+50	947+50	PFM 1, 5
1012+50	1087+50	PFM 1, 5
1117+50	1128+00	PFM 1
1260+00	1275+00	PFM 1
Phase I Length: 31,650 feet (6.0 Miles)		
PHASE II CONSTRUCTION		
539+00	685+00	PFM 1, 5, 18
686+00	697+50	PFM 5
827+50	877+50	PFM 1, 5, 18
947+50	1012+50	PFM 1, 5, 18
1087+50	1117+50	PFM 1
1160+00	1165+00	PFM 1
Phase II Length: 30,750 feet (5.8 Miles)		

Figure IV-1.—Corrective Action Study Phased Improvement Areas

B. Winter Icing Conditions

The Truckee Canal is unique in that normal operations include wintertime diversions from the Truckee River into Lahontan Reservoir. Operation of the canal throughout the winter results in icing during some atmospheric conditions. The ice which forms in the Truckee Canal has resulted in at least one known canal failure and has reportedly caused a near embankment overtopping incident. Changes in the inflows from the Derby Diversion Dam result in raising and lowering of the water surface in the canal. This causes the ice that has formed on the surface to break up. As the pieces of ice begin to flow along the canal, they become “jammed” at check structures, turns, or narrower canal sections. As the ice begins to pile up, it forms a constriction, which results in a rapid rise in the canal water surface upstream of this location.

The long-term risk reduction measures will be developed to minimize the potential for ice jams. These measures will likely include replacing the existing check structures, with new structures better suited to pass flows during icing conditions. Also the linear embankment improvements will be designed to improve the upper portions of the embankment that might be inundated in the event of an ice jam.

C. Hydrologic Loading Conditions

The Truckee Canal was constructed in the early 1900’s when hydrologic hazard and probabilistic loadings were less understood. As such, the lower two thirds (approximately 20 miles) of the canal does not have drainage crossings or evacuation structures (spillways/wasteways). Today Reclamation uses hydrologic hazard analyses to adequately size protective features for its canals and/or limit the flood inflows. Reclamation’s Design Standard No. 3, Chapter 7 [11] outlines design guidelines for canal flood protection features.

For small canals ($Q < 100$ ft^3/s) cross drainages (culverts or overshoots) are sized to convey the expected 25-year peak storm runoff. The drainage crossings are used in combination with upslope detention capacity to pass/retain the expected 100-year runoff volume [11]. This is done to prevent runoff from entering the canal that might result in overtopping. For medium and large sized canals (medium is considered $101 < Q < 1000$ ft^3/s), the design flood frequency for sizing the drainage crossings and upslope detention capacity is to be determined on an individual basis. Factors such as higher consequences in urban areas, potential damage to high value crop lands, or impacts from canal outages are to be considered when selecting the appropriate level of flood protection.

As an alternative to drainage crossings and upslope detention storage, drain inlets which allow runoff to enter the canal may also be considered [11] or used in combination with drainage crossings. Drain inlets are to be sized so that runoff inflow does not exceed 10 percent of the capacity of the canal unless evacuation structures are installed immediately upstream from the drain inlet. Measures to minimize eroded sediment from entering the canal must be considered. The evacuation structures are to be sized such that the maximum canal water surface rise is equal to one half the freeboard for lined canals or one-fourth the freeboard for unlined canals and whichever is least where both lined and unlined conditions exist.

Improving the Truckee Canal to current design standards would be difficult due to private and commercial land development adjacent to the canal. Right of way limitations and impacts to private properties would likely result in implementation delays and high costs. For the Truckee Canal a <u>risk based process for sizing the flood protection features, while limiting the hydrologic loading to what would be selected for a new medium sized canal has been proposed</u> and is described below.

The Truckee Canal has had a history of failures when the canal water surface was raised above the 400 ft^3/s stage-level. These failures have been attributed to embankment flaws (animal burrows and tree roots) in the upper portion of the embankment leading to internal erosion. Flooding has the potential to quickly raise the canal water surface above the 400 ft^3/s stage-level inundating those flaws. If the inflows exceed the canal's storage capacity overtopping will likely occur. For the Truckee Canal, flood protection features will be designed in cooperation with the linear canal embankment improvements to both address the potential for embankment overtopping and minimize the potential for internal erosion and piping (prior to overtopping). To accomplish this, <u>flood protection features should be sized such that the stage level is not allowed to rise more than 1-foot above the 600 ft^3/s vegetated stage level in those areas not improved by the linear embankment improvements (i.e. lining, cutoff wall, or reconstructed embankment) to facilitate the long-term canal flow rate. Where the linear embankment improvements are implemented, the flood protection features should be designed so that at least one foot of canal bank freeboard is maintained.</u> To accomplish this, a combination of protective features such as drainage crossings, spillways/wasteways, upslope storage and detention ponds will be sized to meet the above criteria. Sediment cleaning in the lower Lahontan Reach will be required to achieve the one foot of rise above the 600 ft^3/s vegetated stage level to avoid the potential for overtopping. Additionally, modifications to the Hazen Gauge structure may be required to further lower the Lahontan Reach stage level. Risk reduction analysis will be performed to verify that tolerable long-term risk levels are achieved with the proposed features.

<u>The 6-hour, 100-year event has been selected to size and locate the flood protection alternatives for this CAS.</u> Routing scenarios for evaluation of the flood protection alternatives will assume inflows from pour points 5 through 20 only. It

is considered unlikely that all pour points would be impacted by a thunderstorm type event. Designing flood protection features for inflows from pour points 5 through 20 will likely provide greater than 100-year protection. In the development of feasibility designs further effort will be completed to analyze the storm type and storm development/movement pattern to get a more realistic estimate of the hydrologic impact on project designs.

Flood routing computations will be completed using a HEC-RAS model for the Truckee Canal to size and locate the flood protection features, while meeting the freeboard criteria described above. The flood protection features will be designed so that overtopping from the estimated 100-year runoff inflows does not occur. The risk reduction analysis team will consider the likelihood of the flood loading (100-year) and the conditional likelihood of failure considering the addition of the flood protection features and the freeboard criteria listed above.

D. Seismic Loading Conditions

The Truckee Canal is located in a seismically active area. In October, 2010 a Probabilistic Seismic Hazard Analysis (PSHA) was completed for the Truckee Canal for five locations along the canal [10]. Peak horizontal accelerations were developed for earthquakes with a 1,000 to 50,000-yr return period. Findings from this report have been used to assess the existing seismic risks along the canal and when developing corrective action alternatives. Seismic risks are primarily driven by strong shaking resulting in embankment cracking (without foundation liquefaction), leading to an internal erosion failure. The likelihood of foundation liquefaction was considered to be low and limited to just a few areas along the canal.

When evaluating existing and new structures (non-embankment), seismic loadings will be limited to the approximately 500-yr return period ground motions (10% probability of exceedance in a 50-yr period). This earthquake loading criteria for structures is consistent with design of new canal structures [11].

When evaluating the canal embankments and foundations, seismic loadings will be limited to the 2,500-yr return period ground motions. This return period corresponds to a 2% probability of exceedance in a 50-yr period and is based on design code ASCE 7 [12], currently being adopted in Reclamation's Design Standard No. 3 [11]. Soil related seismic failures are expected to be sudden with little warning. Whereas reinforced concrete structures are expected to perform in a ductile manner and will likely remain functional under the design earthquake loading. Considering this, the return period associated with a threshold earthquake resulting in canal failure due to a structure failure would be much greater than 500 years. Therefore, selection of the 2,500-yr return period ground motion for embankments and foundations is not considered to be excessive and is consistent with current design codes for new structures [11]. The risk reduction analysis team will consider the likelihood of the loading (500- and 2,500-year) and the conditional likelihood of failure considering the addition of the embankment improvement features. The location and linear length of the embankment improvement features will be designed so that the seismic risks lie in the tolerable long-term risk level zone upon full implementation.

A summary of the proposed static, hydrologic and seismic loadings conditions to be considered when developing corrective action alternatives to reduce the risk are presented in Table III-3.

Table III-3 – Loading Conditions for Development of Risk Reduction Corrective Action Alternatives for the Truckee Canal

Static Normal Operations	Notes
Facilitate unchecked canal flow to meet project demand. Subreaches resulting in tolerable short-term and unacceptable risk levels at the Newlands Project demand stage level will be improved incrementally by the risk reduction measures. The minimum level of corrective action will be the Phase I improvements (Increment 1). Phase II improvements will be implemented as needed to meet project demand. Project demand will be determined by modeling in the CAS, and refined through the Environmental Impact Statement analyses as needed.	A phased implementation plan will be developed to incrementally increase flows up to the Newlands Project demand stage level.
Hydrologic	
Flood protection features designed for the 6-hour, 100-year flood event. Flood protection features will be sized to not allow the canal water surface to rise one foot above the Phase II peak operating range stage level in unimproved areas and retain at least one foot of freeboard where embankment improvements are implemented.	Design to include a combination of drainage crossings, detention ponds, upslope storage and spillway/wasteways.
Seismic	
500-yr ground motions.	New and existing structures, per design standards
2500-yr ground motions	Embankments and foundations

E. Operations and Efficiency Improvements

The CAS alternatives should be developed considering improvements to operations and efficiency. Supervisory Control and Data Acquisition (SCADA) and remote operation equipment should be considered for the following:

- Check structures
- Wasteways (new and existing)
- Truckee Canal headworks at Derby Diversion Dam
- Stage-level monitoring equipment
- Flow measurement at turnouts

Benefits of SCADA and remote operations include improved ability to control the canal stage level, isolation of the canal in the event of a failure, better control of diversions from Derby Reach, ability to quickly drain the canal in the event of a failure, better control of Truckee River inflows, and limiting turnout deliveries to the actual needs and calls.

Seepage losses are believed to be dependent on the canal flow rate/stage level and are often highest after an extended shut down. During normal operations seepage loss has been estimated range from about 12 to 20 percent of the flow rate that passes the Wadsworth gauging station. [20] Canal safety improvements which include canal lining will also reduce seepage losses and improve efficiency. When evaluating the extent of canal lining alternatives, reaches where seepage losses are known to occur, but are not included in areas requiring risk reduction, should also be considered for lining to improve efficiency and minimize seepage losses.

There are five check structures at the Truckee Canal that are used to check the water surface to facilitate turnout deliveries. Checking the water surface results in lower flow velocities and reduced freeboard. Replacement of the check structures is being considered as part of the XM project to improve operational control and lower risks by checking only to a level that is required to make the turnout deliveries.

V. Risk Reduction Alternatives

Ten CAS alternatives have been developed to address the PFMs that pose the highest risk to the Truckee Canal. Those are: PFM1-Internal Erosion through the Embankment, PFM5-Ice Jams Leading to Internal Erosion of Overtopping, PFM10-Flooding Leading to Overtopping, PFM11-Flooding Leading to Internal Erosion and PFM18-Seismic Shaking leads to Cracking and Internal Erosion.

The CAS alternatives can be grouped into three categories; 1) linear canal improvements to reduce the likelihood of an internal erosion failure (Alternatives 1 through 5), 2) improvements to inline hydraulic control structures to minimize the potential for ice or debris jams and to improve operational control (Alternative 6), and 3) flood protection features to prevent a rapid stage-level rise or canal bank overtopping (Alternatives 7 through 10). Each of the alternatives are described individually but will be used in combination to address the key PFMs and to achieve the desired risk reduction.

The linear canal improvement alternatives have been developed to facilitate phased implementation. The initial phases would be constructed in areas with the highest risks (i.e. adverse embankment geometry, highest observance of embankment flaws, and highest consequence levels). Subsequent phases would be constructed in areas with the next highest risk levels. The LBAO requested the phased implementation plan be developed to incrementally improve the canal to safely convey peak operating flows ranging from 600 to 900 ft^3/s. A two-phase implementation plan has been developed to achieve this. Figure IV-1 shows the areas to be improved as part of Phase I (flows ranging from 350 to 600 ft^3/s) and then Phase II (flows ranging from 600 to 900 ft^3/s). Each phase is about 6 miles long. Special treatment areas not identified by the conditional risk estimates (i.e. Steam Pad Seep area) were also included in Phase I and II and are discussed below.

Improvements to the in-line hydraulic control structures are limited to Alternative 6 – Replacement of the existing Check Structures. The new check structures will be constructed with wider gate bays and weir openings to minimize the potential for ice and debris jams at the approach. The automated gates will also improve operational control and provide the ability to isolate the canal in the event of a future canal failure.

Flood control features being considered include drainage crossings, gated wasteways, passive spillways, increasing the canal's capacity in select areas, and detention ponds. These alternatives will be located near the largest drainage basins that cross the canal. The flood control features will be used in combination with the linear canal improvement alternatives to lower the risk of internal erosion through the upper portion of the embankment and prevent overtopping from the design storm event (100-year, 6-hour thunderstorm) [19].

Historic operations have included the use of the Bango Check Structure to check the water surface beyond the Mason Check Structure to make deliveries in the Lahontan Reach. The Hazen Gauge (an inline broad crested weir structure) also checks the water surface in the lower Lahontan Reach. Historic operations of the Bango Check and the configuration of the Hazen Gauge have apparently attributed to sediment and aquatic vegetation accumulation in the Lahontan Reach which in turn reduced the capacity of the canal and poses an elevated risk of hydrologic overtopping. Each of the risk reduction plans include modifications to Bango Check operations and replacement of the Hazen Gauge to reduce sediment accumulation and provide an increased flow capacity to convey flood flows through this reach. Costs for replacement of the Hazen Gauge have not been included in risk reduction plans as the LBAO has indicated the gauge will be replaced separate from the XM project. Each of the CAS alternatives are described in the following sections.

A. Special Treatment Areas

Three special treatment areas have been identified from the risk analysis study. Those are; 1) an unlined section of canal approaching Tunnel 3 in the Derby Reach, 2) the Steam Pad Seep area in the Lahontan Reach, and 3) the Red Barn Seep area in the Lahontan Reach. The Tunnel 3 and Red Barn Seep areas were identified by the risk analysis as areas needing structural improvement and are included in the Phase I improvement. The Steam Pad Seep area was not identified by the risk analysis but is known to be an area with excessive seepage and is included in the Phase II improvement. The area just upstream of Tunnel 3 (see Figures II-2 and IV-1) is a 3,500 foot long section of unlined canal with the railroad immediately below the canal. The canal transitions from fully lined to unlined at station 410+90, then unlined to the Tunnel 3 portal at station 445+60. There has been a history of seepage in this area and recently sloughing of the slope above the railroad. For conveyance efficiency and to minimize/eliminate seepage in this area, only the fully geomembrane/concrete lined canal alternative is being considered for this area of improvement.

The Steam Pad and Red Barn Seep areas have been problematic for a number of years and believed to be areas with the highest seepage losses. The canal safety improvements will be extended to treat these areas. Appropriate alternatives include those that will minimize/eliminate seepage. These include a fully lined canal section or an embankment cutoff wall with a positive seepage cutoff to bedrock. These special treatment areas in the Lahontan Reach are described further in the following sections.

Seepage conditions at the Truckee Canal change from year to year. As new seepage areas develop, they should be considered on a case by case basis for inclusion in the phased improvement plans.

B. Areas Removed from the Phased Risk Reduction Plans

Three areas were identified to have tolerable-short term risk levels at the 600 ft^3/s vegetated stage level but will not be included in the phased risk reduction alternative plans. [redacted] These subreaches are in an area where the canal is fully lined and have recently been sealed with Aqualastic. [redacted]

[redacted] Internal erosion is considered to be much less likely in this area due to the presence of a liner and lesser degree of embankment flaws. While the concrete lining in these subreaches has cracked and is deteriorated, any replacements should be done as part of regular O&M practices and not part of the CAS phased risk reduction alternative plan. Lining the Fernley Reach will lower the stage level in this area and increase the freeboard capacity.

[redacted] While these two subreaches are just upstream and downstream of an area that required the canal alignment to be rerouted into the right slope, presumably to address adverse seepage, there have been no known failures in this area even when loaded above the 600 ft^3/s vegetated stage level. [redacted] While the geometry indicates an internal erosion failure is more likely in these areas, the low consequences do not justify inclusion in the phased risk reduction alternative plan. If lining of the Lahontan Reach is considered to decrease seepage losses, the area [redacted] should be improved first to address the potential internal erosion risks.

C. Canal Lining and Cutoff Wall Technologies Considered

A number of canal lining and cutoff wall systems were considered by the CAS team. The following sections describe the screening process and selection of the preferred canal lining and cutoff wall systems.

1. Types of Canal Lining Considered

A number of lining system technologies were considered by the project team and include:

- Compacted clay
- Geosynthetic Clay Liner (GCL) with soil cover
- Geomembrane with soil cover
- Geomembrane with concrete overlay
- Geomembrane with shotcrete overlay
- Unreinforced concrete
- Reinforced Concrete

Factors such as cost, effectiveness, constructability, construction duration, long-term risk reduction, and maintenance were considered by the project team. Each of these considerations have been summarized in Table V-1. Key factors which led to selection of the preferred lining system(s) are indicated by bold text.

Table V-1 – Evaluation of Potential Canal Lining Technologies

Type of Cutoff Wall	Advantages	Disadvantages	Typical Unit Cost ($/sf)
Compacted clay	• Clay liner covers entrance to existing embankment flaws • Allows for reshaping of the canal prism and invert profile.	• Does not remove existing flaws (animal burrows, roots, bad lifts, coarse zones) • **Clay lining vulnerable to future burrowing animal and tree roots** • **Clay lining can desiccate and crack during extend outages**	2 to 5
GCL with soil cover	• GCL cuts off entrance to existing embankment flaws • Allows for reshaping of the canal prism and invert profile. • Failed or damaged GCL areas are easy to repair • GCL is flexible and may span seismic induced transverse cracks	• Does not remove existing flaws (animal burrows, roots, bad lifts, coarse zones) • Construction defects may align with existing embankment flaws • Sediment cleaning activities may damage GCL • GCL vulnerable to future burrowing animal and tree roots • Difficult to inspect and identify flaws • **GCL has low tensile strength and will likely tear from seismic induced transverse cracks**	3 to 8
Geomembrane with soil cover	• **Geomembrane practically eliminates seepage** • Geomembrane cuts off entrance to existing embankment flaws • **Geomembrane is flexible and expected to span seismic induced transverse cracks**	• Does not remove existing flaws (animal burrows, roots, bad lifts, coarse zones) • Construction defects may align with existing embankment flaws • Sediment cleaning activities may damage Geomembrane • Difficult to inspect and identify flaws	8 to 15
Geomembrane with concrete cover	• **Lowers canal lining roughness which increases capacity and lowers the canal stage level for a given flow** • **Concrete eliminates future borrowing** • Controls vegetation (including aquatic vegetation) • Reduced requirements for sediment cleaning activities • **Geomembrane practically eliminates seepage** • For internal erosion to occur, a flaw in the concrete, geomembrane, and embankment would have to align • Long life design – Geomembrane not susceptible to UV degradation • Movement sufficient to cause a tear in the Geomembrane will cause cracks in concrete – easy to spot • **Concrete cover/geomembrane system expected to span seismically induced transverse cracks**	• Does not remove existing flaws (animal burrows, roots, bad lifts, coarse zones) • Concrete cover will crack and deteriorate over time and require replacement • Concrete lined canal makes egress more difficult, ladders will be required	5 to 8
Geomembrane with shotcrete overlay	• **Lowers canal lining roughness which increases capacity and lowers the canal stage level for a given flow** • **Shotcrete eliminates future borrowing** • Controls vegetation (including aquatic vegetation) • Reduced requirements for sediment cleaning activities • **Geomembrane practically eliminates seepage**	• Does not remove existing flaws (animal burrows, roots, bad lifts, coarse zones) • Shotcrete cover will crack and deteriorate over time and require replacement • Shotcrete lined canal makes egress more difficult, ladders will be required • Shotcrete has somewhat higher roughness than concrete paving	5 to 10

Type of Cutoff Wall	Advantages	Disadvantages	Typical Unit Cost ($/sf)
	• For internal erosion to occur, a flaw in the shotcrete, geomembrane, and embankment would have to align • Long life design – Geomembrane not susceptible to UV degradation • Movement sufficient to cause a tear in the Geomembrane will cause cracks in concrete – easy to spot • **Shotcrete cover/geomembrane system expected to span seismically induced transverse cracks**		
Unreinforced concrete	• **Concrete lining cuts off entrance to existing embankment flaws** • **Lowers canal lining roughness which increases capacity and lowers the canal stage level for a given flow** • **Concrete eliminates future borrowing** • Controls vegetation (including aquatic vegetation) • Reduced requirements for sediment cleaning activities	• Does not remove existing flaws (animal burrows, roots, bad lifts, coarse zones) • Concrete liner will crack and deteriorate over time and require replacement • **Seepage through construction joints and cracks can be significant** • **Unreinforced concrete may not span seismically induced cracks**	5 to 7
Reinforced Concrete	• **Concrete lining cuts off entrance to existing embankment flaws** • **Lowers canal lining roughness which increases capacity and lowers the canal stage level for a given flow** • **Concrete eliminates future borrowing** • Controls vegetation (including aquatic vegetation) • Reduced requirements for sediment cleaning activities • Reinforced concrete lining will require less repair/replacement • **Reinforced concrete liner expected to span seismically induced transverse cracks**	• Does not remove existing flaws (animal burrows, roots, bad lifts, coarse zones) • **Seepage through construction joints and cracks can be significant**	15 to 20

Notes: Typical unit costs obtained from Reclamation's project experience.

2. Recommended Lining System

The geomembrane/concrete cover and geomembrane/soil cover options were selected as the preferred lining systems. The geomembrane lining will practically eliminate vertical seepage losses and cutoff horizontal seepage from existing embankment flaws. Geomembrane lining is flexible and is expected to span seismically induced transverse cracks. The concrete and soil covers will protect the geomembrane from ultra-violet (UV) light exposure and extend the design life. The geomembrane/concrete cover lining system has been incorporated as part of CAS Alternatives 1 and 2, full and partial lined prisms respectively. The geomembrane/soil cover lining system has been incorporated as part of Alternatives 2 and 3 and are discussed further below.

3. Types of Cutoff Walls Considered

A number of cutoff wall technologies were considered by the project team and include:

- Synthetic Sheet Pile
- Steel Sheet Pile
- Cement-bentonite Slurry
- Soil-cement-bentonite Mixed In-place
- Controlled Low Strength Concrete
- HDPE Geomembrane

Similar factors when selecting the preferred lining system were used when evaluating the viable cutoff wall systems. Each of these considerations has been summarized in Table V-2. Key factors which led to selection of the preferred cutoff wall system(s) are indicated by bold text.

Table V-2 – Evaluation of Potential Cutoff Wall Technologies

Type of Cutoff Wall	Advantages	Disadvantages	Typical Unit Cost ($/sf)
Synthetic Sheet Pile	• **Vinyl materials do not corrode, have a long design life and low permeability** • Excavation is not required • **Synthetic sheet piles well suited for most of the proposed installation length** • Positive barrier to future burrowing animals and tree roots • Rodent deterrent chemicals can be added to the vinyl materials • All work can be completed from the embankment crest • **Minimal impacts to canal operations** • Fast installation rate • Less noise generated during installation • **Well suited for incremental installation** • Low cost alternative • **Expected to perform well during an earthquake even with large embankment deformations**	• Difficult to install in dense or cobbly soils. Pre-driving or trenching required in these areas • Interlocking joints may separate during installation • Interlocking joints leak • **Specialized equipment/experience required when installing in difficult soil conditions**	15 to 20
Steel Sheet Pile	• **Installation procedures are well established** • Positive barrier to future burrowing animals and tree roots • **Minimal impacts to canal operations** • Excavation is not required • High strength • Able to construct irregular shapes and alignments • Expected to perform well during an earthquake even with large embankment deformations	• **More expensive than synthetic or slurry type walls** • Difficult to install in cobbly materials • More noise generated during driving • Limited depth of penetration • Interlocking joints may separate during installation • Interlocking joints leak • **Steel products corrode which shortens the design life**	25 to 35
Cement-Bentonite Slurry	• Construction techniques well understood and practiced • **For desired depths and soil conditions, a long-reach excavator can be used** • Much stronger than soil-based slurry walls • Would likely be resistant to earthquake induced deformation and serve as a flow limiter	• **Staging areas, haul routes and excavation spoil pile results in a large disturbed area** • Slower installation rate as compared to driven piles • **May not be a barrier to future burrowing animals or tree roots** • Freeze/thaw and desiccation cracking could lead to increased permeability over time	10 to 20

Type of Cutoff Wall	Advantages	Disadvantages	Typical Unit Cost ($/sf)
Soil-Cement-Bentonite Mixing	• Construction techniques well understood • A number of mixing/installation equipment available • Less spoils to be disposed of	• **Staging areas, haul routes and excavation spoil pile results in a large disturbed area** • Substantial volumes of spoils must be disposed of • Specialized equipment/experience required • Slow installation rate • **Would not be a barrier to future burrowing animals or tree roots** • Freeze/thaw and desiccation cracking could lead to increased permeability over time • Low strength, would do little to strengthen the embankment during an earthquake	20 to 40
Controlled Low Strength Concrete Cutoff Wall	• **Concrete wall would be strong and resistant to future burrowing animals and tree roots** • Would likely be resistant to earthquake induced deformation and serve as a flow limiter	• Batch plant near the site would be required throughout the construction • Staging areas, haul routes and excavation spoil pile results in a large disturbed area • **Substantial volumes of spoils must be disposed of** • **High cost alternative**	30 to 40
HDPE Geomembrane	• **HDPE materials do not corrode, have a long design life and low permeability** • Proven installation methods • Sealable interlocking joints • Bio-polymer slurry can be used but would need to be backfilled with excavated materials	• Depth of vibratory installation about 20 feet • Slurry wall required in difficult installation areas • **Specialized equipment/experience required** • Staging areas, haul routes and excavation spoil pile results in a large disturbed area	25 to 35

Notes: Typical unit costs obtained from Reclamation's project experience [18].

4. Recommended Cutoff Wall System

The synthetic sheet pile was selected as the preferred cutoff wall system. Synthetic sheet piles have been a proven cutoff wall technology for addressing similar levee embankment flaw issues [13, 14]. The synthetic sheet piles have a lower installed cost when compared to the other technologies. There would be minimal impacts to canal operations and surrounding public. Synthetic sheet piles are well suited for about 9 of the 10 miles to be improved where the embankment and foundation soils are mostly loose and fined-grained. Areas where coarse alluvial deposits exist may require either pre-driving with an H-pile section or pre-trenching. A field trial is scheduled for the fall of 2017 to evaluate the viability of synthetic sheet piles in the difficult soil locations.

D. Alternative 1 – Geomembrane/Concrete Cover Canal Lining (Full Canal Prism)

This alternative includes addition of a lining system that spans the full canal prism. The lining would be extended to the embankment crest to cutoff existing embankment flaws and lower the risk of internal erosion (see Figure V-1). The existing canal prism will be reshaped and include a 33-foot wide bottom width, 2H:1V side slopes and a prism depth of 13.6 feet. The lining system will include a geomembrane liner with a 3.5-inch-thick unreinforced concrete cover. The geomembrane will be secured in an anchor trench near the embankment crest. At the 600 ft^3/s vegetated stage level this alternative provides about 8-feet of freeboard. The concrete lining will lower the canal roughness, minimize aquatic vegetation affects and increase the conveyance capacity where improved. Increasing the conveyance capacity in select areas will improve the canal's ability to route flood inflows to discharge features or to Lahontan Reservoir. Addition of lining will also improve efficiency by reducing seepage losses.

The following sections discuss the PFMs addressed by the addition of canal lining, selection of the preferred canal lining system, efficiency improvements, impacts to the canal's conveyance capacity, and construction and long-term maintenance considerations.

1. PFMs Addressed by this Alternative

This alternative is being considered to address PFM1-Internal Erosion through the Embankment, PFM5-Ice Jams Leading to Internal Erosion of Overtopping, PFM10-Floodinng Leading to Internal Erosion of Overtopping, PFM11- Flood Inflows Lead to Raised Stage Level and Internal Erosion, and PFM18-Seismic Shaking leads to Cracking and Internal Erosion. The primary benefit of this alternative is cutting off flaws and potential seepage pathways through the embankment. The canal lining will serve as the primary water retaining feature.

Canal lining will also improve the conveyance capacity in those areas of the canal that are lined. This will allow for improved flood conveyance to discharge features being considered as part of this CAS.

Canal lining will also address PFM2-Internal Erosion through the Foundation. By limiting or even eliminating seepage into the foundation, this PFM will no longer be viable.

2. Seepage Reduction

Seepage losses along the length of the Truckee Canal have been estimated to range from about 20 to 40 percent of the diversions being made. Higher losses are typically observed following an extended canal outage. About 25 miles of the canal is currently unlined. Much of the seepage losses travel downwards from the canal's invert, then laterally where shallow-bedrock exists (along the Lower Lakebed Sediment deposits). In some areas the seepage surfaces hundreds or thousands of feet downslope from the canal. A well-constructed geomembrane will practically eliminate seepage losses where employed. Seepage loss reduction will be achieved in areas where lining is used to address the internal erosion risks. Lining should also be used in areas with known excessive seepage losses (i.e. Steam Pad and Red Barn Seep areas). If additional efficiency improvements are needed/justified, lining may be extended outside of the areas requiring risk reduction (i.e. additional areas throughout the Lahontan Reach).

3. Construction and Long-term Maintenance

Lining is advantageous in that it allows for incremental installation as funding becomes available. Construction of any lining system will require a canal outage. Construction of the canal lining will need to be scheduled around periods when deliveries to the Truckee Division are not needed (i.e. fall and winter months). To shorten the canal outage a specialty contractor should be used with canal paving equipment similar to what is shown in Figure V-2. This equipment shapes, prepares the surface, places the geomembrane and concrete cover in one pass. Two to three mile segments of canal are typically required to justify mobilization of this type equipment.

Concrete lining will eliminate the potential for burrowing animals and woody vegetation within the canal prism. A burrowing animal and vegetation control program will still be required for the outer banks. Concrete lining will reduce aquatic vegetation affects and minimize the requirements for vegetation removal. The lining system will require periodic inspections, maintenance and replacement over time.

Figure V-2.—Example of full prism canal paving equipment. (GOMACO, www.gomaco.com)

4. Impacts to Canal Conveyance Capacity

In the segments where improved, the full geomembrane/concrete cover lining is employed canal's capacity will be increased to about 3,000 ft^3/s. This will allow for improved flood conveyance to discharge features being considered as part of this CAS. <u>However, the conveyance capacity of the Truckee Canal will continue to be limited by those areas not improved with canal lining</u>. Sediment accumulation in the Lahontan Reach has reduced the vegetated (summertime) capacity to about 600 ft^3/s and the unvegetated (wintertime) capacity to about 900 ft^3/s. Until the remaining sediment is removed the capacity of the Truckee Canal will be limited by the capacity of the Lahontan Reach, regardless of the capacity of the improved areas.

5. Considerations for Future Alternative Development

As part of the risk reduction analysis for Alternative 1, the project team listed factors to be considered related to future design development, implementation, and O&M. They are:

- Concrete lining makes egress more challenging as compared to current conditions. Egress ladders will be required.
- Reshaping and improvement to the right embankment slope should be kept to a minimum as this slope is mostly in cut and in better condition as compared to the left.

- If implemented in relatively short lengths (approx. one mile every few years), this approach could incur excessive mobilization costs.
- The concrete lining and embankment height could be raised if future flow increases are needed/justified.
- Precast concrete panels could be considered instead of cast-in-place panels for ease of future replacement.
- Special considerations should be given to areas that transition from lined to unlined to avoid adverse flow conditions and erosion.
- This alternative should be used in combination with modifications to the Bango Check Structure operations and replacement of the Hazen Gauge. These structures are contributing to the sediment accumulation in the Lahontan Reach which ultimately controls the capacity of the Truckee Canal. Costs for replacement of the Hazen Gauge have not been included in the risk reduction plans as the LBAO has indicated the gauge will be replaced separate from the XM project.
- Earthwork quantities for this alternative were developed from a combination of the 2008 aerial survey and ground cross-section surveys performed after sediment cleaning. Feasibility level design should include a new, full aerial survey of the canal when dewatered to better estimate the required earthwork quantities.

[redacted]

E. Alternative 2 – Geomembrane/Concrete Cover (Left Canal Bank Only)

This alternative is similar to Alternative 1 except that the geomembrane/concrete cover lining system will only be placed against the left bank slope. To reduce the potential for seepage that might pass beneath the left bank lining, the geomembrane will be extended across the invert and protected by a compacted soil cover (see Figure V-3). The geomembrane will be secured in an anchor trench near the left embankment crest. The lined prism will have a minimum depth of 13.6-feet. At the 600 ft^3/s vegetated stage level this alternative provides about 5.3-feet of bank freeboard. Using concrete on only the left bank slope reduces the concrete volume and associated costs by about two-thirds.

1. PFMs Addressed by this Alternative

This alternative is being considered to address PFM1, PFM5, PFM10, PFM11 and PFM18. The primary benefit of this alternative is cutting off flaws and potential seepage pathways through the left embankment. Canal lining will also improve the conveyance capacity in those areas of the canal that are lined. This will allow for improved flood conveyance to discharge features being considered as part of this CAS. Lining the canal invert will also address PFM2-Internal Erosion through the Foundation, by reducing seepage into the foundation.

2. Construction and Long-term Maintenance

This lining system will also allow for incremental installation as funding becomes available. Construction will require a canal outage. Construction of the canal lining will need to be scheduled around periods when deliveries to the Truckee Division are not needed (i.e. fall and winter months). Concrete paving of only the left bank slope may be accomplished with hand placements or by using a paving machine (see Figure V-4).

Concrete lining of the left bank slope will eliminate the potential for burrowing animals and woody vegetation within left side of the canal prism. A burrowing animal and vegetation control program will still be required of the outer banks. Only one-third of the canal prism will be concrete lined. An aquatic vegetation control program will be required to clear the invert and right bank slope. Periodic sediment cleaning will also be required. Care must be taken as to avoid tearing the geomembrane in the invert during cleaning activities. The lining system will require periodic inspections, maintenance and replacement over time.

Figure V-4.—Example of partial prism canal paving equipment. (GOMACO, www.gomaco.com)

3. Seepage Reduction

This alternative is expected to reduce foundation seepage in those areas that are treated. Two-thirds of the canal prism will be lined with a geomembrane. As shown on Figure V-3 the right bank will remain unlined. Some lateral and downward seepage will continue from the right side of the canal prism.

4. Impacts to Canal Conveyance Capacity

In the segments where improved, the partial geomembrane/concrete cover lining is employed canal's capacity will be increased to about 1,350 ft^3/s.

This will allow for improved flood conveyance to discharge features being considered as part of this CAS. As discussed for Alternative 1, the conveyance capacity of the Truckee Canal will continue to be limited by those areas not improved with canal lining.

5. Considerations for Future Alternative Development

As part of the risk reduction analysis for Alternative 2, the project team listed factors to be considered related to future design development, implementation, and O&M. They are:

- Multiple lining systems (i.e. concrete/geomembrane and soil/geomembrane) complicates O&M.
- Soil along the invert and right prism slope will continue to require vegetation and sediment removal activities.
- Concrete lining makes egress more challenging as compared to current conditions. Egress ladders will be required.
- Precast concrete panels could be considered instead of cast-in-place panels for ease of future replacement.
- This alternative should be used in combination with modifications to the Bango Check Structure operations and replacement of the Hazen Gauge. These structures are contributing to the sediment accumulation in the Lahontan Reach which ultimately controls the capacity of the Truckee Canal. Costs for replacement of the Hazen Gauge have not been included in the risk reduction plans as the LBAO has indicated the gauge will be replaced separate from the XM project.
- Earthwork quantities for this alternative were developed from a combination of the 2008 aerial survey and ground cross-section surveys performed after sediment cleaning. Feasibility level design should include a new, full aerial survey of the canal when dewatered to better estimate the required earthwork quantities.

[redacted]

F. Alternative 3 – Geomembrane/Soil Cover Canal Lining (Full Canal Prism)

This alternative is similar to Alternative 1, except that the geomembrane will be covered with an 18-inch-thick compacted soil cover instead of concrete paving (see Figure V-5). The lining system will span the full canal prism and extend to the canal bank crest. Reshaping and improvement to existing soils is required to support the new lining, as well as to restore the originally designed canal invert profile. The geomembrane will be secured in an anchor trench near the embankment crest. The lined prism will have a minimum depth of 13.6-feet. At the 600 ft^3/s vegetated stage level this alternative provides about 3.2-feet of freeboard. This alternative is less robust than Alternative 1, as the geomembrane is more vulnerable to being damaged from burrowing animals, tree roots or torn during cleaning activities. In the absence of flaws, the geomembrane will practically eliminate both vertical and horizontal seepage losses.

1. PFMs Addressed by this Alternative

This alternative is being considered to address PFM1, PFM5, PFM11, and PFM18. The primary benefit of this alternative is cutting off flaws and potential seepage pathways through the embankment. The geomembrane liner will be extended to the embankment crest to provide protection against internal erosion in advance of overtopping during flood conditions. By extending the liner across the canal invert, the risks associated with internal erosion through the foundation will be lowered.

2. Construction and Long-term Maintenance

This lining system will also allow for incremental installation as funding becomes available. Construction will require a canal outage. Construction of the canal lining will need to be scheduled around periods when deliveries to the Fernley Division are not needed (i.e. fall and winter months). The geomembrane/soil cover system can be constructed with the District's forces or by a specialty contractor.

A burrowing animal and woody vegetation control program will be required. Burrowing animals and tree roots have the potential to create flaws in the geomembrane that could align with existing flaws in the embankment. An adequate inspection and maintenance program will be key to the success of this alternative.

The compacted soil cover will not eliminate or substantially reduce aquatic vegetation. An aquatic vegetation control program will be required. Periodic sediment cleaning will also be required. Care must be taken as to avoid tearing the geomembrane during cleaning activities. The lining system will require periodic inspections, maintenance and replacement over time.

Figure V-6.—Example of geomembrane and soil cover placement. (Huesker, www.huesker.com)

3. *Impacts to Canal Conveyance Capacity*

In the segments where improved, the geomembrane/soil concrete cover lining is employed canal's conveyance capacity will be increased to about 1,200 ft^3/s. As discussed for Alternative 1, the conveyance capacity of the Truckee Canal will continue to be limited by those areas not improved with canal lining.

4. *Considerations for Future Alternative Development*

As part of the risk reduction analysis for Alternative 3, the project team listed factors to be considered related to future design development, implementation, and O&M. They are:

- Sloughing of the soil cover is expected requiring repair. Long-term exposure of the geomembrane lining should be avoided.
- The soil cover will continue to require vegetation and sediment removal activities.
- This geomembrane/soil cover system is expected to have a shorter design life as compared to the geomembrane/concrete cover system.
- This alternative is expected to require more maintenance as compared to the geomembrane/concrete cover system.

- This alternative should be used in combination with modifications to the Bango Check Structure operations and replacement of the Hazen Gauge. These structures are contributing to the sediment accumulation in the Lahontan Reach which ultimately controls the capacity of the Truckee Canal. Costs for replacement of the Hazen Gauge have not been included in the risk reduction plans as the LBAO has indicated the gauge will be replaced separate from the XM project.
- Earthwork quantities for this alternative were developed from a combination of the 2008 aerial survey and ground cross-section surveys performed after sediment cleaning. Feasibility level design should include a new, full aerial survey of the canal when dewatered to better estimate the required earthwork quantities.

[redacted]

G. Alternative 4 – Embankment Cutoff Wall

This alternative includes installation of a cutoff wall through the left embankment of the canal to cutoff existing flaws (i.e. animal burrows, tree roots, and construction flaws). The key PFMs involve seepage and internal erosion along existing or seismically induced flaws in the embankment. The height of the embankment (fill thickness) in the highest risk areas typically ranges from about 5 to 10 feet. Therefore, a cutoff wall height of 15 feet was selected to fully intercept flaws in the embankment section and extend some distance into the foundation soils. A 15-foot embedment results in the base of the cutoff wall about 2 to 3 feet below the canal invert elevation. Extending the cutoff wall deeper into the foundation is not warranted as the PFMs involving internal erosion through the foundation do not result in unacceptable risk levels.

A cutoff wall is a viable improvement alternative for the Steam Pad Seep area. Instead of terminating the cutoff wall at a predefined embedment depth the cutoff wall would extend into the underlying claystone bedrock materials (Lower Lakebed Sediments) to create a positive seepage barrier. The depth to bedrock in these areas ranges from about 10 to 20 feet.

The cutoff wall is advantageous because installation can be incremental and can be installed during canal operations, whereas a lining system will require a canal outage. Where a segment of cutoff wall is terminated, care will be taken as to not exacerbate “end around” effects. The competency of the embankment at the terminal end will be evaluated and if there is a concern that flaws might exist, the wall would either be extended or a section of the embankment reconstructed to remove any flaws. The geometry of the embankment where the wall is terminated will also be considered as to avoid areas with elevated seepage gradients.

At the recently replaced turnouts in the Fernley Reach the cutoff wall will be extended into the reconstructed embankment section that was filtered (see Figure V-7). The cutoff wall will be extended laterally as close as practical to the turnout out conduit to avoid leaving existing embankment that has not been improved.

1. PFMs Addressed by this Alternative

This alternative is being considered to address PFM1, PFM5, PFM11, and PFM18. The primary benefit of this alternative is cutting off flaws and potential seepage pathways through the embankment (i.e. animal burrows and root systems). Once installed all viable seepage pathways through the embankment will be cutoff by the sheet pile wall. In the event of an earthquake that damages the embankment section, the cutoff wall (regardless of type selected) is expected to strengthen the embankment or deform with the embankment and limit seepage flows through cracks that might develop in the embankment.

2. Seepage Reduction

This alternative is not expected to significantly reduce foundation seepage where the cutoff wall is not extended to a "positive cutoff" layer. In most areas the cutoff wall would be terminated in the Upper Lakebed Sediment materials (soil like deposits). As shown on Figure V-8, downward seepage will likely continue from the canal prism where it will eventually reach the bedrock materials (Lower Lakebed Sediment materials). Some seepage will then permeate into the rock fractures or travel laterally along the soil/bedrock contact and exit downslope of the canal as observed in the upper Lahontan Reach.

If the bedrock contact is shallow, the cutoff wall could be extended and embedded in the Lower Lakebed Sediment materials to form a positive cutoff. Areas with shallow bedrock have historically been areas with seepage either exiting at the toe of the embankment or many hundreds of feet downslope. While the seepage which had been exiting downslope will likely diminish after installation of a cutoff wall, seepage in a downwards direction will likely continue.

Due the variability in the foundation conditions it would be very difficult to quantify seepage reduction provided by this alternative. This alternative will be very effective at cutting off flaws in the embankment and to address known areas with excessive seepage, if shallow bedrock can be used to form a positive cutoff.

[redacted]

3. Construction and Long-term Maintenance

Installation of the synthetic sheet piles can be completed during canal operations (no outages will be required). All work will be completed from the left embankment crest road within the existing right-of-way. First a shallow trench will be excavated along the alignment with a tracked excavator so the top of the sheet piles will remain below the existing crest elevation and to minimize sheet pile driving through the coarser road surfacing. Next a fixed-mast track-mounted vibratory hammer will install the sheet piles to the target depth. Subsurface investigations [8] indicate most of the areas to be improved consist of loose to medium dense silty sands and the installation rate is expected to be fast (150 to 300 linear feet per day).

About 0.5 to 0.75 miles of canal in the Ricci Lane area will likely require special installation methods such as pre-trenching, pre-driving, or pre-augering due to cobbly foundation materials. A synthetic sheet pile field trial will be held in this area in the fall of 2017. Findings from the study will be used to develop viable installation methods for the cobbly foundation soils.

Once installed and buried there will be no inspection or maintenance required. The design life of the synthetic sheet piles is expected to be in excess of 100 years.

Figure V-9.—Example of track-mounted, fixed-mast vibratory hammer equipment that would be used for the sheet pile installation. (CMI Shoreguard)

4. Impacts to Canal Conveyance Capacity

This alternative includes installation of a cutoff wall through the left embankment only. Modification to the canal prism geometry will not be required. There will be no change to the canal conveyance capacity as compared to existing conditions.

5. Considerations for Future Alternative Development

As part of the risk reduction analysis for Alternative 4, the project team listed factors to be considered related to future design development, implementation, and O&M. They are:

- The synthetic sheet pile wall system will require no long-term maintenance.
- The synthetic sheet pile wall system can be installed without a canal outage.
- This alternative is suited for incremental buildout.
- The earthen canal prism will continue to require vegetation and sediment removal activities.

[redacted]

H. Alternative 5 – Embankment Reconstruction

This alternative includes reconstruction of the existing left embankment and canal invert (see Figure V-10). The objective is to replace the left (downslope) canal embankment with an embankment without flaws. Materials excavated from the existing embankment will be used to construct the new embankment. The left half of the canal prism will be reshaped with a 2H:1V side slope. The canal invert will be returned to the original design profile.

1. PFMs Addressed by this Alternative

This alternative is being considered to address PFM1, PFM5, and PFM11. The primary benefit of this alternative is to eliminate flaws and potential concentrated seepage pathways through the embankment.

2. Seepage Reduction

This alternative is not expected to reduce foundation seepage in those areas that are improved. Downwards seepage will likely continue from the canal prism where it will eventually reach the bedrock materials. The seepage will then permeate into the rock fractures or travel laterally along the soil/bedrock contact.

3. Construction and Long-term Maintenance

A canal outage will be required for this work. Construction of this alternative includes excavation of the existing embankment and invert materials and then reuse of suitable excavated materials for reconstruction. Earthwork work will be staged such that the excavated materials are immediately placed in an adjacent area where the excavation has been completed. This will allow for the use of scrapers or large haul trucks to more quickly/efficiently complete the work and to avoid double-handling. The fill will be compacted with the scraper traffic and heavy compaction equipment. Shrinkage is expected as the fill will be placed in a denser state as compared to existing conditions. Imported fill from an offsite borrow will be required. The finished embankment will be similar in geometry to the existing conditions, except where possible the crest will have a minimum width of at least 25 feet. The existing left embankment toe will be preserved as to not expand the disturbed footprint or interfere with private land or other right-of-ways. The finished exterior slope will be treated with erosion control matting, topsoil and seeding. New gravel surfacing will be placed on the crest for the service road.

An aggressive burrowing animal and vegetation control program will be required for this alternative to not allow embankment flaws to develop in the future. This will include aquatic vegetation management and periodic canal prism cleaning.

4. Impacts to the Canal Conveyance Capacity

This alternative is not expected to significantly impact the canal conveyance capacity where the improvements are made. Some improvements to the conveyance capacity may be realized by reshaping the left prism slope and returning the canal prism to a uniform slope. The capacity of the Truckee Canal will be controlled by areas in the Lahontan Reaches as discussed in the Alternative 1 description section.

5. Considerations for Future Alternative Development

As part of the risk reduction analysis for Alternative 5, the project team listed factors to be considered related to future design development, implementation, and O&M. They are:

- Reconstructing the embankment with a minimum crest width will likely cause right-of-way impacts.
- The earthen canal prism will continue to require vegetation and sediment removal activities.
- The SOP would need to enforce the importance of maintaining the reconstructed embankment.
- This alternative is expected to require more maintenance as compared to the other linear alternatives.

[redacted]

I. Alternative 6 – Check Structure Replacement

This alternative includes replacement of the existing check structures with new structures having wider gate bays and weir openings to minimize the potential for ice and debris jams at the approach. The new check structures will be fitted with remotely operated gates to improve operational control and provide the ability to isolate canal segments in the event of a future canal failure.

There are four check structures, three in the Fernley Reach and one in the Lahontan Reach, that are being considered for replacement. They include the Fernley, Anderson, Allendale, and Mason Checks. The Fernley, Anderson and Allendale Check Structures are approaching their design life and should be reconfigured to minimize the potential for ice jams (see PFM discussion below). The Mason Check Structure is a non-gated structure and operated with the use of stop logs in four bay openings.

The Bango Check Structure was reportedly constructed in the 1970's and is in good condition. A review of the available survey indicates the invert of the Bango Check Structure was constructed above the original canal invert profile. This high point has reduced the average slope along the lower portion of the Lahontan Reach, which causes lower flow velocities and is believed to be contributing to sediment and aquatic vegetation accumulation. Historic operation in the Lahontan Reach has included checking the water surface at the Bango Check beyond the Mason Check location (about 5 miles upstream) to avoid changing of the stop log setting at the Mason Check. Replacement of the Mason Check with a gated structure will improve operational control in the upper Lahontan Reach will avoid the need for checking the water surface from the Bango Check.

1. PFMs Addressed by this Alternative

This alternative addresses PFM5. The Truckee Canal is unique in that it diverts flows from the Truckee River year round, when available. The canal has historically had issues with ice jams developing at constrictions in the canal and at the check structures (see Figure V-11). The ice jams have the potential to create a blockage leading to a rapid stage-level rise that can cause an internal erosion failure or eventually overtopping of the embankment. The new check structures will be designed to have wider gate and side weir openings to more easily pass ice flows. The remotely operated gates will also allow for canal isolation in the event of a future canal failure. Isolation of the canal will limit the volume of water that exits the breach thereby lowering the flood impacts and consequence levels.

Figure V-11.—Example of ice jams that have occurred historically at the check structure approaches.

2. Construction and Long-term Maintenance

A canal outage will be required for replacement of the check structures. The estimated construction duration is about 5 to 6 months for each check structure. If funding is available, multiple checks can be replaced at the same time. Ideally construction would be completed when deliveries in the Truckee Division are not needed. A system of cofferdams, pumps and pipes could be used to make deliveries downstream of the work area if needed.

Construction will initially include foundation improvement through excavation and structural backfill. A number of concrete placements will be required for the cutoff walls, base slabs, piers, weir walls and decks. Once all of the concrete placements have been made, the canal could be put back in operation during installation of the mechanical equipment. Mechanical equipment includes, radial gates, hoists, electrical controls, control building, SCADA system, and backup power system.

Maintenance of the new check structures will be similar to what is required for the existing structures. Periodic gate rehabilitation, coating reapplication, and concrete repair will be required.

3. Impacts to Canal Conveyance Capacity

During winter operations, the radial gates will be lifted completely above the water surface. The “unchecked” capacity of the new check structures openings will be at least 1,200 ft^3/s.

[redacted]

J. Hydrologic Design Criteria

As discussed previously, flood protection features will be designed in cooperation with the linear canal embankment improvements to both address the potential for embankment overtopping and minimize the potential for internal erosion and piping (prior to overtopping). To accomplish this, flood protection features should be sized such that the stage level is not allowed to rise more than 1-foot above the 600 ft^3/s vegetated stage level in those areas not improved by the linear embankment improvements (i.e. lining, cutoff wall, or reconstructed embankment) to facilitate the long-term canal flow rate. Where the linear embankment improvements are implemented, the flood protection features should be designed so that at least 1-foot of canal bank freeboard is maintained. To accomplish this, a combination of protective features such as drainage crossings, spillways/wasteways, upslope storage and detention ponds will be sized to meet the above criteria. The following sections describe the hydrologic protective features considered by the CAS team.

K. Alternative 7 – Drainage Crossings and Channels

This alternative includes construction of drainage crossings over the Truckee Canal (chutes) and improvement to existing drainage channels downslope of the canal (see Figure V-13). The drainage crossings will be used to convey rainfall runoff at the major pour points to the left downslope side of the canal. Channelization of the flows from the right upslope side of the canal will be required to route the runoff to the drainage crossing structures. The existing channels on the left downslope side of the canal will need to be enlarged to convey the runoff.

1. PFMs Addressed by this Alternative

This alternative is being considered to address PFM10 and PFM11. Drainage crossings will be used to eliminate/reduce large runoff inflows into the canal. Drainage crossing would be installed at pour points which have the potential to cause the largest runoff inflows.

2. Construction and Long-term Maintenance

Construction of this alternative includes excavation of existing embankment, forming and placing concrete for the foundations and piers, replacing embankment material, forming and placing concrete for the chute from one side of the canal to the other, and channel improvements on both sides of the canal. A brief canal outage will be required during construction of the foundations for the reinforced concrete piers. Once the piers are placed, the canal can be put back in operation. Improvement to the downslope drainage channel will include numerous roadway and railroad culvert crossings. This work can be done while the canal is in operation.

A regular cleaning schedule of the drainage crossing approach channels and downslope drainage channels will be required. The drainage crossings are passive, and do not require intervention by TCID during the flooding event.

3. Considerations for Future Alternative Development

As part of the risk reduction analysis for Alternative 7, the project team listed factors to be considered related to future design development, implementation, and O&M. They are:

- Additional ROW will be required along the length of the newly improved channels.
- This alternative routes runoff over the Truckee Canal. Decision makers should better understand whether Reclamation is responsible for downslope channel improvements, if the runoff is allowed to pass the canal.
- The SOP would need to enforce the importance of maintaining the drainage channels, especially at the crossing approach.
- This alternative could be combined with a downslope detention pond in lieu of enlarged drainage channels. The detention pond could be used to release the runoff at a controlled rate as to not exceed the existing channel capacities.

[redacted]

Figure V-14. Example of an Overchute Drainage Crossing.

L. Alternative 8 – New Gated Wasteway(s)

This alternative includes construction of new gated wasteway(s) through the left embankment and enlargement of existing drainage channels downslope of the canal (see Figure V-15). The gated wasteways would be used during a flood event to prevent a rapid stage-level rise and/or overtopping. The gated wasteways would be remotely operated to allow for rapid response in the event of flooding or should a breach occur and there is a need to reduce the volume of water released from the breach. The existing drainage channels on the left downslope side of the canal would need to be enlarged to convey flows of about 600 ft^3/s each.

1. PFMs Addressed by this Alternative

This alternative is being considered to address PFM10 and PFM11. The automated wasteways will provide operators the ability to quickly respond to rapid stage-level rise or drain the canal in the event of a breach. The wasteway(s) would be located near or downstream of the largest pour points contributing the potential stage-level rise.

2. Construction and Long-term Maintenance

A canal outage will be required to construct the new wasteways. Construction of this alternative includes excavation of existing left embankment, forming and placing reinforced concrete for the wasteway structure, replacing embankment material, installation of gates and controls, and channel improvements on the left downslope side of the canal. Improvement to the downslope drainage channel will include numerous roadway and railroad culvert crossings. This work can be done while the canal is in operation.

Regular maintenance and operation of the wasteway structure will be required to ensure it can be used during a flooding event. A regular cleaning schedule of the downslope drainage channels will be required.

This alternative would need to be combined with the linear embankment improvement alternatives to safely convey the runoff through the canal until it can be released further downstream. Viable locations for the wasteways are mostly in the lower Fernley Reach or throughout the Lahontan Reach. Enlarging the drainage channels through the City of Fernley would be difficult due to right-of-way limitations.

The wasteway gates could be opened automatically by establishing stage-level targets with the existing automated staff gauge monitoring equipment. Alternatively, alarms could be triggered that would require TCID to open the wasteway gates.

3. Considerations for Future Alternative Development

As part of the risk reduction analysis for Alternative 8, the project team listed factors to be considered related to future design development, implementation, and O&M. They are:

- This alternative has challenges in that right-of-way will be required to enlarge the drainage channels. Also a number of large roadway and railroad culvert crossings will be required. The drainage channels will likely cost more than the wasteways themselves.
- Drainage channels should be located where Reclamation and TCID have existing right-of-way for irrigation channels.
- This alternative discharges runoff entering the Truckee Canal from one location to another drainage location. Decision makers should better understand whether Reclamation is responsible for downslope channel improvements, if the runoff is allowed to pass the canal.
- The SOP would need to list requirements for regular maintenance and operation of the wasteways.

[redacted]

M. Alternative 9 – New Passive Spillway(s)

This alternative includes construction of new passive spillway(s) through the left embankment and improvement to existing drainage channels downslope of the canal (see Figure V-16). The passive spillways are being considered to minimize the potential for a rapid stage-level rise or overtopping as a result of flood inflows or mis-operation. The sill of the passive spillway will be set at the 600 ft^3/s vegetated stage level. As the stage level rises above the spillway sill, water will be discharged from the canal. The existing drainage channels on the left downslope side of the canal would need to be enlarged to convey flows of about 300 ft^3/s each.

1. PFMs Addressed by this Alternative

This alternative is being considered to address PFM10 and PFM11. The passive spillways reduce the potential for a rapid stage-level rise and/or overtopping. The spillway(s) would be located near or downstream of the largest pour points contributing the potential stage-level rise.

2. Construction and Long-term Maintenance

A canal outage will be required to construct the new passive spillways. Construction of this alternative includes excavation of existing embankment on the left bank, forming and placing reinforced concrete for the spillway structure, replacing embankment material, and channel improvements on the left downslope side of the canal. Improvement to the downslope drainage channel will include numerous roadway and railroad culvert crossings. This work can be done while the canal is in operation.

Regular maintenance and operation of the spillway structure will be required to ensure it can be used during a flooding event. A regular cleaning schedule of the downslope drainage channels will be required.

This alternative would need to be combined with the linear embankment improvement alternatives to safely convey the runoff through the canal until it can be released further downstream. Viable locations for the spillways are mostly in the lower Fernley Reach or throughout the Lahontan Reach. Enlarging the drainage channels through the City of Fernley would be difficult due to right-of-way limitations.

The spillways as configured are passive structures, and will not require intervention by TCID to make releases during a flood event.

3. *Impacts to the Canal Conveyance Capacity*

This alternative will limit the water surface from rising above a certain level. Tentatively the spillway sill will be set at the 600 ft^3/s vegetated stage level. Canal flows above this level will result in discharges from the spillways. If a higher stage level is permitted following linear canal embankment improvements, then stop logs or flash boards could be used to raise the spillway sill and permit higher canal flows.

4. *Considerations for Future Alternative Development*

As part of the risk reduction analysis for Alternative 8, the project team listed factors to be considered related to future design development, implementation, and O&M. They are:

- This alternative has challenges in that right-of-way will be required to enlarge the drainage channels. Also a number of large roadway and railroad culvert crossings will be required. The drainage channels will likely cost more than the spillways themselves.
- Drainage channels should be located where Reclamation and TCID have existing right-of-way for irrigation channels.
- This alternative discharges runoff entering the Truckee Canal from one location to another drainage location. Decision makers should better understand whether Reclamation is responsible for downslope channel improvements, if the runoff is allowed to pass the canal.
- The SOP would need to list requirements for regular maintenance and operation of the spillways.

[redacted]

N. Alternative 10 – Detention/Infiltration Pond(s)

This alternative includes construction of detention pond(s) at select locations where there is the potential for large runoff into the canal. Detention ponds are used to retain the runoff volume and then release the water from the pond in a controlled manner. The 2016 hydrologic hazard analysis study [6] indicated Pour Points No. 8, 16 and 19 have the potential to produce a cumulative peak discharge of about 1,600 ft^3/s (about 60 percent of the potential total inflows in the Fernley and Lahontan Reaches) during the 100-year, 6-hour event. Inflows from Pour Points No. 8, 16 and 19 have the potential to result in a rapid stage-level rise and/or overtopping.

Pour Point No. 8 in the Fernley Reach was selected for development of an appropriately sized detention pond to attenuate the 100-year, 6-hour runoff hydrograph [6]. The selected detention pond site is located on the west side of U.S. Alt. 95, just south of the canal (see Figure V-17). This area is currently undeveloped but approved for expansion of a nearby housing development.

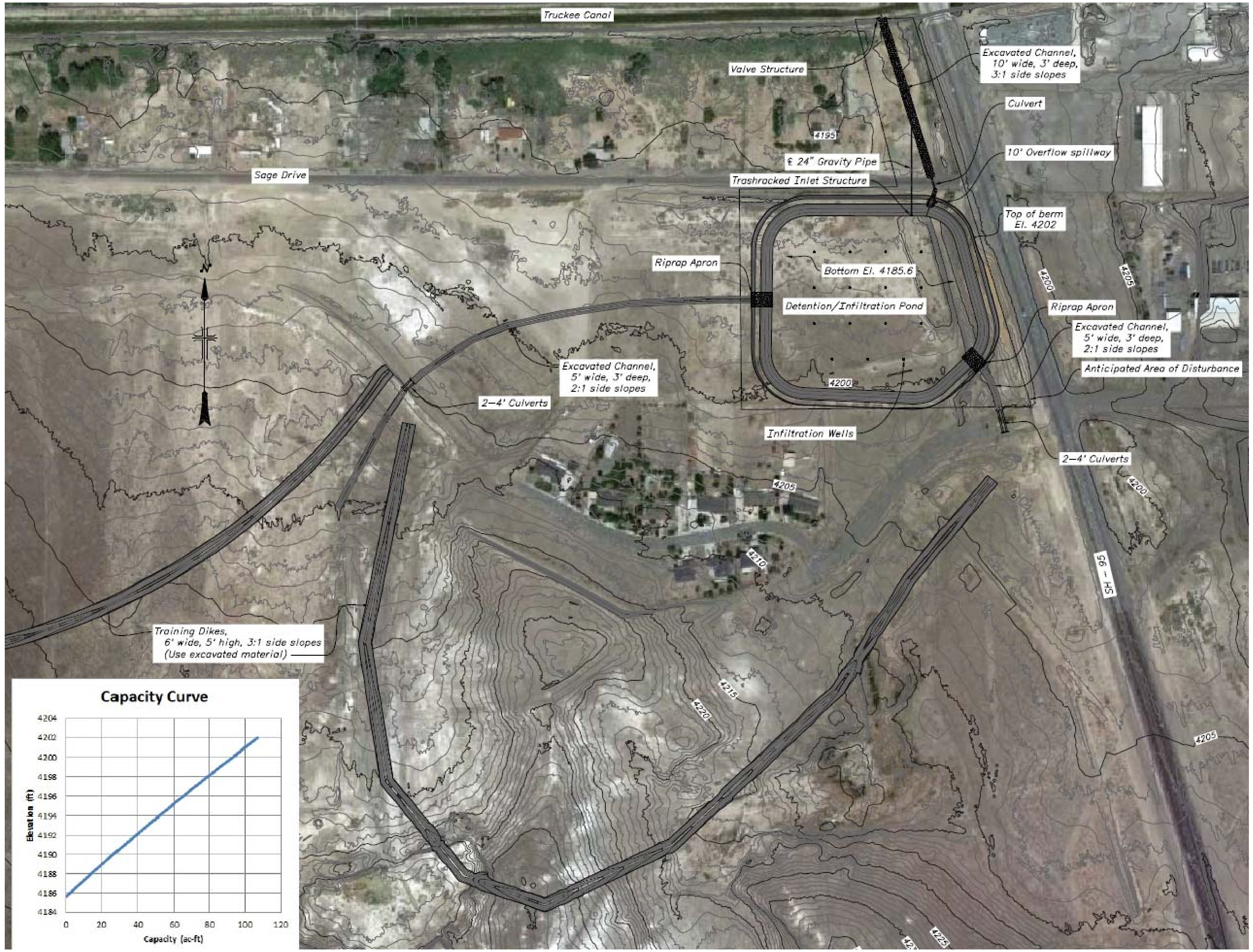


Figure V-17.— Detention Pond Site Plan at Pour Point 8

Runoff at Pour Point No. 8 originates from a 45 square mile basin which drains onto an alluvial fan south of canal alignment. Distributive surface runoff at the toe of the alluvial fan will be directed into the detention pond with training dikes and excavated channels. The training dikes will be about 3 to 5 feet high and constructed from materials obtained during the detention pond excavation. The excavated channels will then carry the runoff into the detention pond. The channels will be about 5 feet wide, 3 feet deep, and have 2H:1V side slopes. Riprap aprons will be used where the channels enter the detention pond.

The detention pond will be mostly excavated below the existing grade. The excavated material will be used to construct the training dikes and a low-height containment berm along the north side of the pond. The maximum height of the containment berm is 6 feet with a top elevation of 4202. The pond side slopes will be 2H:1V or flatter. The base elevation of the pond will extend below the invert of the canal (elevation 4185.6). As an ancillary benefit, this will allow delivery of water from the canal into the detention pond and could allow infiltration as part of the City of Fernley's aquifer storage recharge program, discussed further below.

The detention pond volume was sized to retain the full 100-year, 6-hour event hydrograph volume (Figure V-18). The runoff volume from the 100-year, 6-hour event is expected to be about 67 acre-feet.

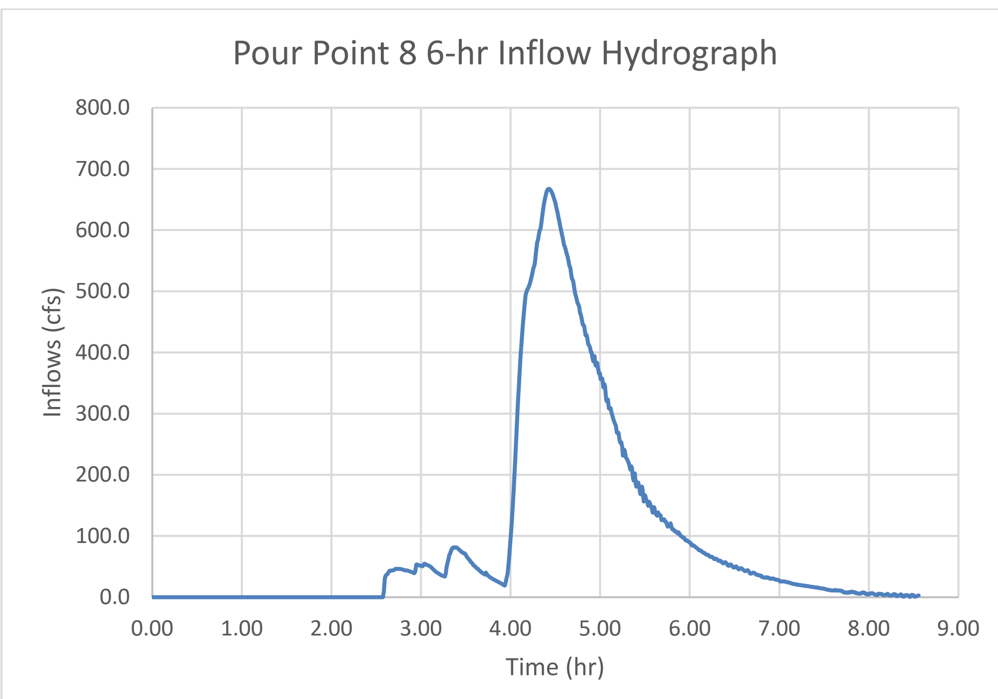


Figure V-18.—6-hr Inflow Hydrograph for Pour Point 8

After the storm, water retained in the detention pond can be slowly released into the Truckee Canal through a 24-inch buried gravity pipe. The pipe flow will be regulated by a gated turnout structure along the right side of the canal just west of the U.S. Alt. 95. A trash-racked intake structure will be used where the buried pipe enters the detention pond (Figure V-19).

Figure V-19.— Example of trashrack outlet structure.

An overflow spillway notch will be constructed on the north east corner of the detention pond. The spillway was sized to pass an additional 20 percent flow beyond the anticipated 100-year, 6-hour peak discharge. The spillway is 10 feet wide, has 2H:1V side slopes and sill 3 feet below the berm crest (elevation 4199). Articulated concrete masonry unit blocks will be used to armor the spillway and protect it from erosion should the spillway operate (Figure V-20). Flow from the spillway would be conveyed along a riprap lined channel. The channel would be 10 feet wide, 3 feet deep, and have 3H:1V side slopes.

Figure V-20.— Example of an Articulated CMU block spillway (Contech Engineered Solutions)

The City of Fernley's municipal water supply is currently obtained from groundwater wells throughout the city limits. Features could be added to the Pour Point No. 8 detention pond that would promote infiltration and allow the City to utilize its surface water rights to augment the aquifer recharge program they are undertaking. The buried 24-inch gravity pipe for draining the detention pond, can also be used to deliver water from the canal into the pond. A flow measurement gauge would be installed to meter the deliveries. Drilled shafts, about 18 inches in diameter, would be installed throughout the base of the detention pond and then backfilled with free draining aggregate. The drilled shafts are tentatively 100 feet deep and will allow recharge of the cities groundwater supply. The 350 ft^3/s stage level would maintain about 2 to 3 feet of water in the detention pond. The flood retention volume was calculated above this level. Figure V-21 includes a cross section which shows how the detention pond would be used as an aquifer recharge feature. A subsurface investigation program will be required to investigate the viability of an infiltration pond at the Pour Point No. 8 site. If the infiltration function is viable, the detention pond at Pour Point No. 8 could be developed and cost shared with the City of Fernley. The area around the detention pond could be developed as an open space park. Figure V-22 shows an example of how the site could be developed.

The detention pond sized for Pour point No. 8 could also be used at Pour Points No. 16 and 19. The infiltration function would not be required at these sites.

[redacted]

Figure V-22.— Example of how the Detention Pond could be developed as a City Park.

1. PFMs Addressed by this Alternative

This alternative is being considered to address PFM10 and PFM11. A combination of detention ponds and other hydrologic protective features are being used in combination to limit the stage-level rise and eliminate the potential for overtopping from the 100-year, 6-hour loadings.

2. Land Acquisition and Maintenance

The potential site at Pour Point No. 8 has been approved for additional housing units. If Reclamation and TCID choose to pursue this alternative, communications with the City of Fernley and the housing developers should be initiated as soon as possible. Reclamation and TCID should work with the City of Fernley to obtain these lands for future construction of the detention pond once funding is available. The potential sites at Pour Points No. 16 and 19 are rural and either fully or partially lie within Reclamation withdrawn lands. Various permits will also be required.

Periodic removal of sediment and debris from the detention ponds will be required following storms which produce runoff. If installed, the infiltration wells may require periodic flushing. Regular maintenance of the drainage channels, training dikes, berms and erosion protection features will be required. This work may be shared between TCID and the City of Fernley if the surrounding area is developed as an open space park. Maintenance of the turnout structure would be the responsibility of TCID.

O. Requirements in the Lahontan Reach to Improve the Conveyance Efficiency

1. Management of Sediment Accumulation

Sediment accumulation occurs in the Truckee Canal requiring periodic cleaning. In 2012 and 2014 about 2 to 3 feet of sediment was removed along the Fernley and upper Lahontan Reaches (see Figure V-23). A survey in January of 2015 indicates some of this sediment remains in the lower portion of the Lahontan Reach.

The updated 2015 HEC-RAS model indicates the remaining sediment in the lower portion of the Lahontan Reach is causing an elevated stage level extending into the Fernley Reach. The January 2015 survey also indicated the sediment removal was sporadic and did not result in a uniform canal invert slope. Areas where little or no sediment was removed are resulting in a constriction and causing an increase in the stage.

As part of the long-term risk reduction measures TCID should re-establish the canal invert to the originally designed profile along the length of the canal. This should be done with survey equipment while the sediment removal is being completed. The volume and amount of sediment being removed should be recorded to aid in future understanding of where the sediment is coming from, rate at which it is accumulating, and measures that might minimize future sedimentation. <u>Going forward, TCID should survey the canal invert at least once every three years and maintain the canal invert within one foot of the originally designed invert elevation to optimize conveyance capacity</u>. Checking of the canal water surface in the Lahontan Reach should be minimized to avoid slower flow velocities which have likely contributed to the sediment accumulation.

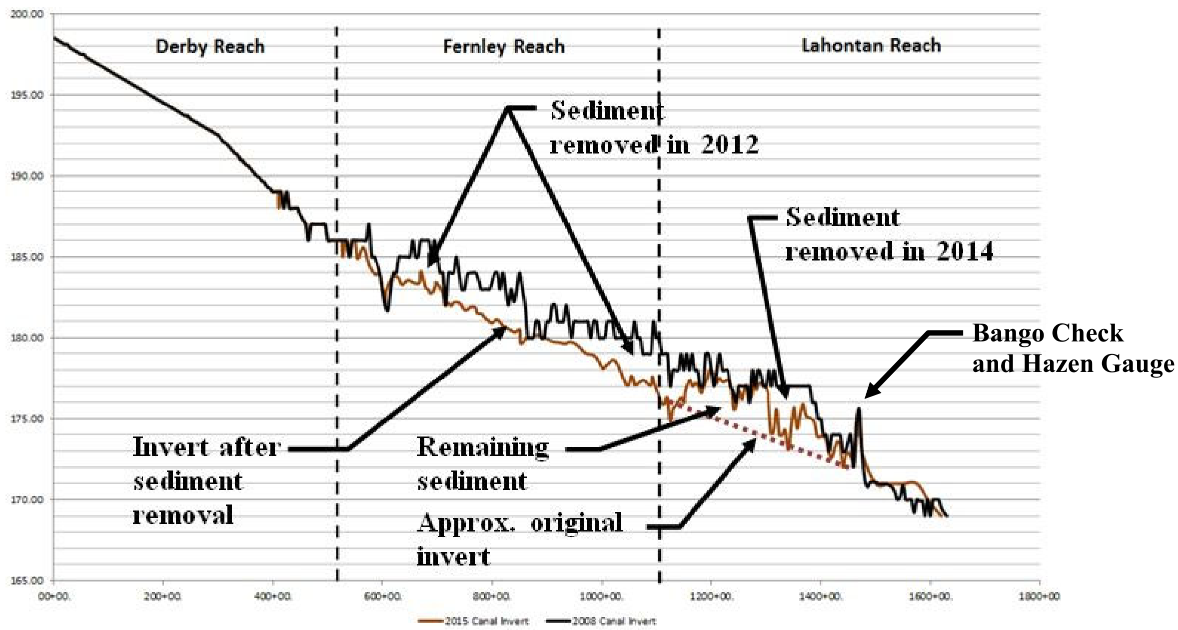


Figure V-23.—A comparison of the 2008 and 2015 canal bed elevations.

2. Removal and Control of Vegetation

The HEC-RAS model study developed as part of the updated risk analysis study [1] evaluated the effects of vegetation throughout the year. Vegetation has the potential to raise the stage level 1 to 4 feet for a given flow rate and considerably decrease the flow rate for a given stage level. These effects reportedly vary year to year, and vary along the length of the canal. Vegetation is more prominent in the lower Fernley Reach and throughout the Lahontan Reach.

During the summer months TCID should inspect the canal daily and in areas which develop extensive vegetation employ mechanical means for removing the vegetation. Where loose vegetation accumulates at hydraulic structures it should be removed as soon as possible.

3. Changes to the Bango Check Operations

The Bango Check has historically been used to check the water surface in the Lahontan Reach extending upstream of the Mason Check location (about 5 miles upstream). Checking the water surface in the lower Lahontan Reach has apparently attributed to sediment accumulation and aquatic vegetation development. The sediment and vegetation load in addition to reduced freeboard when the water surface is checked attribute to a higher risk of hydrologic overtopping. Checking the water surface higher than what is needed to make deliveries at turnout TC-13 should be avoided.

4. Replacement of the Hazen Gauge

Field survey measurements and the HEC-RAS model study developed as part of the updated risk analysis study [1] indicate that the Hazen Gauge has a sill elevation well above the original canal profile (see Figure V-23). This structure is causing the stage level to backup throughout the Lahontan Reach which is causing slower velocities and is contributing to the sediment accumulation.

The existing Hazen Gauge is a combined low flow V-notch and broad-crest weir. The sill of the weir is about 3 feet above the canal invert (see Figure V-24). This configuration "checks" the water surface and slows the flow velocity upstream of this location. The slower velocities are contributing to the sediment and aquatic vegetation accumulation in the lower Lahontan Reach. The CAS recommends the weir measurement device be replaced with a long-throated flume (see Figure V-25). This type of structure has less backwater effects. The estimated cost for replacing the Hazen Gauge is about $115,000. Costs for replacement of the Hazen Gauge have not been included in the risk reduction plans as the LBAO has indicated the gauge will be replaced separate from the XM project.

Figure V-24.— Existing Hazen Gauge

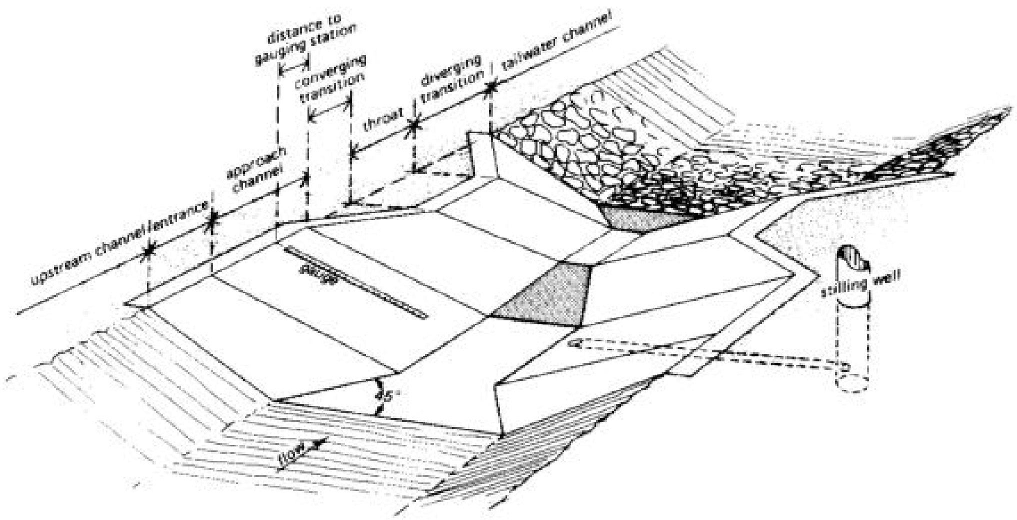


Figure V-25.— Example of a Long-throated Flume

VI. Risk Reduction Analysis

A risk reduction analysis meeting was held at the TSC the week of May 2, 2016 to evaluate how the CAS alternatives would lower the risks posed by the controlling PFMs. The risk analysis team (RET) consisted of project team members from the TSC, staff and management from the LBAO, staff from the MP region, and representation from TCID. Most of the RET members from the 2014 updated risk analysis study [1] participated in the risk reduction analysis. As identified in the 2014 updated risk analysis study, the controlling PFMs for the Truckee Canal are;

- PFM1 – Internal Erosion through the Embankment
- PFM5 – Ice Jams leads to Internal Erosion through the Embankment or Embankment Overtopping
- PFM10 – Flooding Leads to Embankment Overtopping
- PFM11 – Flooding Leads to Stage-level rise and Internal Erosion through the Embankment
- PFM18 – Seismic Induced Cracking leads to Internal Erosion through the Embankment

The CAS alternatives were developed to address internal erosion through the embankment and to minimize the potential for flood inflows that might cause a rapid stage-level rise or overtopping. The risk reduction analysis evaluated to what degree each of the CAS alternatives addressed these key potential failure modes (i.e. from high to low likelihood).

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Figure VI-1.—Example of Seismic Related Embankment Cracking, All American Canal Lateral, 2010.

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VII. Appraisal-level Cost Estimates

Appraisal-level quantities and field construction cost estimates were prepared for each of the CAS alternatives individually. The quantities and cost estimates for each of the CAS alternatives are included in Appendix A. Those costs were then used to develop total field costs and an estimate of the costs over time for each of the risk reduction alternative plans described in Section IX. The risk reduction alternative plans are expected to span more than 50 years and expenditures over time will vary for each of the alternative plans. Therefore, the appraisal-level cost estimates were developed using material, labor and equipment rates for the year 2016. Escalation should be considered during the feasibility-level study when there is a better understanding of which risk reduction plan will be carried forward into final design and how the project will be funded over time.

The following information and assumptions were used when developing the appraisal-level quantities and cost estimates:

- Topography generated from the 2008 aerial survey and from ground surveys completed in 2013 and 2014 following sediment removal activities were used to develop earthwork quantities.

- Material quantities were measured from design sketches shown on figures in Section V.
- Quantities for the linear canal embankment improvement alternatives were made for Phase I only (approx. 6.0 miles of canal). Costs per linear foot for Phase I were then used to estimate the costs of Phase II (approx. 5.8 miles of canal).
- Costs were prepared for the individual hydrologic protective features. Costs were also developed for a number of downslope drainage channels. Total costs for the hydrologic protective features vary depending on the combination of features included in the risk reduction plan.
- The addition of a 5 percent mobilization cost.
- The addition of a 10 percent design contingency for unlisted design items.
- The addition of a 20 percent contingency for changes during construction.
- Costs for escalation were not included.
- Noncontract costs for design data collection, design, permitting and compliance were not included as this work has already been funded as part of the final design project(s) with the TSC and the EIS with Environmental Management and Planning Solutions, Inc. (EMPSi).
- Noncontract costs for contract administration, construction management and documentation were not included and are to be developed by the client office.
- Costs for land acquisition were provided by the LBAO. For lands to be acquired in the City of Fernley a rate of 110,000 $/acre was assumed and in the surrounding rural areas a rate of 23,000 $/acre was assumed.

Table VII-1 summarizes the appraisal-level construction cost estimates for the individual alternatives. The alternatives have been used in combination to prepare risk reduction plans, described in Section IX.

Life cycle costs were not considered during this CAS. The feasibility-level study should consider life cycle costs for alternatives requiring significant maintenance and replacement costs (i.e. concrete lining alternatives and mechanical hydraulic control structures).

Table VII-1.—Appraisal-level cost estimates

Alternative	Estimated Phase I Field Cost (cost per linear foot)
Alt. 1 – Geomembrane/Concrete Cover Canal Lining (Full Canal Prism)	$28,000,000 ($885) [1]
Alt. 2 - Geomembrane/Concrete Cover Canal Lining (Left Canal Bank Only)	$16,500,000 ($520)
Alt. 3 – Geomembrane/Soil Cover Canal Lining (Full Canal Prism)	$14,000,000 ($440)
Alt. 4 - Embankment Cutoff Wall	$14,000,000 ($440)
Alt. 5 - Embankment Reconstruction	$31,000,000 ($980)
Alt. 6 - Check Structure Replacement	$2,800,000 each
Alt. 7 - Drainage Crossings and Channels [2]	$480,000 each
Alt. 8 - New Gated Wasteway(s) and Channels [2]	$2,000,000 each
Alt. 9 - New Passive Spillway(s) and Channels [2]	$230,000 each
Alt. 10 - Detention/Infiltration Pond(s)	$6,800,000 each
Alt. 11 – Replace the Hazen Gauge	$115,000

Notes: [1] Costs in parenthesis represent the estimated cost per linear foot for the linear canal embankment improvement alternatives. [2] Costs for the drainage channels vary depending on the location and range from $5,500,000 to $12,000,000.

VIII. Alternatives Screening Evaluation

To aid in selection of the preferred risk reduction alternative(s) and to identify viable combinations of the alternatives, a series of evaluation criteria were established. The screening criteria include; 1) risk reduction, 2) cost, 3) constructability, 4) operations and maintenance, 5) efficiency improvements. Table VIII-1 summarizes a brief discussion of how each risk reduction alternative affects the screening criteria.

A scoring system with values from 1 to 5 were assigned to each of the risk reduction alternatives. Each of the screening criteria were weighted to calculate a weighted score. The following weighting values were selected for each criteria; risk reduction (30%), cost (25%), constructability (20%), operations and maintenance (15%), and efficiency improvements (10%).

Only the linear canal embankment alternatives (1 through 5) and the hydrologic protective features (7 through 10) could be compared to one another. No scores were assigned to Alternative 6 (check structure replacement), as this alternative will be included in each of the risk reduction plans.

Results of the screening analysis indicate Alternative 4 (embankment cutoff wall) receives the highest score for the linear canal embankment improvement alternatives. This is due to the degree of risk reduction, lowest construction costs, and ability to construct this alternative without a canal outage. The next highest score was Alternative 1 (geomembrane/concrete cover full lining system). Alternative 1 also provides adequate risk reduction but also reduces O&M requirements and eliminates seepage losses.

The third highest score was Alternative 2 (geomembrane/concrete cover half lining system). It provides adequate risk reduction but less improvement to O&M requirements and efficiency as compared to alternatives which line the full canal prism.

Alternatives 3 (geomembrane/soil cover full prism) and Alternative 5 (embankment reconstruction) <u>do not provide adequate risk reduction in areas where the consequence level is 3 (Fernley Reach)</u>. Alternatives 3 and 5 may be appropriate for the Lahontan Reach. Alternative 3 is most suitable for areas where seepage loss reduction is the primary objective.

Alternative 10 (detention / infiltration ponds) received the highest score for the hydrologic protective feature alternatives. This is due to the ability to reduce or eliminate flood inflows at the largest drainage crossings and eliminates the need for costly drainage channels downstream of the canal required for the other alternatives. There may be an opportunity to partner with the City of Fernley to allow detention ponds in the Fernley Reach to serve as infiltration ponds to assist with their aquifer recharge program. The next highest score was Alternative 8 (new gated wasteways). Alternative 8 provides the ability to discharge flood inflows at select locations to limit the stage-level rise in the canal and can be used to limit outflows in the event of a future breach.

Alternatives 7 and 9 can be used to reduce the hydrologic risks, but a number of these features will be required to achieve the same risk reduction a single gated wasteway can achieve.

Table VIII-1.—CAS Alternatives Screening Evaluation

Alternative	[redacted]	Cost	Constructability	Operations and Maintenance	Efficiency Improvements	Weighted Score
Alt. 1 – Geomembrane/Concrete Cover Canal Lining (Full Canal Prism)		This alternative has the second highest cost per linear foot. There will also be lost benefits during construction due to outage. Score: 1	Construction of the canal lining will require multiple lengthy canal outages. Canal lining allows for a phased construction approach. Score:2	The geomembrane / concrete lining materials will require annual inspection, periodic repair and replacement. Less vegetation management will be required. Less sedimentation affects. Burrowing animals will be limited to the outer embankment slopes. Score:4	A full lining system would reduce seepage losses to near zero in those areas that are improved. Score:5	3.3
Alt. 2 - Geomembrane/Concrete Cover Canal Lining (Left Canal Bank Only)		This alternative is less expensive than the full prism lining alternatives lining. There will also be lost benefits during construction due to outage. Score: 4	Construction of the canal lining will require multiple lengthy canal outages. Canal lining allows for a phased construction approach. Score:3	The geomembrane / concrete lining materials will require annual inspection, periodic repair and replacement. Concrete replacement costs can be high. The invert and right bank will be soil lined. Vegetation management will be required. Burrowing animals will be limited to the outer embankment slopes. Score:2	A partial lining system would reduce seepage losses in those areas that are improved. Some seepage from the right side of the prism would continue. Score:3	3.4
Alt. 3 – Geomembrane/Soil Cover Canal Lining (Full Canal Prism)		This alternative is less expensive than the full prism lining with a concrete overlay. This alternative has the lowest cost per linear foot. Score: 5	Construction of the canal lining will require multiple lengthy canal outages. Canal lining allows for a phased construction approach. Score:2	The geomembrane will require periodic repair and replacement. Inspection of the geomembrane will be difficult. Slumping of the soil cover is expected. Vegetation management will be required. An enhanced burrowing animal control program will be required. Score:3	A full lining system would reduce seepage losses to near zero in those areas that are improved. Score:5	3.2

Alternative	[redacted]	Cost	Constructability	Operations and Maintenance	Efficiency Improvements	Weighted Score
Alt. 4 - Embankment Cutoff Wall		This alternative has the lowest cost per linear foot. Score: 5	A synthetic sheet pile cutoff wall can be installed from the crest of the existing embankment. Minimal earthwork is required. A canal outage is not required . A cutoff wall could be installed with TCIDs' forces. A cutoff wall allows for a phased construction approach. Score: 5	The cutoff wall will require no maintenance. Vegetation management will be required. A burrowing animal control program will be required. Score:4	A 15-foot deep cutoff wall will limit lateral seepage through the embankment and shallow foundation soils. Downward seepage from the canal's prism and then lateral spreading along the bedrock layers will continue. Overall seepage reduction will be minimal. A cutoff wall could be used to provide a positive seepage barrier where the bedrock is shallow. This alternative will not improve the canal's conveyance capacity. Score:1	4.5
Alt. 5 - Embankment Reconstruction		This alternative has the highest cost per linear foot. Score: 1	A large volume of earthwork will be required. Large staging areas may be required. Reconstructing the embankment will require multiple lengthy canal outages. Embankment reconstruction allows for a phased construction approach. Score: 2	Vegetation management will be required. An enhanced burrowing animal control program will be required. Score:3	This alternative includes recompaction of the canal invert materials. This will reduce seepage losses. The canal's prism could be reshaped to improve the conveyance capacity. Score:2	1.9
Alt. 6 - Check Structure Replacement		Automation of the checks results in elevated costs.	A canal outage will likely be required during construction. The checks can be replaced individually, or all at once by a larger contractor.	Additional inspection and maintenance of the new check structures, radial gates and automation equipment will be required as compared to current practices.	The new automated check structures will allow for improved operational control. No changes to seepage losses expected.	No Alternative Comparison

Alternative	[redacted]	Cost	Constructability	Operations and Maintenance	Efficiency Improvements	Weighted Score
Alt. 7 - Drainage Crossings and Channels	[redacted]	While the drainage crossing features are less than gated wasteway structures, the cost of construction or improvements to drainage channels downstream of the canal may be prohibitive. Multiple drainage channels will be required. Score: 3	Construction of the drainage crossings will require a canal outage. Improvements to the drainage channels downstream could be done in phases. Obtaining drainage channel right-of-way may be challenging. Large culverts under highways and railroads will be required. Score: 3	Frequent cleaning of the drainage crossing structures and downstream drainage channels will be required. Score: 3	No changes to the canal's efficiency expected from this alternative. Score: 3	2.7
Alt. 8 - New Gated Wasteway(s)	[redacted]	The automated wasteway structures are expensive. The cost of construction or improvements to drainage channels downstream of the canal may be prohibitive. A large drainage channel will be required at each wasteway location. Score: 2	Construction of the wasteways will require a canal outage. Improvements to the drainage channels downstream could be done in phases. Obtaining drainage channel right-of-way may be challenging. Large culverts under highways and railroads will be required. Score: 3	Additional inspection and maintenance of the new wasteway structures, radial gates and automation equipment will be required as compared to current practices. Frequent cleaning of the downstream drainage channels will be required. Score: 2	No changes to the canal's efficiency expected from this alternative. Score: 3	2.9
Alt. 9 - New Passive Spillway(s)	[redacted]	The passive spillway structures are less expensive than automated wasteways. The cost of construction or improvements to drainage channels downstream of the canal may be prohibitive. A large drainage channel will be required at each wasteway location. Score: 3	Construction of the passive spillway will require a canal outage. Improvements to the drainage channels downstream could be done in phases. Obtaining drainage channel right-of-way may be challenging. Large culverts under highways and railroads will be required. Score: 3	Inspection and repair to the concrete spillway will be required. Frequent cleaning of the downstream drainage channels will be required. Score: 3	No changes to the canal's efficiency expected from this alternative. Score: 3	2.7

Alternative	[redacted]	Cost	Constructability	Operations and Maintenance	Efficiency Improvements	Weighted Score
Alt. 10 -Detention/Infiltration Pond(s)	[redacted]	The costs associated with land acquisition and large earthwork volumes could be high. The detention pond(s) are less costly than discharge structures and channels. Partnering with the City of Fernley to allow the detention pond to serve as an infiltration pond could allow for cost sharing. Score: 4	Construction mostly consists of earthwork. No canal outage would be required. Score: 5	Periodic sediment removal from the detention pond and cleaning of the drainage pipelines will be required. Maintenance responsibilities could be shared with the City of Fernley. Score: 3	No changes to the canal's efficiency expected from this alternative. Score: 3	4.0

IX. Risk Reduction Alternative Plans

Combinations of the CAS risk reduction alternatives have been identified to develop a series of viable "risk reduction alternative plans" to address both internal erosion and hydrologic risks. Factors such as the individual alternative risk reduction, constructability, cost/benefit analysis and combinability of the alternatives to achieve the desired long-term risk reduction and canal conveyance capacity were considered when developing the risk reduction alternative plans. Improvements to operational control, consequence reduction and efficiency were also considered.

Each of the risk reduction alternative plans include replacement of the check structures to improve operational control and modifications to the Lahontan Reach to reduce sedimentation effects and increase the canal's capacity. In general these activities occur early in each of the identified risk reduction alternative plans.

To address the static and seismic internal erosion risks, two phases of linear canal embankment improvement have been identified. Phase I is about 6 miles long and includes areas where unacceptable and short-term tolerable risks at the 350 ft^3/s vegetated stage level were indicated in the 2014 risk analysis study [1]. Phase II is about 5.8 miles long and includes areas where unacceptable and short-term tolerable risks at the 600 ft^3/s vegetated stage level were indicated. Limits of Phase I and II are shown on Figure IX-1. The phased implementation will allow for incremental increases to the flow/stage level restriction. The linear canal embankment improvement measures include canal lining (three types), an embankment cutoff wall and embankment reconstruction. Alternative 3 (geomembrane liner with soil cover) and Alternative 5 (embankment reconstruction) did not achieve the desired risk reduction in the Fernley Reach and therefore were not included in the risk reduction alternative plans. They may be suitable for select, low consequence areas, in the Derby and Lahontan Reaches. The remaining linear canal embankment improvement alternatives (Alternative 1 - full geomembrane/concrete cover lining, Alternative 2 – partial geomembrane/concrete cover lining, and Alternative 4 – embankment cutoff wall) had similar degrees of internal erosion risk reduction.

The geomembrane/concrete cover alternatives will allow for an increase in the canal's conveyance capacity where the improvements are made. The lining will extend to the embankment crest and reduce the risk of internal erosion in the event of an elevated stage level from flooding. The concrete lining will also allow for an increase in capacity to convey flood inflows from Pour Point No. 8 to locations in the lower Fernley Reach and/or upper Lahontan Reach for discharge via a wasteway or passive spillway. If the canal lining is extended through the Lahontan Reach, flood inflows could be conveyed to Lahontan Reservoir.

Alternative 4 reduces the risk of internal erosion but does not appreciably increase the canal's conveyance capacity. Therefore, hydrologic protective features which limit or eliminate flood inflows were better suited for combination with Alternative 4. If the embankment cutoff wall is extended through the Fernley and Lahontan Reaches, then water in the canal could be retained to within a foot of the embankment crest, which does allow for some increase in capacity but this option will still need a discharge location upstream of Lahontan Reservoir to avoid overtopping.

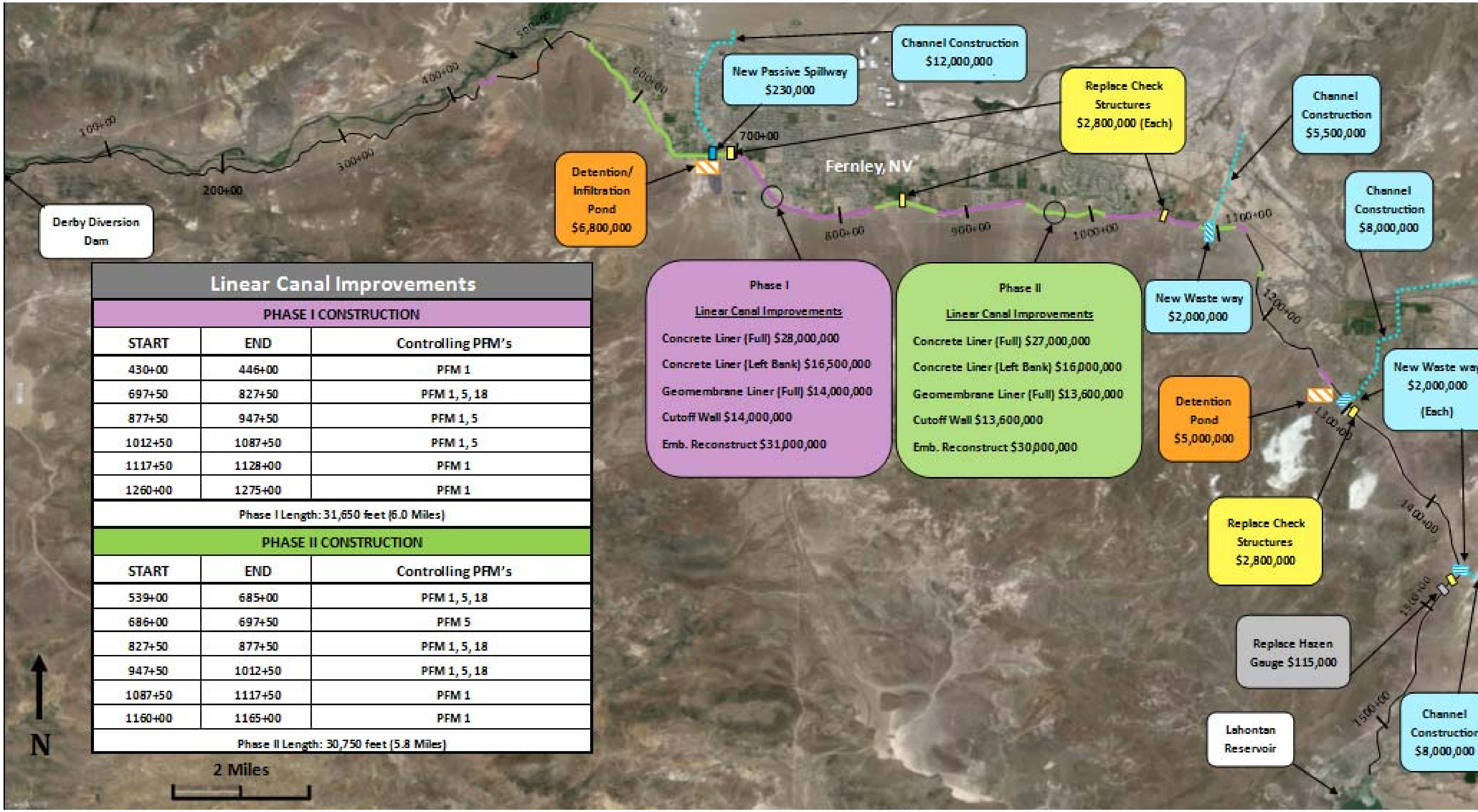
Five viable risk reduction alternatives plans were developed for consideration by decision makers. Table IX-1 summarizes the risk reduction alternative plans and likely phased implementation order. Each of the risk reduction alternative plans are discussed in the following sections.

A. Project Funding Approach

[redacted]

[redacted]

[redacted]



Linear Canal Improvements		
PHASE I CONSTRUCTION		
START	END	Controlling PFM's
430+00	446+00	PFM 1
697+50	827+50	PFM 1, 5, 18
877+50	947+50	PFM 1, 5
1012+50	1087+50	PFM 1, 5
1117+50	1128+00	PFM 1
1260+00	1275+00	PFM 1
Phase I Length: 31,650 feet (6.0 Miles)		
PHASE II CONSTRUCTION		
START	END	Controlling PFM's
539+00	685+00	PFM 1, 5, 18
686+00	697+50	PFM 5
827+50	877+50	PFM 1, 5, 18
947+50	1012+50	PFM 1, 5, 18
1087+50	1117+50	PFM 1
1160+00	1165+00	PFM 1
Phase II Length: 30,750 feet (5.8 Miles)		

Figure IX-1.—Risk Reduction Features Considered as part of the Risk Reduction Plans and Appraisal-level Costs

Table IX-1.—Risk Reduction Plan Summary

Risk Reduction Plan	Activity (Activity cost) (Implementation years)							Risk Reduction Plan Total Cost
1	Replace Check Structures and Hazen Gauge **($11,200,000) (year 2019)**	Phase I Canal Lining, Geomembrane/concrete cover (full prism) **($28,000,000) (years 2020 to 2049)**	Phase II Canal Lining, Geomembrane/concrete cover (full prism) **($27,000,000) (years 2050 to 2079)**	Hazen Wasteway and Drainage Channel **($10,000,000) (year 2088)**	-	-	-	**$76,200,000**
2	Replace Check Structures and Hazen Gauge **($11,200,000) (year 2019)**	Phase I Canal Lining, Geomembrane/concrete cover (full prism) **($28,000,000) (years 2020 to 2049)**	Phase II Canal Lining, Geomembrane/concrete cover (full prism) **($27,000,000) (years 2050 to 2079)**	Phase III Canal Lining, Geomembrane/concrete cover (full prism) **($23,300,000) (years 2080 to 2104)**	Phase IV Canal Lining, Geomembrane/concrete cover (full prism) **($23,300,000) (years 2105 to 2129)**	-	-	**$112,900,000**
3	Replace Check Structures and Hazen Gauge **($11,200,000) (year 2019)**	Phase I Embankment Cutoff Wall **($14,000,000) (years 2020 to 2034)**	Pour Point No. 8 Detention Pond **($6,800,000) (year 2041)**	Pour Point No. 16/19 Detention Pond **($5,000,000) (year 2046)**	Phase II Embankment Cutoff Wall **($14,000,000) (years 2047 to 2061)**	-	-	**$51,000,000**
4	Replace Check Structures and Hazen Gauge **($11,200,000) (year 2019)**	Phase I Embankment Cutoff Wall **($14,000,000) (years 2020 to 2034)**	Phase II Embankment Cutoff Wall **($14,000,000) (years 2035 to 2049)**	Phase III Embankment Cutoff Wall **($9,300,000) (years 2050 to 2059)**	Phase IV Embankment Cutoff Wall **($14,000,000) (years 2060 to 2074)**	Rock Ditch Wasteway and Drainage Channel **($10,000,000) (year 2084)**	-	**$72,500,000**
5	Replace Check Structures and Hazen Gauge **($11,200,000) (year 2019)**	Phase I Canal Lining, Geomembrane/concrete cover (partial prism) **($16,500,000) (years 2020 to 2036)**	Phase II Canal Lining, Geomembrane/concrete cover (partial prism) **($16,000,000) (years 2037 to 2052)**	Hazen Wasteway and Drainage Channel **($10,000,000) (year 2061)**	-	-	-	**$53,700,000**

Notes: [1]Costs for replacement of the Hazen Gauge are not included in the risk reduction plans. [2]Costs for adding geomembrane/soil cover lining in the Lahontan Reach will add about $23M to plan Nos. 1, 3 and 5.

B. Risk Reduction Alternative Plan 1

Risk reduction alternative plan No.1 includes replacement of the Hazen Gauge, a geomembrane/concrete cover liner (full prism) in the Phase I and II areas in the Fernley Reach, replacement of the remaining check structures, and construction of a new wasteway and drainage channel near the Mason Check. The Hazen Gauge includes a broad crested weir that causes the stage to back up through the lower portions of the Lahontan Reach and is contributing the sedimentation accumulation. The Hazen Gauge would be replaced with a flume section for flow measurement instead of the existing weir. The Hazen Gauge will be replaced ~ 2019 after issuance of the EIS record of decision to improve the conveyance capacity in the Lahontan Reach and minimize backwater effects. Costs for replacement of the Hazen Gauge have not been included risk reduction plans as the LBAO has indicated the gauge will be replaced separate from the XM project.

The geomembrane/concrete cover lining in the Fernley Reach provides the adequate level of static and seismic internal erosion risk reduction. Replacing the remaining check structures will lower the risk associated with ice jams and provide improved operational control of the canal stage level. The concrete lining will allow flood flows from Pour Point No. 8 to be conveyed downstream of the Fernley Reach for discharge in the Lahontan Reach. A wasteway at the existing Mason Check (Hazen Wasteway) will also allow for discharge of potentially large flood inflows at Pour Points No. 16 and 19.

Three special treatment areas will be addressed as part of risk reduction plan No. 1. They include 1,600 feet of unlined canal upstream of Tunnel 3, 500 feet of canal where there is known excessive seepage loss near the Steam Pad Seep (Station 1162+00), and about 1,500 feet of canal centered at the Red Barn Seep (Station 1267+00). The geomembrane/concrete cover alternative is best suited for the area upstream of Tunnel Three as this will allow for extension of existing concrete lining to the tunnel entrance. For the two special treatment areas in the Lahontan Reach (Steampad and Red Barn seep areas) is was assumed the geomembrane/soil cover lining system will be constructed in these areas in this risk reduction alternative plan.

The total cost for risk reduction alternative plan No. 1 is expected to be about $76,200,000. Assuming a project funding of about one million dollars per year, this plan will take about 70 years to fully implement. About 14 construction contracts will be required. Table IX-1 presents an estimation of the construction cost expenditures over time.

Risk reduction alternative plan No. 1 includes completion of the Phase I linear embankment improvements in year 2049. Until then the canal will continue to operate under the current revised restriction which has a peak operating flow ranging from 300 to 540 ft^3/s (vegetated/unvegetated). Once the Phase I linear embankment improvements are complete the restriction will be revised and allow for a peak operating flow ranging from 350 to 600 ft^3/s. Phase II of the linear embankment improvements will be completed in year 2074. At that point the restriction will be revised and allow a peak operating flow ranging from 600 to 900 ft^3/s, but not exceeding the project demand required to meet water supply reliability. Maintaining the canal free of aquatic vegetation will allow the maximum diversions to approach the reported peak operating flows for each of the stage-level restriction increments. Figure IX-2 shows how the maximum allowable diversions will be increased overtime as the stage-level restriction is incrementally raised.

C. Risk Reduction Alternative Plan 2

Risk reduction alternative plan No.2 includes replacement of the Hazen Gauge, a geomembrane/concrete cover liner (full prism) in the Phase I and II areas in the Fernley Reach, replacement of the remaining check structures, and to address hydrologic risks extension of the geomembrane/concrete cover liner through the Lahontan Reach to Lahontan Reservoir.

Early replacement of the Hazen Gauge is proposed for the reasons listed above in risk reduction plan No. 1. Costs for replacement of the Hazen Gauge have not been included in the risk reduction plans as the LBAO has indicated the gauge will be replaced separate from the XM project.

The geomembrane/concrete cover lining in the Fernley Reach provides the adequate level of static and seismic internal erosion risk reduction. Replacing the remaining check structures will lower the risk associated with ice jams and provide improved operational control of the canal stage level. Extending the geomembrane/concrete cover liner through the Lahontan Reach will increase the capacity in the Fernley and Lahontan Reaches to about 3,000 ft^3/s. The increase in capacity will allow the canal to convey flood inflows to Lahontan Reservoir and eliminate the need for discharge structures and downslope drainage channels. Lining the Lahontan Reach will improve efficiency by eliminating seepage losses. Lining of the Fernley and Lahontan Reaches will greatly reduce aquatic vegetation affects.

The three special treatment areas in the Derby and Lahontan reaches will be improved similarly to what was described in risk reduction plan No. 1.

The total cost for risk reduction alternative plan No. 2 is expected to be about \$112,900,000. Assuming a project funding of about one million dollars per year, this plan will take about 112 years to fully implement. About 22 construction contracts will be required. Table IX-1 presents an estimation of the construction cost expenditures over time.

The implementation plan for risk reduction alternative plan No. 2 results in a similar scheduled increase to the flow/stage level restriction over time as shown for risk reduction alternative plan No. 1 above. Figure IX-2 shows how the maximum allowable diversions will be increased overtime as the stage-level restriction is incrementally raised.

D. Risk Reduction Alternative Plan 3

Risk reduction alternative plan No.3 includes replacement of the Hazen Gauge, an embankment cutoff wall in the Phase I and II areas in the Fernley Reach, replacement of the remaining check structures, and to address hydrologic risks construction of two detention ponds (one at Pour Point 8 and one near Pour Points No. 16 and 19).

Early replacement of the Hazen Gauge is proposed for the reasons listed above in risk reduction plan No. 1. Costs for replacement of the Hazen Gauge have not been included in the risk reduction plans as the LBAO has indicated the gauge will be replaced separate from the XM project.

The embankment cutoff wall in the Fernley Reach provides the adequate level of static and seismic internal erosion risk reduction. Replacing the remaining check structures will lower the risk associated with ice jams and provide improved operational control of the canal stage level. The detention pond at Pour Point No. 8 will attenuate potentially large runoff inflows at this location. The Pour Point No. 8 detention pond has been developed with features that will allow gravity connection with the canal and promote “leakage” from the base of the pond to aid the City of Fernley’s aquifer recharge program. The diversion pipe will allow TCID to make surface water deliveries to the City of Fernley. The combined detention/infiltration function may allow for cost sharing between TCID and the City of Fernley. An additional detention pond is proposed near Pour Point No. 16 and 19 (largest pour points in the Lahontan Reach). A drainage channel will be required to route the runoff from the two nearby drainages to a centralized detention pond. The detention pond at Pour Point No. 16/19 will not have the infiltration features. Detention ponds are best combined with the sheet pile cutoff wall option, because the canal’s conveyance capacity will not be increased to convey flood inflows downstream. Detention ponds will limit the flood inflows at the largest pour points and minimize the potential for stage-level rise.

The three special treatment areas in the Derby and Lahontan reaches will be improved similarly to what was described in risk reduction plan No. 1.

The total cost for risk reduction alternative plan No. 3 is expected to be about \$51,000,000. Assuming a project funding of about one million dollars per year, this plan will take about 44 years to fully implement. About 9 construction contracts will be required. Table IX-1 presents an estimation of the construction cost expenditures over time.

Risk reduction alternative plan No. 3 includes completion of the Phase I linear embankment improvements in year 2034. Until then the canal will continue to operate under the current revised restriction which has a peak operating flow ranging from 300 to 540 ft^3/s (vegetated/unvegetated). This is about 15 years earlier than risk reduction plans No. 1 and 2 since the embankment cutoff wall is about half the cost of the geomembrane/concrete alternative. Once the Phase I linear embankment improvements are complete the restriction will be revised and allow for a peak operating flow ranging from 350 to 600 ft^3/s. To address hydrologic risks, risk reduction alternative plan No. 3 includes construction of the two detention ponds in years 2041 and 2046. Phase II of the linear embankment improvements will be completed in years 2047 to 2061. This is about 30 years earlier than risk reduction plans No. 1 and 2. At that point the restriction will be revised and allow a peak operating flow ranging from 600 to 900 ft^3/s. Maintaining the canal free of aquatic vegetation will allow the maximum diversions to approach the reported peak operating flows for each of the stage-level restriction increments. Figure IX-2 shows how the maximum allowable diversions will be increased overtime as the stage-level restriction is incrementally raised.

E. Risk Reduction Alternative Plan 4

Risk reduction alternative plan No.4 includes replacement of the Hazen Gauge, an embankment cutoff wall in the Phase I and II areas in the Fernley Reach, replacement of the remaining check structures, and to address hydrologic risks continuation of the embankment cutoff wall through the Lahontan Reach, and construction of a new wasteway and drainage channel near the Bango Check (Rock Ditch Wasteway).

Early replacement of the Hazen Gauge is proposed for the reasons listed above in risk reduction plan No. 1. Costs for replacement of the Hazen Gauge have not been included risk reduction plans as the LBAO has indicated the gauge will be replaced separate from the XM project.

The embankment cutoff wall in the Fernley Reach provides the adequate level of static and seismic internal erosion risk reduction. Replacing the remaining check structures will lower the risk associated with ice jams and provide improved operational control of the canal stage level. Extending the embankment cutoff wall through the Lahontan Reach will protect the upper portions of the embankment during runoff inflows and stage-level rise. To prevent overtopping, a new wasteway and drainage channel will be constructed just upstream of the Bango Check.

The three special treatment areas in the Derby and Lahontan reaches will be improved similarly to what was described in risk reduction plan No. 1.

The total cost for risk reduction alternative plan No. 4 is expected to be about $72,500,000. Assuming a project funding of about one million dollars per year, this plan will take about 67 years to fully implement. About 13 construction contracts will be required. Table IX-1 presents an estimation of the construction cost expenditures over time.

The implementation plan for risk reduction alternative plan No. 4 results in a similar scheduled increase to the flow/stage level restriction over time as shown for risk reduction alternative plan No. 3 above except that Phase II of the linear embankment improvement would be completed in year 2050 allow a peak operating flow ranging from 600 to 900 ft^3/s to be achieved earlier. Figure IX-2 shows how the maximum allowable diversions will be increased overtime as the stage-level restriction is incrementally raised.

F. Risk Reduction Alternative Plan 5

Risk reduction alternative plan No.1 includes replacement of the Hazen Gauge, a geomembrane/concrete cover liner (half liner) in the Phase I and II areas in the Fernley Reach, replacement of the remaining check structures, and construction of a new wasteway and drainage channel near the Mason Check.

Early replacement of the Hazen Gauge is proposed for the reasons listed above in risk reduction plan No. 1. Costs for replacement of the Hazen Gauge have not been included risk reduction plans as the LBAO has indicated the gauge will be replaced separately from the XM project.

The geomembrane/concrete cover lining in the Fernley Reach provides the adequate level of static and seismic internal erosion risk reduction. Replacing the remaining check structures will lower the risk associated with ice jams and provide improved operational control of the canal stage level. The partial concrete lining will allow improved ability to convey flood flows from Pour Point No. 8 for discharge in the Lahontan Reach. A wasteway at the existing Mason Check (Hazen Wasteway) will also allow for discharge of potentially large flood inflows at Pour Points No. 16 and 19.

The three special treatment areas in the Derby and Lahontan reaches will be improved similarly to what was described in risk reduction plan No. 1.

The total cost for risk reduction alternative plan No. 1 is expected to be about $53,700,000. Assuming a project funding of about one million dollars per year, this plan will take about 44 years to fully implement. About 9 construction contracts will be required. Table IX-1 presents an estimation of the construction cost expenditures over time.

The implementation plan for risk reduction alternative plan No. 5 results in a similar scheduled increase to the flow/stage level restriction over time as shown for risk reduction alternative plan No. 4 above. Figure IX-2 shows how the maximum allowable diversions will be increased overtime as the stage-level restriction is incrementally raised.

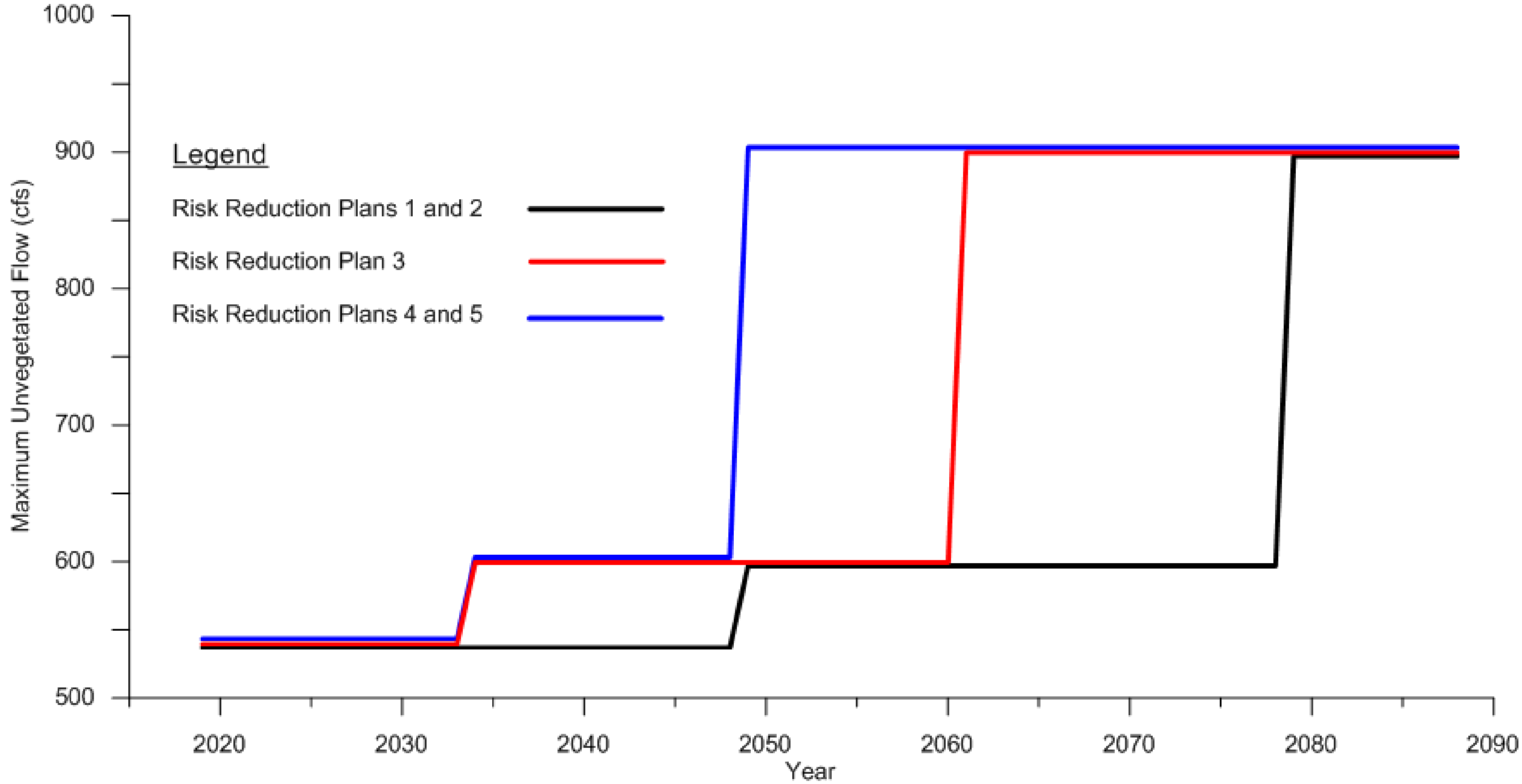


Figure IX-2.—Peak Operating Flow over Time for Risk Reduction Plans

X. Hydrologic Analysis

A hydrologic analysis was completed by the LBAO to aid in the evaluation and selection of the recommended risk reduction plan(s). The analysis utilized the Truckee-Carson River RiverWare Planning Model (Planning Model) to evaluate how the various CAS alternatives would impact deliveries within the Newlands Project. The analysis also evaluated potential impacts to reservoir storage, other deliveries, Truckee River flows and Pyramid Lake levels to inform the EIS. The input parameters, modeling approach and results are summarized in a memorandum titled *RiverWare Modeling for the Truckee Canal CAS*, dated 4/15/2017 [2] and is included in Appendix B. A summary of the analysis is provided below.

A. Modeling Approach

LBAO used an existing RiverWare model developed by Reclamation with current data from the *Truckee Basin Study* [16], updated to represent the estimated current demand, to estimate how each of the risk reduction alternative plans compared to the *modeled historical water supply reliability* in the Newlands Project which includes the Truckee River Operating Agreement (TROA) operating criteria to complete the hydrologic analysis. Nine scenarios were developed to represent a range of peak operating flows from 0 ft^3/s to 900 ft^3/s (maximum canal diversions modeled during the 1997 Final Adjusted Operating Criteria and Procedures (OCAP) Study). The scenarios represent a range of peak operating flows that might be in place as canal safety improvements are completed and the flow/stage level restriction is relaxed over time. The nine model scenarios are listed in Table X-1.

Historical hydrology from 1901 through 2000 was used as the hydrologic input to evaluate water deliveries to the Newlands Project water users if the historical hydrology was to be repeated. The period from 1901 through 2000 represents a range of hydrological conditions from extended drought to repetitive wet years. Average model conditions during dry (1960s) and wet (1920s) periods were analyzed to evaluate their effects as the risk reduction plans would be implemented overtime.

Results of the analysis will be used to develop the modeled historical water supply reliability scenario, identify which risk reduction plan(s) achieve the modeled historical water supply reliability, and to provide information for the completion of the economic and financial feasibility analyses (see Section XI).

Table X-1.—Hydrologic Analysis Scenarios

Scenarios for Water Supply Reliability	Risk Reduction Specifics	Max Allowable Flow (cfs)	Seepage Loss Reduction
		Yrs. 1901 - 2000	
Scenario 1	Truckee Canal Decommissioning	0	No
Scenario 2	Long-term no action	140	No
Scenario 3	Restriction imposed after the 2008 breach	350	No
Scenario 4	Revised Stage-level Restriction	450	No
Scenario 5	Revised Stage-level Restriction	540	No
Scenario 6	Full Lining of 5.7 Miles of Canal	600	Yes (25%)
Scenario 7	Full Lining of 11.4 Miles of Canal	900	Yes (50%)
Scenario 8	Non-lining Improvement Measures of 5.7 Miles of Canal	600	No
Scenario 9	Non-lining Improvement Measures of 11.4 Miles of Canal (modeled historical water supply reliability scenario)	900	No

B. Modeled Historical Water Supply Reliability

For the purpose of the CAS, the modeled historical water supply reliability scenario was developed using the Planning Model and demand data from the Truckee Basin Study and recent OCAP annual determinations. The modeled historical water supply reliability scenario is defined as the approximate level of service current Newlands Project water users would have experienced from 1901 through 2000 if the canal was operated with the current operating parameters (e.g., OCAP, TROA, etc) and under canal condition assumptions of the 1997 OCAP modeling (i.e., maximum Truckee Canal capacity of 900 cfs). The modeled historical water supply reliability scenario indicates that water users in the Newlands Project (both Truckee and Carson Divisions) would have historically received at least 95 percent of their modeled water demands in 91 years out of the 100 evaluated (i.e., 9 water short years out of a 100).

C. Seepage Loss Reduction from the CAS Alternatives

CAS Alternatives 1 and 3 include lining the full canal prism with a geomembrane liner. Where installed seepage losses presumably would be zero. Alternative 2 (partial geomembrane/concrete lining is also expected to reduce seepage losses from current levels, however; since the right half of the prism will be unlined seepage losses would continue. For the purposes of this CAS it was assumed Alternative 2 would not provide appreciable seepage loss reduction. The remaining CAS alternatives do not provide a positive seepage cutoff and are not expected to appreciably reduce seepage losses. Scenarios 6 and 7 represent the expected peak operating flows and estimated seepage loss reduction for CAS Phases I and II, respectively. The Fernley and Lahontan Reaches are mostly unlined (about 23 miles in length). It was assumed if 5.7 miles of canal was fully lined (Phase I), seepage losses would be reduced by about 25 percent. If both Phase I and II were constructed (11.4 miles) it was assumed the seepage loss would be reduced by about 50 percent. A comparison of the results from scenarios 6 and 7 with results from 8 and 9, will indicate the potential efficiency improvements and whether the water savings justify the higher cost for canal lining.

D. RiverWare Modeling Results

Table X-2 summarizes results of the analysis for the Carson Division under estimated current demand condition. The number of water short years ranged from 48 (canal decommissioning) to 9 (improve canal peak operating flow to at least 540 ft^3/s, scenarios 5 through 9). Results of the analysis indicate scenarios 1 through 4 do not meet the modeled historical reliability in the Carson Division (see Figure X-1). Results for Scenarios 5 through 9 (i.e. peak operating flows greater than 540 ft^3/s) suggest if the Phase I linear canal embankment improvements are implemented, the Truckee Canal will provide sufficient water to meet the modeled historical water supply reliability (9 water short years) in the Truckee and Carson divisions of the Newlands Project. This assumes an aquatic vegetation control program is in place to maintain a year round peak operating flow of at least 540 ft^3/s. Above a peak operating flow of about 540 ft^3/s the average annual Truckee Canal diversions and total Carson Division shortages are similar (see Figures X-2 and X-3). Section XI summarizes a cost/benefit analysis as it relates to the amount of canal safety improvements to achieve the modeled historical reliability while being financially feasible.

The water operations modeling results, based on the historical hydrological inflow data used in the model, indicate alternatives which had higher peak operating canal flows resulted in less water (volume) being diverted from the Truckee River and more water available to Pyramid Lake. By increasing the safe capacity of the Truckee Canal, more water can be diverted early in the season when higher flows in the Truckee River are available. This will allow Lahontan Reservoir to reach storage targets earlier. Later as runoff diminishes, less diversions to the Truckee Canal will be required. This in general, will allow higher flows in the Truckee River below the Derby Diversion Dam, during summer and fall months. For example: should the canal be improved to safely convey 600 ft^3/s for 20 days during the winter/spring runoff, the diversions can be reduced to 150 ft^3/s for 40 days later in the year, and provide the same amount of water if the canal was operated continuously at 300 ft^3/s. Additionally with higher Lahontan Reservoir storage levels, the allowable diversions to the Truckee Canal will be less later in the year in accordance with OCAP. The considerations above only apply to years when Lahontan Reservoir is low early in the season and larger winter/spring Truckee River flows are available. While improving the Truckee Canal to have a peak operating flow greater than 540 ft^3/s may not appreciably affect the water supply reliability in the Carson Division, it may allow for reduced diversions later in the season when lower Truckee River flows below Derby Diversion Dam may affect fish habitat.

A comparison of scenarios 6 and 8 (600 ft^3/s lining versus non-lining alternatives) indicate average annual seepage reductions from 2 to 7 ft^3/s (about 1,500 to 5,000 acre-feet per year) (see Figure X-4). A comparison of scenarios 7 and 9 (900 ft^3/s lining versus non-lining alternatives) indicate average annual seepage reductions from 5 to 15 ft^3/s (about 3,500 to 10,000 acre-feet per year) (see Figure X-5). These volumes represent about 1 to 2 percent of the total water that enters both Pyramid Lake and Lahontan Reservoir. It is acknowledged that canal seepage losses can be a significant percentage of the Truckee River flows below the Derby Diversion Dam during the summer and fall months. A cost/benefit evaluation of the canal lining alternatives is further discussed in Section XI.

Table X-2.—Hydrologic Analysis Results for the Carson Division

Scenarios for Water Supply Reliability	Risk Reduction Specifics	Max Allowable Flow (cfs)	Seepage Loss Reduction	Results		
		Yrs. 1900 - 2000		Number of Shortage Years	Average Percent Demand Met (%)	Average Shortage over 9 driest years (AF)
Scenario 1	Truckee Canal Decom.	0	No	48	NA	154,788
Scenario 2	Long-term no action	140	No	31	88.4	124,031
Scenario 3	Restriction imposed after the 2008 breach	350	No	14	95.0	86,100
Scenario 4	Revised Stage-level Restriction	450	No	11	95.5	81,023
Scenario 5	Revised Stage-level Restriction	540	No	9	95.7	79,216
Scenario 6	Full Lining of 5.7 Miles of Canal (~25%)	600	Yes (25%)	9	95.8	77,788
Scenario 7	Full Lining of 11.4 Miles of Canal (~50%)	900	Yes (50%)	9	96.0	72,868
Scenario 8	Non-lining Improvement of 5.7 Miles of Canal	600	No	9	96.0	74,522
Scenario 9	Non-lining Improvement of 11.4 Miles of Canal	900	No	9	96.4	65,690

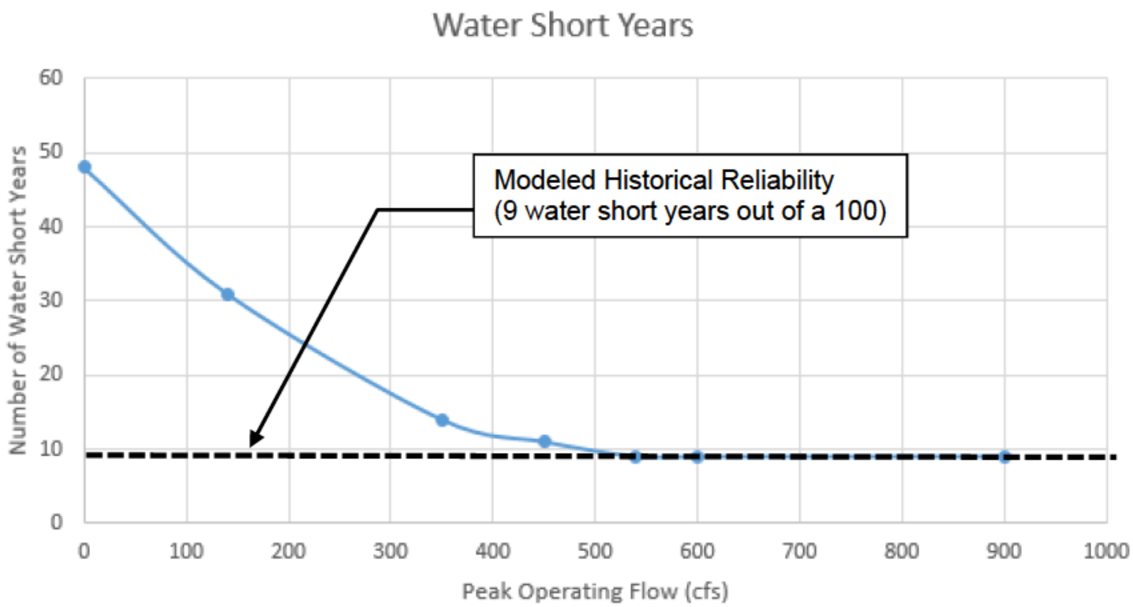


Figure X-1: Water Short Years versus Truckee Canal Peak Operating Flow

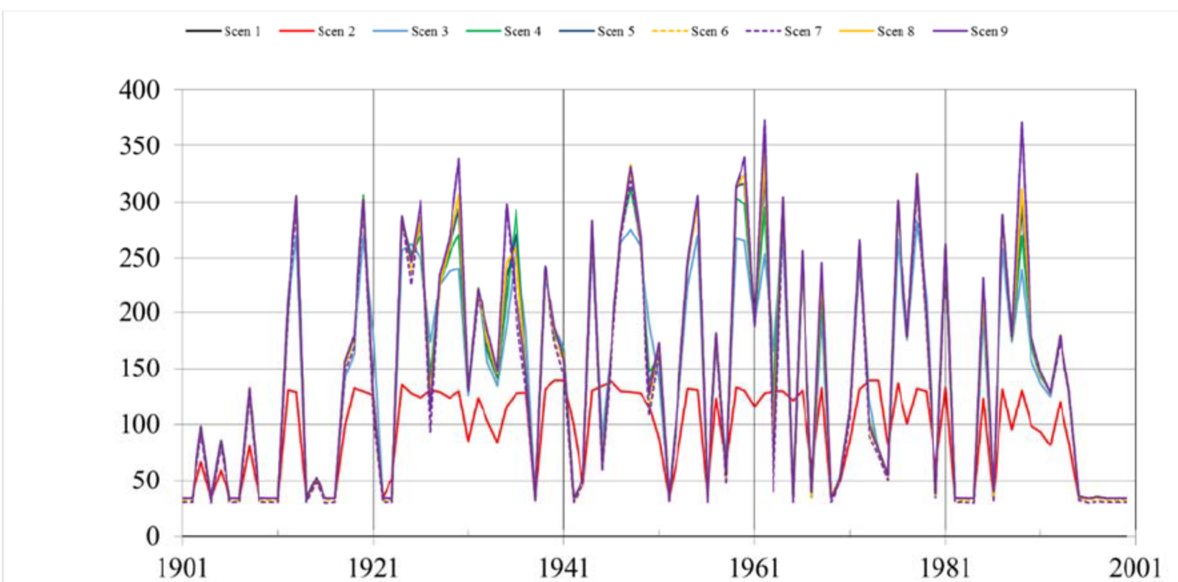


Figure X-2: Annual Average Truckee Canal Diversions at the Wadsworth Gauge. (note average annual diversions are typically less than 350 ft^3/s due to OCAP and TROA restrictions)

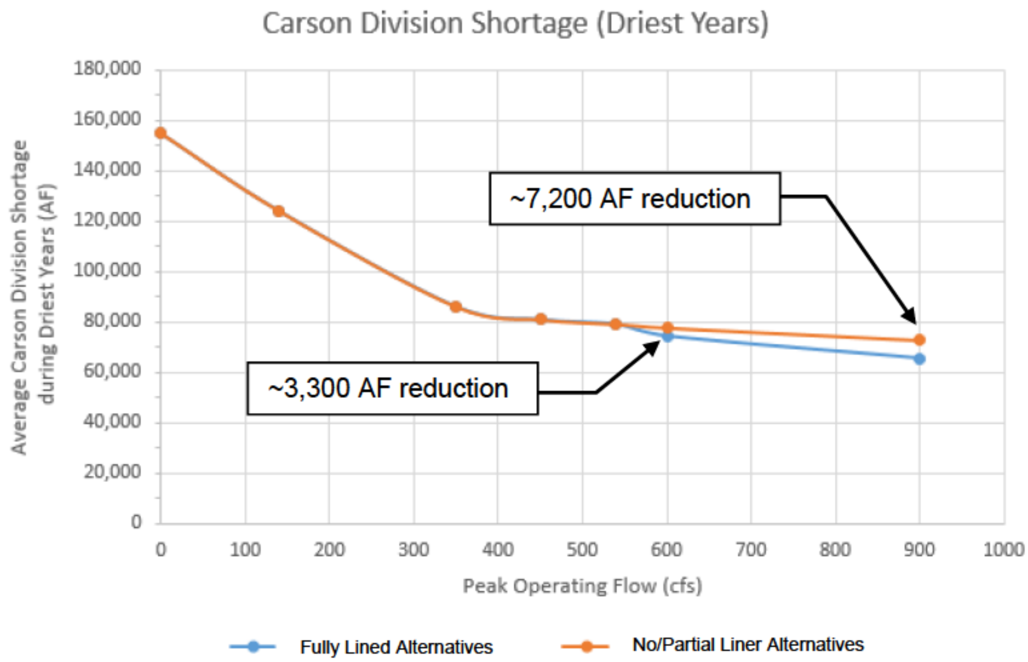


Figure X-3: Average Carson Division Shortages during the 9 Driest Years in the Simulation

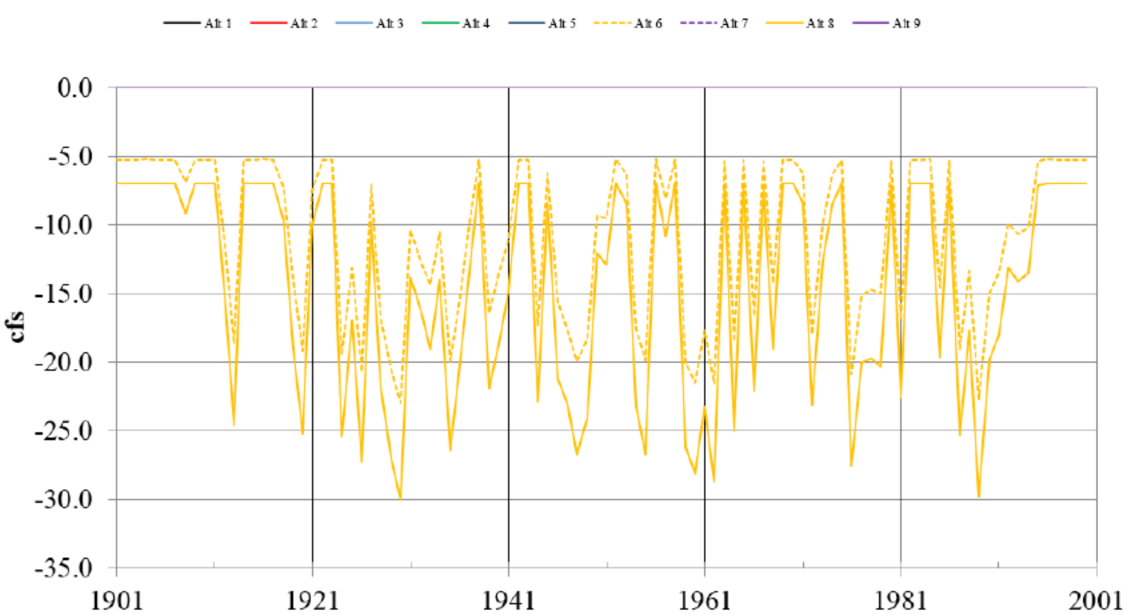


Figure X-4: Annual Seepage losses in the Truckee Canal for Scenarios 6 and 8 (600 ft^3/s).

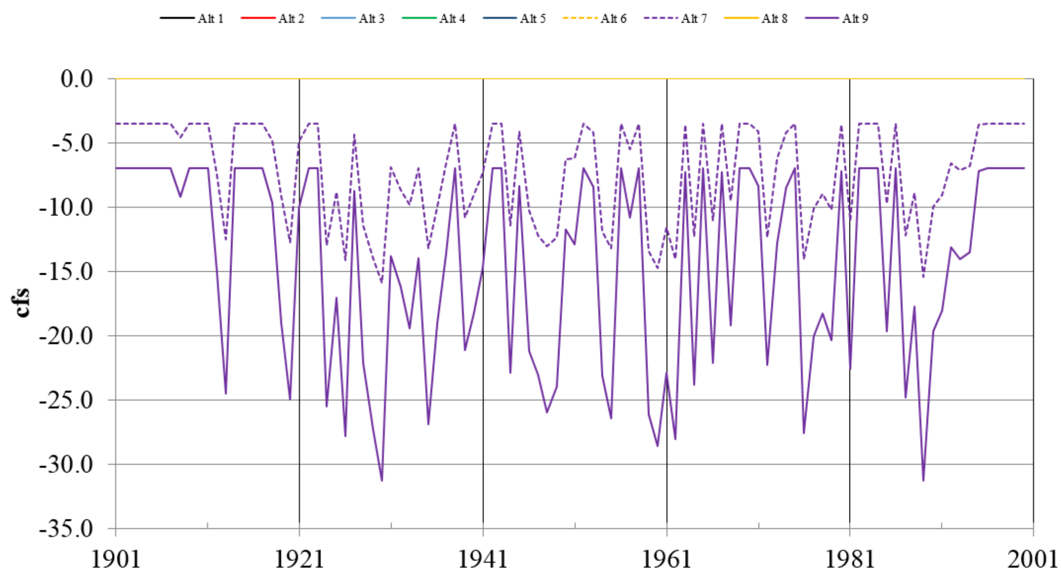


Figure X-5: Annual Seepage losses in the Truckee Canal for Scenarios 7 and 9 (900 ft^3/s).

XI. Economic and Financial Feasibility Analysis

A. Approach

In this Section, an appraisal level economic benefit analysis and a corresponding appraisal level financial feasibility analysis are conducted. Taken together, the goal of these studies is to support the decision-making process by providing a consistent basis for comparing the economic efficiency and financial feasibility of the proposed risk reduction plans.

The economic benefit analysis (Benefit Analysis) consists of three linked parts. First, the economic benefits attributable to the Newlands Project are estimated on a per unit basis. Second, the benefits and costs of the CAS risk reduction alternatives are evaluated, and only those plans whose benefits exceed the costs are carried forward to step three. Third, a cost effective analysis is conducted to determine the preferred least cost risk reduction plan that meets the long-term risk reduction goals.

The financial feasibility analysis assesses the ability of TCID to cover existing costs and any additional costs incurred by implementation of a risk reduction plan. TCID’s ability-to-pay (ATP) is developed by estimating the potential revenue streams paid to TCID; consisting of assessments on water users, District hydropower revenues, and other District revenues.

The economic analysis conducted herein and all Federal water-related projects initiated prior to June 2015 are subject to the *Economic and Environmental Principles and Guidelines for Water and Related Land Resources Implementation Studies* (P&Gs). In accordance with the P&Gs, a *Plan Formulation and Evaluation Rate* (Planning Rate), established by the U.S. Department of the Treasury, is utilized and the period of analysis (POA) is 50 years. The Planning Rate for fiscal year 2017 is 2.875 percent and is applied when calculating the present value of forecasted benefits. This study is conducted under conditions representative of the circumstances faced by Newlands Project water users in 2015 (study year) and all monetary values have been indexed to "real" 2015 dollars[2] using indices applicable to the values being studied (e.g., Consumer Price Index).

B. Economic Analysis

1. National Economic Benefit Valuation

A *national economic benefit valuation* (benefit valuation) estimates the gross benefits generated by the Newlands Project from a national perspective for each purpose on a per acre-foot (AF) basis. For this TM, the Newlands Project generates benefits in the following five different benefit categories: irrigation water supply, municipal and industrial (M&I) water supply, wetlands and wildlife (W&W) water supply, hydropower generation, and recreation.

The per-AF benefit values are determined by deriving the marginal value of water based on end use. The derivation method employed in estimating benefit values varies based on the nature of the benefit, project scope, and availability of information. Although the methods for determining the values vary, the marginal project benefits are all estimated using a *with vs. without* framework. This conceptual framework allows for the isolation and estimation of the marginal benefit values that are generated by the Newlands Project under an alternative condition by comparing them with the benefits generated in the baseline condition. The *with-conditions* are defined by those conditions that would likely occur in the planning area over the established 50-year POA with a risk reduction plan in place. The *without-conditions* are defined as the most likely future condition to occur without a risk reduction plan in place.

[2] A "real dollar" value is an adjusted value that attempts to account for the impacts of inflation—the change in the purchasing power of a dollar. A real dollar value is used in this analysis to allow for better comparability of resource values across time. The estimated purchasing power of a dollar in year 2015 is used as the base value of a dollar (reference year).

a. Irrigation Water Supply Benefits

Project water supply is defined as irrigation water in PEC P05[3] (p. 3, note 6) when the water is used to "…irrigate land primarily for the production of commercial agricultural crops or livestock, and domestic and other uses that are incidental thereto." As further described in PEC P05, irrigation use does not include uses such as "…watering golf courses; lawns and ornamental shrubbery used in residential and commercial landscaping, household gardens, parks and other recreational facilities; pasture for animals raised for personal purposes or for nonagricultural commercial purposes; cemeteries; and similar uses…" In addition, irrigation use does not include "…commercial agricultural uses that do not require irrigation, such as fish farms and livestock production in confined feeding or brooding operations…" (e.g., dairy farm operations).

(1) Approach to Valuing Irrigation Water Supply Benefits

Agricultural benefits resulting from a project, as outlined in the P&Gs, can be attributable to flood damage reduction, drainage, irrigation, erosion control, and sediment reduction. In this TM, only the Newlands Project irrigation benefits resulting from the application of Newlands Project water to lands within TCID for irrigation use are examined. Specifically, benefits that may accrue from changes in agricultural production due to the availability and dependability of Newlands Project water. For example, the availability of project water might allow farm operators to increase their agricultural production and farm income by changing their cropping pattern to a more lucrative crop.

In this TM, a *farm budget analysis* (FBA) approach is used to approximate the marginal benefits attributable to project water used for irrigation. An FBA attempts to measure project benefits by estimating the difference in *net farm income* (NFI) resulting from a change in project water supply under future with-conditions as compared to future without-conditions. In other words, the NFI is calculated for farms representative of those assumed to be cultivated under the future with- and without-conditions and the resulting difference between the estimated NFIs is attributed to the value of project irrigation water supply. For clarification, NFI is the difference between the estimated gross farm income and the estimated farm expenses, including returns to the farm family's labor and management.

As detailed in the proceeding sections, three farm budgets are constructed to represent, to the extent possible, commercial agricultural operations in TCID under the various conditions. The with-condition is represented by the major cropping patterns currently found in TCID (alfalfa, corn silage, and irrigated pasture). The without-project condition is represented by the most likely cropping

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[3] Bureau of Reclamation. *Reclamation Manual,* "Directive and Standards," PEC P05, *Water-Related Contracts–General Principles and Requirements.* July 24, 2013

pattern assumed to exist on land where project deliveries are no longer made (dryland pasture).

Agricultural Acreage and Representative Farm Types

The estimated irrigated agricultural acreage, water use definitions (e.g., irrigation vs. M&I), representative cropping patterns, and average Project water demands used in this benefit analysis are displayed in Table XI-1 and Table XI-2 XI-2.

The total quantity of acres in TCID in the 2015 study year that are defined as agricultural lands and that hold Project water rights match the acreage estimates established in the *RiverWare Modeling for the Truckee Canal CAS* technical memo (Lahontan Basin Area Office, 2017), hereafter referred to as the Operations Model. For this appraisal level analysis, the Operations Model, and therefore the benefit valuation, hold the acreage estimates and Newlands Project water demands equal to the study-year estimates over the 50-year POA.

For the Carson Division, agricultural acreage estimates were computed as the average acreage over a ten-year timespan from 2004-2014 and are similar to the demands used in the Truckee Basin Study (Lahontan Basin Area Office, 2015). The acres in agricultural production in the Truckee Division acreages are assumed to be equal to those determined in the past OCAP annual determinations.

The average annual cropping patterns representative of the commercial agriculture acreage identified in TCID were approximated using data from the Agriculture Cropland Data Layer (USDA-NASS, 2017), the TCID Water Conservation Plan (2010), and the 2012 Census of Agriculture (USDA-NASS, 2014). In the Carson Division, on average, the majority—roughly 90 percent—of all irrigated agricultural land is planted to alfalfa (70 percent) and rotation crops (20 percent) such as small grains for forage or corn silage. The final 10 percent of irrigated agricultural land is dedicated to irrigated pasture. In the Truckee Division, 80 percent of all agricultural land is planted to alfalfa, and the other 20 percent is made up of alfalfa rotation crops, on average.

The P&Gs recommend that enough farm types be analyzed to represent the farm operations of the study area. As it is often not practical to complete farm budgets for all crops grown in an irrigation district, certain crops that are grown only on a small percentage of total district acres can be represented by a more extensively grown crop in the same general category of crops (e.g., forage, grain, orchard, vegetables, etc.).

For this TM, three separate farm budgets (two for the Carson Division and one for the Truckee Division) were prepared to represent the farming operations in TCID under the with-conditions. Overall, in TCID, the 20 percent of irrigated agricultural acres comprised of corn silage, small forage grains, and other crops were grouped together and represented by corn silage acreage that is planted in rotation with alfalfa in a single representative farm. This grouping was based on the similarity in gross revenues of the other crops with corn silage and the relatively small amount of total acreage planted in the other crops.

For the Carson Division, the first representative farm consists of 200 total acres of agricultural crops—156 acres of alfalfa and 44 acres corn silage—representing 90 percent of the irrigated agricultural acreage that is planted in alfalfa and rotation crops. The acreage split among the crops reflects the weighted NFI per acre of the representative crops when applied to the estimated average annual acreage in alfalfa and rotation crops.[4] The second representative farm consists of 135 acres of irrigated pasture, representing the 10 percent of Carson Division acreage that is planted to irrigated pasture.

The representative farm for the Truckee Division consists of 200 total acres of agricultural crops—160 acres of alfalfa and 40 acres corn silage—representing 100 percent of the irrigated agricultural acreage. The acreage split among the crops reflects the weighted NFI per acre of the representative crops when applied to the estimated average annual irrigated agricultural acreage in the Truckee Division.[5]

A single dryland pasture budget was developed to represent the without-condition, as it is assumed that when farmers are faced with a shortage of Newlands Project water they will consolidate water as to not deficit irrigate crops by reducing the amount of irrigated acres and converting the now non-irrigated acres to dryland pasture. This is a simplifying assumption that approximates the average representative farmer's approach to dealing with water shortages and reduces the number of assumptions that would be required to model a deficit irrigation approach or a change in cropping patterns. However, at a feasibility level analysis, approaches such as deficit irrigation or the changing of cropping patterns may warrant further investigation.

Based on the definition of irrigation water supply defined at the beginning of this Section, agricultural acreage being served with Project water was classified as either Irrigation use or M&I use. The classification of acreage was determined primarily by data obtained from USDA-NASS publications (USDA-NASS, 2014; USDA-NASS, 2013). Through this process, it was established that all of the irrigated agricultural acreage in TCID meets the definition of irrigation use. Note, it is possible that the classification of water use on identified agricultural acres may change at a feasibility level analysis.

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[4] $(156/200)*(.9) \approx 70\%$, $(44/200)*(.9) \approx 20\%$
[5] $(160/200)*(1) \approx 80\%$, $(40/200)*(1) \approx 20\%$

The weighted average water demand included in column six of Table XI-1 and Table XI-2 are derived from Table 3 of the Operations Model TM (Lahontan Basin Area Office, 2017). The weighted demand is calculated as the product of the quantity of acres in each water duty category multiplied by the AF water duty established in the Newlands Project.

Table XI-1.—Carson Division Representative Crops: With-Condition

Crop (1)	Total Irrigated Agricultural Acres (2)	Irrigated Agricultural Acres as a Percent of Total (3)	Irrigated Agricultural Acres Classified as M&I Use (4)	Irrigated Agricultural Acres Classified as Irrigation Use (5)	Weighted Avg. Water Demand (AF/acre) (6)
Alfalfa	45,839	70%	0	32,087	3.56 AF
Corn Silage		20%	0	9,168	
Irrigated Pasture		10%	0	4,584	

Table XI-2.—Truckee Division Representative Crops: With-Condition

Crop (1)	Total Irrigated Agricultural Acres (2)	Irrigated Agricultural Acres as a Percent of Total (3)	Irrigated Agricultural Acres Classified as M&I Use (4)	Irrigated Agricultural Acres Classified as Irrigation Use (5)	Weighted Avg. Water Demand (AF/acre) (6)
Alfalfa	2,100	80%	0	1,680	4.50 AF
Corn Silage		20%	0	420	

(2) Farm Budget Analysis

This Section states the assumptions made, budget inputs, and the steps used in developing the farm budgets and evaluating the NFI of the modeled farm operations. The FBA-generated output reports for the benefit analysis, which detail specific assumptions and calculations, are included in Appendix C.

Crop Yields

Table XI-3 displays the average yields per acre utilized in this TM in column (4). County level crop yields were sourced from USDA-NASS's Quick Stats Tool (Quick Stats, 2017). Crop yield data was vetted in discussions with the University of Nevada Cooperative Extension (UNCE) and comparisons with recent UNCE and other relevant crop enterprise budgets. When possible, representative average annual yields utilized in this report are represented by the five year average yields from 2011 to 2015.

Prices Received

Table XI-3 reports the *prices received* that are used as inputs to the FBA in column (5). In accordance with the P&Gs, statewide average prices in Nevada over the previous three years were used when available. When Nevada prices were not available National level prices were used. Government program payments are not included in a Reclamation benefits evaluation. For this evaluation, farmers are assumed to be price takers and prices received are assumed to be unaffected by changes in project water availability.

Gross Farm Revenue

Table XI-3 reports the summary findings of gross farm revenue by crop. The gross farm revenue of an agricultural operation is simply calculated as the product of the market price for a commodity and quantity sold. All commodities are produced on-farm and assumed to be sold after harvest.

Table XI-3.—Average Annual Gross Farm Revenue per Acre by Crop (2015$)

Condition (1)	Crop (2)	Unit (3)	Annual Average Yield/Acre (4)	Price Received ($/Unit) (5)	Avg. Annual Gross Farm Revenue ($/acre) (6)
With-Condition	Alfalfa[1]	Ton	6	$203.67	$1222.02
	Corn Silage	Ton	27	$39.23	$1059.21
	Irrigated Pasture[2]	Ton	2.5	$104.33	$260.83
Without-Condition	Dryland Pasture	Ton	0.65	$52.17	$33.91

[1/] Alfalfa yields are the same in establishment and production years.
[2/] Irrigated pasture yields reflect an annual water entitlement of 1.5 AF/acre

Farm Expenses

The majority of fixed and operational farm expenses—both quantities and prices—were sourced from crop enterprise budgets and other publications developed by cooperative extensions for crops similar to the representative crops modeled in this analysis. The crop enterprise budgets and publications utilized were sourced from the following cooperative extensions: UNCE, University of Idaho Extension, and University of California Davis Extension. Further, other relevant sources were used in refining the farm budgets developed for this analysis such as the *Costs of Owning and operating Farm Machinery in the Pacific Northwest: 2011*, published by Pacific Northwest Extension. A copy of each of the enterprise budgets and publications used are provided in Appendix C. The source for each expense input can be found in the FBA reports that are provided in Appendix C.

Return to Farm Family

Expenses related to labor and management provided by the farm operator and farm family (return to farm family) are represented by the opportunity cost to the Nation of providing the services. The returns to the farm family are noncash allowances for the operator's factors of production and are deducted from the net farm income when determining NFI.

Return to Management

An allowance of 6 percent of variable costs is made to capture management expenses. The return to management can be explained as an opportunity cost to the Nation, representing the farm operator's ability to earn income by applying their management skills in another operation.

Return to Labor

The farm operator's labor is valued at the 2015 wage rate ($21.73) for supervisory farm labor for the crop type in the study area as reported in the 2015 *State Occupational Employment and Wages Estimates Report* (Bureau of Labor Statistics, 2017). Labor performed by the farm operator's family is valued at the 2015 wage rate ($13.98) for farmworkers as reported in (Bureau of Labor Statistics, 2017). The return to labor is calculated by adding the farm operator's wages and the farm family labor wages.

Farm Budget Analysis Results: Net Farm Income

As detailed in the preceding sections, four farm budgets were constructed to represent the with- and without-conditions. The estimated average annual NFI per acre resulting from the FBA are reported in column (4) of Table XI-4.

Table XI-4.—Farm Budget Analysis Results: Net Farm Income

Condition (1)	TCID Division (2)	Representative Farm (3)	Net Farm Income ($/Acre) (4)
With-Condition	Carson Division	Alfalfa/Corn Silage	$224.89
		Irrigated Pasture	-$272.56
With-Condition	Truckee Division	Alfalfa/Corn Silage	$231.74
Without-Condition	Carson & Truckee Divisions	Dryland Pasture	-$284.85

(3) Irrigation Benefit Valuation Results

The per-acre gross benefits of Newlands Project water attributable to irrigation is approximated as the difference in NFI per-acre between the with-condition representative farm and the without-condition dryland pasture farm. At the division level, the average benefit per irrigated acre is calculated by computing the weighted average of all irrigated acres in a division. The weighted average is calculated by completing the following two steps: 1) take the product of the estimated per-acre benefits and the total cropped acres of each representative farm in the division, and 2) divide the product by the total irrigated acreage within the division.

Table XI-5 displays a summary of the estimated gross irrigation use benefits by representative farm. The first two columns of the table identify the Newlands Project Divisions and the representative farm being evaluated. The third column reports the estimated NFI per acre for each representative farm under the with-conditions. The fourth column presents the calculated NFI per acre generated by the representative farm under the assumed without-conditions (dryland pasture). The gross benefits per acre are then reported as the difference between the with-conditions NFI and the without-conditions NFI by representative farm in column five. The weighted average NFI per acre is calculated and displayed in column seven.

Note, the NFI of representative farms can be negative under both of the project conditions and still generate positive irrigation use benefits. Specifically, this is because gross benefits are calculated as the difference between the NFI under the with- and without-conditions. Therefore, an agricultural operation can still provide positive gross benefits to the Nation if the with-condition benefits are less negative than the without-condition gross benefits.

Table XI-5.—Gross Irrigation Benefits by Representative Farm (2015$)

TCID Division (1)	Representative Farm (2)	With-Condition NFI (3)	Without-Condition NFI (4)	Gross Benefits: (with NFI – without NFI) (5)	Irrigation Use Acres (6)	Weighted Avg. Gross Benefit ($/acre)[1] (7)
Truckee Division	Alfalfa/Corn Silage	$231.74	-$284.85	$516.59	2,100	$516.59
Carson Division	Alfalfa/Corn Silage	$224.89	-$284.85	$509.74	41,255	$460.00
	Irrigated Pasture	-$272.56	-$284.85	$12.29	4,584	

[1]/ Weighted District Average Gross Benefit by Farm ($/acre) = $(Gross\ Benefits\ by\ Farm\ \times Acres\ per\ Crop) \div (Total\ Division\ Acres)$.

The net irrigation use benefits for Project water are reported in Table XI-6. The net irrigation use benefits per acre in column five are calculated as the difference between the average gross benefits per acre in column three and the average annual per-acre District costs associated with the Newlands Project ($79.46) in column four. The annual per-acre District costs are calculated as the average operations, maintenance, and administration costs incurred over the period from 2011–2015, minus the average annual income from the District General Assessment discussed in Section XI.C.1.c. Note that the total average annual per-acre District costs are only those costs attributable to operating the Newlands Project for water use purposes and do not included hydropower costs or revenues. The net weighted irrigation use benefits per AF reported in column eight can be calculated as the quotient of the net irrigation use benefits per acre in column five and the weighted average water demand per acre in column six.

Table XI-6.—Net Irrigation Benefits by Representative Farm (2015$)

TCID Division (1)	Irrigated Crop (2)	Weighted Avg. Gross Benefit ($/acre) (3)	Annual District Project Costs ($/acre) (4)	Weighted Avg. Net Benefit ($/acre) (5)	Weighted Avg. Water Demand [1] (AF/acre) (6)	Weighted Avg. Gross Benefit ($/AF) (7)	Weighted Avg. Net Benefit ($/AF) (8)
Truckee Division	Alfalfa/Corn Silage	$516.59	$79.46	$437.13	4.50 AF	$114.80	$97.14
Carson Division	Alfalfa/Corn Silage Irrigated Pasture	$460.00	$79.46	$380.53	3.56 AF	$129.21	$106.89

[1]/ See Table XI-1 and Table XI-2 and accompanying narrative for derivation.

b. Municipal & Industrial Water Supply Benefits

M&I water supply is defined in Reclamation Manual PEC P05 (p. 3, note 6) as "The use of contract water for municipal, industrial, and miscellaneous other purposes not falling under the definition of 'irrigation use' or within another category of water use under an applicable Federal authority." Based on the definition of irrigation also established in PEC P05, M&I water supply is further defined as tbe use of contract water tbat is not used to irrigate land primarily used for the production of commercial agricultural crops or livestock. Thus, M&I water supply includes uses such as watering golf courses; lawns and ornamental shrubbery used in residential and commercial landscaping, household gardens, parks and other recreational facilities; pasture for animals raised for personal purposes or for nonagricultural commercial purposes; cemeteries; and similar uses. In addition, commercial agricultural uses that do not require irrigation, such as fish farms and livestock production in confined feeding or brooding operations (e.g., dairy farm operations) are also classified as M&I use.

(1) Approach to Valuing M&I Benefits

The economic benefit value of water is measured in terms of *willingness to pay* (WTP). WTP can be defined as the dollar amount that an entity is willing to give up or pay to use Project water. In short, the WTP for Project water is based on the quantity of water available and demanded for water at a certain price level.

For this analysis, WTP is estimated using a *revealed preference approach* (RPA), where actual observed market behavior (i.e., water rights transactions) is analyzed to derive the value of Project M&I water supply. However, due to the scope of this analysis and the lack of water rights transactions available for long-term Project water supply scenarios similar to the without-conditions, the RPA used here is limited in its capacity to estimate the "true" WTP at the range of quantities (i.e., shortages) estimated under the proposed risk reduction plans and the long-term no action alternative. Therefore, under alternatives that have a limited impact to the quantity of Project water available, the following WTP calculation are within a feasible range, but as the change in quantity increases the accuracy of the WTP estimated in this analysis decreases. The WTP calculations provided in this TM do, however, provide a lower bound estimate of the economic value of Project water used for M&I purposes. Note that, in general, as the quantity of water available decreases the price will increase.

A more comprehensive discussion of the methods for valuing M&I water can be found in Reclamation's technical memorandum *Evaluating Economic and Financial Feasibility of Municipal and Industrial Water Projects* (Piper, 2009).

(2) The Economic Value of M&I Water Supply

Although Project surface water rights are fully appropriated, there is an active local water rights market in the study area where permanent rights are bought and sold. Using the RPA, an examination of the available water rights transactions allows for the observation of water users WTP for the permanent use of Project water. Further, purchasers of water rights in this analysis are assumed to have included future annual O&M costs in their purchase decision and, therefore, the estimated WTP reflects the net present benefit value of surface water. In other words, the costs associated with the operations and maintenance of the Newlands Project and the delivery of Project water have already been taken into account. In summary, the analysis of water right transactions allows for the estimation of the net present benefit value of an AF of Project M&I water supply into perpetuity. The water rights market data used in this analysis was obtained from the Churchill County Recorder's Office (Recorder Document Index, 2017) and included nine transactions spanning the years from 2011–2014. The water rights transactions examined included transactions of Project water that were readily identified as being used for M&I water supply and excluded transactions that were clearly used for purposes other than M&I. However, transactions that did not clearly specify the end use of the water were not removed from the analysis. At a feasibility level of analysis further vetting of the available transaction data should be completed (e.g., confirmation of transaction amounts, determination of end use of water). All values were indexed to 2015 dollars using the *implicit price deflator* (Bureau of Economic Analysis, 2017) The water right transaction data utilized in this report is provided in Appendix C.

The results of the WTP estimation are provided in Table XI-7. The WTP for the right to use Newlands Project water for M&I purposes was estimated using the above-specified data in three steps. First, the net present value per AF was determined by taking the arithmetic mean of the permanent water right transactions ($1,345). Second, the *equivalent annual net benefit* (EANB)—annual WTP—value of an AF of M&I water supply into perpetuity was calculated as the product of the Planning Rate and the calculated net present value per AF ($1,345 * 0.02875 = $38.67). Third, the net present value for the use of an AF of Project water for M&I purposes over the POA is calculated as the present value of the EANB over the 50-year POA ($1,019). In summary, the net present value of M&I benefits over the POA is equal to the sum of the discounted EANB over the POA.

Note, the EANB value is constant over time, however, due to the period for which benefits accrue, the present value of holding a water right over the POA differs from that of holding a water right into perpetuity.

Table XI-7.—Economic Value of M&I Water Supply (2015$/AF)

Period of Analysis	Equivalent Annual Net Benefit ($/AF)	Net Present Value of M&I Water Supply[1] ($/AF)
Perpetual	$38.67	$1,345
50-Years	$38.67	$1,019

[1]/ The present value for the 50-year POA was used in this TM.

c. Wildlife & Wetlands Water Supply Benefits

There are three primary locations within the study area where Project water is used for the generation of W&W benefits: Stillwater National Wildlife Refuge (Stillwater NWR), Fallon Reservation Wetlands (FRW), and Carson Lake and Pasture (CLP).

Stillwater NWR is managed by the U.S. Fish & Wildlife Service (USFWS) to benefit breeding and migrating waterfowl, shorebirds, and other water birds and wintering waterfowl and is classified as a Site of International Importance by the Western Hemispheric Shorebird Reserve Network. Stillwater NWR occupies approximately 124 square miles (about 77,000 acres) in the Lahontan Valley at the northeastern most edge of the Newlands Project and has a total Project water entitlement of 21,644.8 AF (Lahontan Basin Area Office, 2017).

The State of Nevada manages CLP primarily for wildlife, habitat, and public use, though a portion of its lands are permitted for grazing. CLP includes 10,800 acres of wetlands with 6,710.5 AF of Newlands Project water rights (Lahontan Basin Area Office, 2017).

The FRW has Newlands Project water rights equaling 1399.9 AF (Lahontan Basin Area Office, 2017).

(1) Approach to Valuing W&W Benefits

As discussed in more detail in Section (1) *Approach to Valuing M&I Benefits*, the economic benefit value of W&W water supply is measured in terms of WTP. For this analysis, WTP for W&W water supply is estimated using an RPA, where actual observed market behavior is analyzed to derive the benefit value of Project W&W water supply. Based on the reasoning provided in Section (1) The WTP calculations provided in this TM provided a lower bound estimate of the economic value of Project water used for W&W purposes.

(2) The Economic Value of W&W Water Supply

Project water rights have been purchased by various entities that have converted water rights from irrigation water use to the purpose of W&W. The water rights have been purchased and converted at a rate of 2.99 AF per acre of water rights purchased. The water rights market data used in this analysis was obtained from the Churchill County Recorder's Office (Recorder Document Index, 2017) and included nine transactions spanning the years from 2011–2015. The water rights transactions examined included transactions of Newlands Project water that were readily identified as being used for W&W water supply and excluded transactions that were clearly used for purposes other than W&W. At a feasibility level of analysis, further vetting of the available transaction data should be completed (e.g., confirmation of transaction amounts, determination of end use of water). All values were indexed to 2015 dollars using the *implicit price deflator* (Bureau of Economic Analysis, 2017). The water right transaction data utilized in this report is provided in Appendix C.

The results of the WTP estimation are provided in Table XI-8. The WTP for the right to use Project water for W&W purposes was estimated using the above-specified data in three steps. First, the net present value per AF was determined by taking the arithmetic mean of the permanent water right transactions ($2,782). Second, the EANB—annual WTP—value of an AF of W&W water supply into perpetuity was calculated as the product of the Planning Rate and the calculated net present value per AF ($2,782 * 0.02875 = $79.98). Third, the net present value for the use of an AF of Project water supply for W&W purposes over the POA is calculated as the present value of the EANB over the 50-year POA ($2,108). In summary, the net present value of W&W benefits over the POA is equal to the sum of the discounted EANB over the POA.

Note, the EANB value is constant over time, however, due to the period for which benefits accrue, the present value of holding a water right over the POA differs from that of holding a water right into perpetuity.

Table XI-8.—Economic Value of W&W Water Supply

Period of Analysis	Equivalent Annual Net Benefit ($/AF)	Net Present Value of W&W Water Supply[1] ($/AF)
Perpetual	$79.98	$2,782
50-Years	$79.98	$2,108

[1]/ The present value for the 50-year POA was used in this TM.

d. Lahontan Reservoir Recreation Benefits

This section estimates the value of Project water as it contributes to the recreation opportunities and therefore the economic benefits provided by Lahontan Reservoir. While the study area offers a large array of water-based recreation opportunities that rely on Project water to some degree, due to its scope, this TM focuses on recreation opportunities at Lahontan Reservoir.

Lahontan Reservoir, which is managed by Nevada State Parks (NSP), is the largest body of water in Lahontan valley, with a surface area of 10,000 acres at the top of conservation pool, and provides numerous water-related recreation opportunities including boating, water skiing, fishing, camping, picnicking, hunting, wildlife viewing, and swimming. According to recreation visitation data provided by NSP for the years 2009-2016, Lahontan Reservoir received approximately 23 percent of the total recreation visits—an average of roughly 221,000 annual visits—in Nevada's Northern Region (which includes Rye Patch Reservoir, Walker Lake, Fort Churchill, and Berlin-Ichthyosaur State Park). When water levels are at an adequate level, Lahontan Reservoir is one of the top five heaviest-used camping and boating parks in the State system.

(1) Approach to Valuing Recreation Benefits

A three-step process is used to estimate the economic value of Project water as it contributes to the recreation opportunities provided at Lahontan Reservoir based on the water level elevation at the Reservoir. First, a recreation model using regression analysis techniques was developed to estimate the relationship between recreation visitation and water levels at Lahontan Reservoir. For example, the change in the number of recreation visits if the water level were to be decreased by one foot. Second, the economic value per recreation visit is determined using a *benefit transfer approach.* Third, the recreation benefit related to a one-foot change in water level elevation at Lahontan Reservoir is calculated as the product of the estimated visitation relationship from step one and the economic value per recreation visit established in step two.

(2) The Economic Value of Recreation Benefits

Recreation Model

Recreational use of Lahontan Reservoir is strongly tied to water levels. Over the 2009–2016 time-span annual visitation to the Reservoir was shown to exceed 300,000 people during average and above-average water years, but decline substantially to levels below 80,000 visitors in years when water levels are low. According to the Nevada Division of State Parks, a storage volume of 150,000 AF (water elevation 4,144.9 feet) is preferred during July, the most important month for recreation at the Reservoir. Further, a volume of 120,000 AF (water elevation 4139.5 feet) is the minimum water volume for reasonable use of boat ramps at the reservoir, and below 90,000 AF (water elevation 4133.3 feet), virtually no power boat use is possible.

The basic recreation model developed to estimate the relationship between recreation visitation and water levels consists of only one explanatory variable.

Mathematically,

$$visits_t = ß_0 + ß_1 \mathrm{wl}_t + \varepsilon_t$$

Where:

$visits_t$ = Historical annual recreation visits to Lahontan Reservoir

wl_t = Annual average Lahontan Reservoir elevation

The historical annual recreation visits to Lahontan Reservoir were provided by NSP for the years spanning 2009–2016. Historical annual average elevation levels for the Reservoir were obtained from the U.S. Geological Survey's National Water Information System (USGS, 2017). Figure XI-1 provides a graph of this data by year with the annual recreation visitation on the primary y-axis and Reservoir elevation on the secondary y-axis.

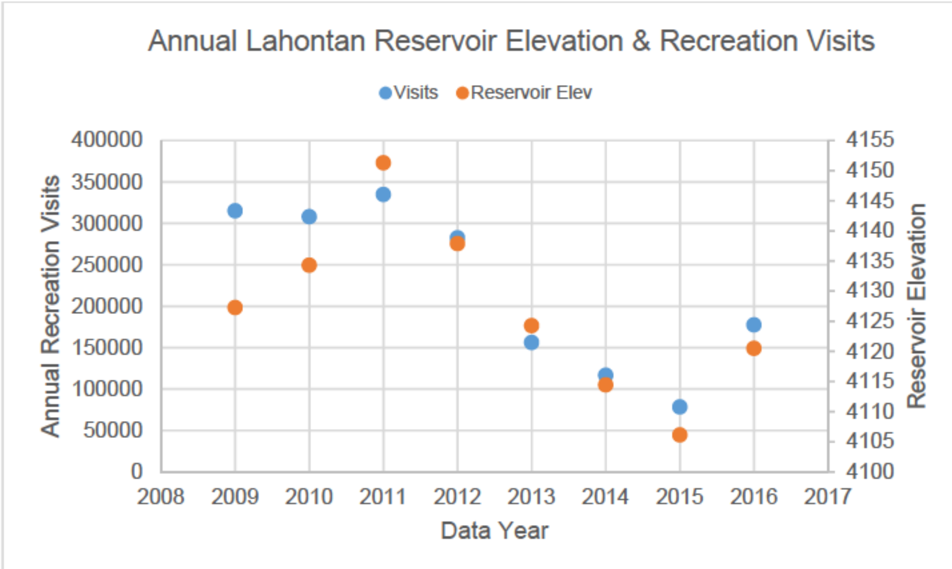

Figure XI-1.—Recreation Model Data

A regression model was run for the recreation model and the data described above using the Microsoft Excel 2013 Analysis ToolPak. The modeling results are reported in Table XI-9. The coefficient estimated for *wl* indicates the effect of annual average Reservoir elevation on annual average recreation visits to the Reservoir. The t-statistic provides an indication of the extent to which an estimated coefficient is statistically different from zero. The t-statistic for *wl* indicates that the estimated coefficient for *wl* is highly significant at less than the 1 percent level.

The overall fit and statistical significance of the recreation model is evaluated using the adjusted R-squared. The adjusted R-squared indicates the percentage of variation in the dependent variable (recreation visitation) that is explained by the model. The resulting adjusted R-squared for this model is very good at 0.888 which means that 88.8 percent of the variation in visitation is explained by the elevation of Lahontan Reservoir.

Although the overall modeling results were good and provide a good first approximation of the visitation–reservoir elevation relationship, the recreation model is rudimentary in its design and potentially suffers from several econometric issues such as omitted variable bias and non-stationarity. At a feasibility level, these issues should be address and a higher resolution model (e.g., monthly data by activity) should be undertaken.

Using the modeling results presented in Table XI-9, the estimated relationship between recreation visitation and changes in Reservoir elevation, holding all other variables constant, can be approximated as the coefficient for *wl*. The *wl* coefficient can be interpreted as, holding all else constant, the average impact to annual visitation at the Reservoir given a 1-foot change in the elevation of Lahontan Reservoir. Therefore, based on the modeling results, a 1-foot decrease in the annual average water level elevation at Lahontan Reservoir corresponds to an estimated average decrease of approximately 6,293 annual recreation visits to the Reservoir, given the relevant sample range.

Table XI-9.—Recreation Model Results

Explanatory Variable	Coefficient	*t*-statistic	P-value
wl	6293.184	4.74*	0.003
Constant	-25750935.357	-4.70	0.003
Adjusted R-squared	0.888	* Significant at the 1% level	

Recreation Visitation Values

A summary of the benefit values by recreation activity employed in this study can be found in Table XI-10. Per recreation visitor benefit values for the primary water-related recreation opportunities provided by Lahontan Reservoir were estimated using the *Recreation Use Values Database for North America* (Rosenberger, 2011). This database contains 352 economic valuation studies that estimate the use value of recreation activities, measured in net WTP (consumer surplus) for recreational access in the U.S. and Canada from 1958 to 2006. In an effort to increase the accuracy of benefit values and obtain values that are more site specific, the recreation benefit values estimated in this analysis employed only a subset of the aforementioned database—only those economic valuation studies conducted after 1980 in the Western Census Region. Further, the median rather than the mean benefit values were taken to avoid potential skewness and distortion from outliers. All reported values have been indexed to 2015 dollars using the *Consumer Price Index* (CPI) (BLS, 2017).

The recreation benefit value of Project water to the Nation is calculated as the estimated net WTP to participate in water-related recreation opportunities provided by Lahontan Reservoir (estimated above) and the net WTP for the most likely alternative recreation activity under the without-condition. Therefore, from a national perspective, the value of recreation benefits can be calculated as the incremental difference between the consumer surplus received between recreating at Lahontan Reservoir (with-condition/preferred recreation) and an alternative recreation activity (without-condition). In this TM, it is assumed that when recreators substitute their preferred site or activity with a less desirable one, or vice versa, approximately 25 percent of their consumer surplus is impacted and 75 percent of their consumer surplus is retained. This is likely a lower bound estimate of the impact to recreation benefits. Recreation substitution is case specific and always somewhat speculative. Further research is beyond the scope of this analysis, however, in a feasibility level analysis a more robust approach to determine the loss in consumer surplus per visit should be attempted.

Table XI-10.—Lahontan Reservoir: Economic Value per Recreation Visitor (2015$)

Primary Recreation Activities	Median Recreation Value Per Visitor
Freshwater Fishing	$60.01
Motor Boating	$32.10
Non-Motorized Boating	$60.46
Picnicking	$22.61
Swimming	$26.88
General Recreation	$22.29
Wildlife Viewing	$49.60
Waterfowl Hunting	$41.85
Hiking	$35.28
Camping	$15.57
Average With-Condition Benefit Value	$36.67
Average Without-Condition Benefit Value	$27.50
Recreation Benefit Value (with-condition) – (without-condition)	**$9.17**

This study is bounded to an analysis of the average annual impacts of the proposed alternatives and does not include the potential additional impacts of drought conditions and their residual effects (e.g., the loss of fish stocks).

The value of Project water, as it relates to benefits generated by the recreation opportunities provide at Lahontan Reservoir at different water levels, is provided in Table XI-11. The benefits are calculated as the product of the change in visitation associated with a one foot change in Reservoir elevation and the estimated average recreation benefit value per visit to the Reservoir.

Table XI-11.—Value of Recreation Visitation at Lahontan Reservoir Elevation (2015$)

Recreation Site	Change in Visitation Associated with a One Foot Change in Reservoir Elevation	Avg. Benefit per Visitor ($/visit)	Economic Value of a One Foot Change in Reservoir Elevation
Lahontan Reservoir	6,293	$9.17	$57,707

e. Hydropower Benefits

Hydropower production is not one of the primary purposes of the Newlands Project. Hydropower generation is accomplished in an incidental manner, meaning that the production of hydropower does not influence the timing and volume of hydropower-generating flows within the Newlands Project. However, the power generated by TCID is sold through purchase agreements, and the sale of this power represents a significant portion of TCID income. For this reason, the generation of power by Project operations is an important component of alternatives evaluated for this study.

Hydropower is generated by releases from Lahontan Reservoir and from flows below Lahontan Reservoir routed through the V Canal within the Carson Division. Two hydropower plants capitalize on releases from Lahontan Reservoir, and are referred to as the "Old" and "New" power plants. The Old Lahontan Plant is a 1.9 megawatt (MW) facility built in 1911, while the New Lahontan Plant is a 4.8 MW facility built in 1982 and owned by TCID (FWS, 1996). These two plants are operated conjunctively, with the intended result of maximizing power output across the range of Lahontan Reservoir elevations (heads) and the flow rates. Hydropower on the V Canal is generated at the 26-Foot Drop Power Plant, which was built by TCID in 1955 (FWS, 1996). Generation at the 26-Foot Drop Power Plant relies on flows routed to TCID customers who receive water from the V Canal. Typically, 70 percent of all releases from Lahontan Dam are routed through the V Canal (Reclamation, 2013).

(1) Approach to Valuing Hydropower Benefits

Hydropower net benefits are calculated as hydropower gross benefits less hydropower generation expenses. The gross benefits generated by TCID hydropower plants are assumed to be equal to the cost required to procure an equivalent amount of energy on the wholesale energy market. The costs to procure an equivalent amount of energy should be based on market data for the region in which TCID resides and provides power to. Expenses to be counted against gross benefits to derive net benefits are those costs incurred to TCID for hydropower generation operations.

(2) Hydropower Gross Benefits

Hydropower gross benefits are the product per unit of generated hydropower multiplied by the market price of electricity per unit for that power. Required inputs to calculate TCID hydropower gross benefits under a given scenario include: (1) The energy generated by TCID powerplants under the given scenario; and (2) The market price of the electricity produced by TCID powerplants. The following sections detail these primary inputs.

TCID Hydropower Generation

The Operations Model includes forecasted hydropower generation at TCID powerplants over a 100-year period for each of the nine Truckee Canal maximum flow scenarios (Reclamation, 2017). Two relevant metrics for this hydropower

benefits analysis are output by the Operations Model: (1) the monthly sum of energy generated in gigawatt hours (GWh) at the 26 Foot Drop powerplant; and (2) The monthly sum of energy generated in GWh at Lahontan reservoir—or the combined monthly energy generated at the Old Lahontan and New Lahontan powerplants.
The energy generation input used in this benefits analysis is the average energy generation over the 100-year period for each month. Electricity prices can vary considerably month to month, so generation values and electricity prices are aggregated at monthly time-steps to allow for maximum accuracy when calculating hydropower benefits. Electricity prices are reported in dollars per megawatt hour (MWh) and, therefore, energy generation values are converted from GWh to MWh.

Figure XI-2 below displays average monthly hydropower generation at TCID powerplants under Scenario 2 (long-term no action) as forecasted by the Operations Model. Note that, on average, the combination of Old Lahontan and New Lahontan powerplants makes up about 91 percent of total TCID hydropower generation.

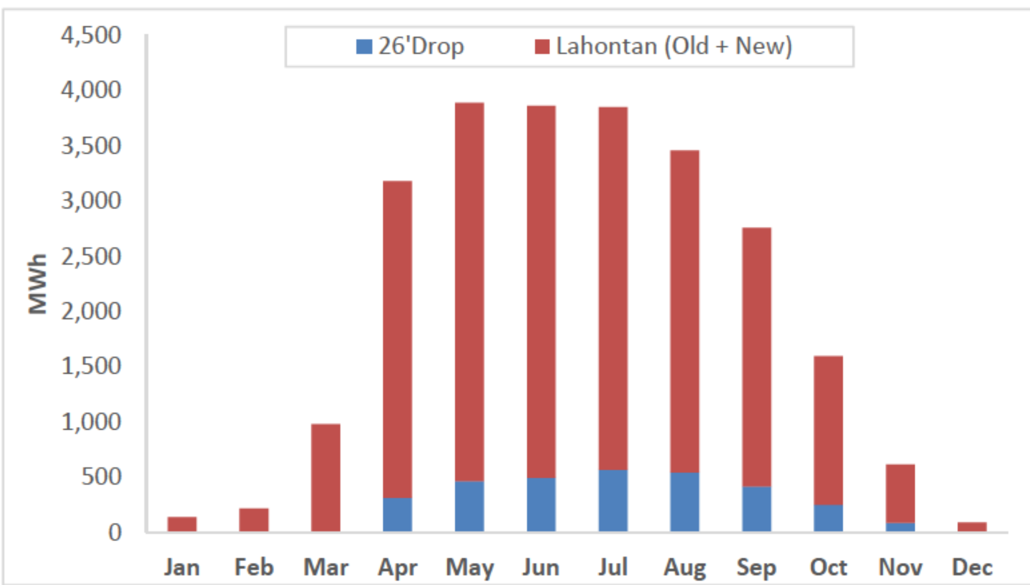

Figure XI-2.—TCID average hydropower generation by month under Scenario 2 over 100-year modeling period

Electricity Prices

For a hydropower benefits study, the electricity price to be used should be the wholesale rate for the region of interest in current-year dollars. If there is evidence that electricity prices are escalating faster than the rate of inflation, an escalation factor might be incorporated to forecast future years included in the time horizon of the benefit study. As demonstrated later in this section, there is no evidence that electricity prices in the region of interest are escalating faster than the rate of inflation, and therefore no escalation factor is incorporated. The metric developed for electricity prices in this study is the 2015 monthly expected price for wholesale electricity in the Northern California region.

The US Energy Information Administration (EIA) reports wholesale electricity prices for eight major hubs throughout the US (EIA, 2017). This market data is collected by the Intercontinental Exchange (ICE) and includes daily volumes, high and low prices, and weighted-average prices. TCID and utilities served by TCID hydropower fall within the Northern California region—hub name NP-15. Daily wholesale electricity historical data is available for NP-15 beginning in 2009.

The primary electricity price input data used in this study is the daily weighted-average prices reported for NP-15. The daily weighted-average price is developed using Equation (1) below (EIA, 2017).

$$I = \frac{\sum(P * V)}{T} \tag{1}$$

Where:

I	=	Volumetric Weighted Average Index Price
P	=	Price or premium of individual transaction
V	=	Volume of individual transaction
$\sum(P * V)$	=	Sum of each transaction's price multiplied by its volume
T	=	Total volume of all qualifying transactions

The monthly average of daily weighted-average electricity prices were calculated for every month from April 2009 (the first month of available data) through December 2016. As illustrated in Figure XI-3, there is no evidence that NP-15 regional wholesale electricity prices are escalating faster than the rate of inflation. Though this seven-year period is rather limited, the slightly negative slope of the linear trendline regressed over the dataset (-1×10^{-5}) is evidence that no adjustment for electricity price escalation should be made for the purpose of this study.

Figure XI-3.—Monthly wholesale electricity prices for NP-15, Apr 2009–Dec 2016

To estimate the 2015 monthly expected price for wholesale electricity in the Northern California region a price trend estimation using linear regression analysis is employed. A price trend analysis takes into account the overall movement and historical direction of a price due to various price influences, such as changing factors of production, market conditions, and inflation. The price trend estimation was calculated for each month utilizing the monthly averages of seven years (2009–2016) of historical NP-15 daily weighted-average electricity prices. Figure XI-4 displays 2015 monthly expected price for wholesale electricity in the Northern California region.

Figure XI-4.—2015 monthly expected price for NP-15 wholesale electricity

Calculation of Hydropower Gross Benefits

Annual gross hydropower benefits under a given scenario is equal to the sum of monthly gross hydropower benefits. Monthly gross hydropower benefits are the product of 2015 monthly expected price for wholesale electricity in the Northern California region multiplied by the average total monthly hydropower generated at TCID powerplants as modeled by the Operations Model. Hydropower gross benefits are estimated in 2015 dollars.

Figure XI-5 displays TCID monthly hydropower gross benefits under Scenario 2 (long-term no action). Note that nearly 90 percent of hydropower gross benefits are generated between the months of April through September (inclusive), which coincides with maximum runoff flows and the TCID irrigation season.

[redacted]

[redacted]

[redacted]

(4) Calculation of Hydropower Net Benefits

Annual net hydropower benefits are calculated as the difference between annual gross hydropower benefits and annual hydropower generation operations expenses. Table XI-12 below displays the annual gross benefits, annual operating expenses, annual net benefits, and the present value of net benefits under each modeled scenario. The present value of annual net benefits is calculated using the Planning Rate over the 50-year POA.

[redacted]

[redacted]

f. National Economic Net Benefit Valuation Summary

Table XI-13 provides a summary of the per unit net benefit values calculated in this Section. These values will be used in the following benefit-cost analysis and cost-effective analysis. A single unit value was not established for hydropower as the benefit values used in this analysis vary by month, see Section XI.B.1.e for details.

Table XI-13.—Summary of Unit Benefit Values (2015$)

Benefit Category	Unit	Estimated Annual Net Benefit Value	Estimated Present Value (50-year POA)
Irrigation Use Water Supply (Carson Division)	AF	$106.89	$2,817
Irrigation Use Water Supply (Truckee Division)	AF	$97.14	$2,560
M&I Use Water Supply	AF	$38.67	$1,019
Wildlife & Wetlands Water Supply	AF	$79.98	$2,108
Lahontan Reservoir Recreation	Reservoir Elevation (1 foot)	$57,707	$1,520,683

2. *Benefit-Cost Analysis*

In this Section, the proposed CAS erosion risk reduction alternatives are evaluated based on their construction costs and benefits generated through the increased availability of Project water. This benefit-cost analysis is undertaken in order to determine if the benefits generated by the alternatives exceed the costs and are therefore economically justified from a national perspective. Alternatives that are shown not to be economically justified are not carried forward to the cost-effective analysis in the proceeding section. Note that both Phase I and Phase II, and the lined and unlined alternatives, all achieve the desired long-term erosion risk levels (i.e. each provide a similar level of residual risk for the associated flow/stage-level restriction). In other words, one risk reduction plan does not provide more risk reduction (mitigated damages) than another.

Water supply benefits (irrigation, M&I, and W&W) are calculated for each alternative based on their ability meet TCID water supply demands relative to the modeled historical water supply reliability scenario. The modeled historical reliability scenario is represented in the Operations Model as the water supply available from Scenario 9 in Table X-1 (i.e. the Truckee Canal returned to its modeled OCAP maximum capacity, 900 cfs). Figure XI-9 displays the average annual Project water supply shortage relative to the modeled historical reliability scenario as estimated by the Operations Model. As shown in Figure XI-99 there is a small reduction to the water supply shortage when the canal is improved to safely convey a peak operating flow above 350 to 600 ft^3/s (Phase I) and a marginally smaller reduction in shortages between Phase I and Phase II.

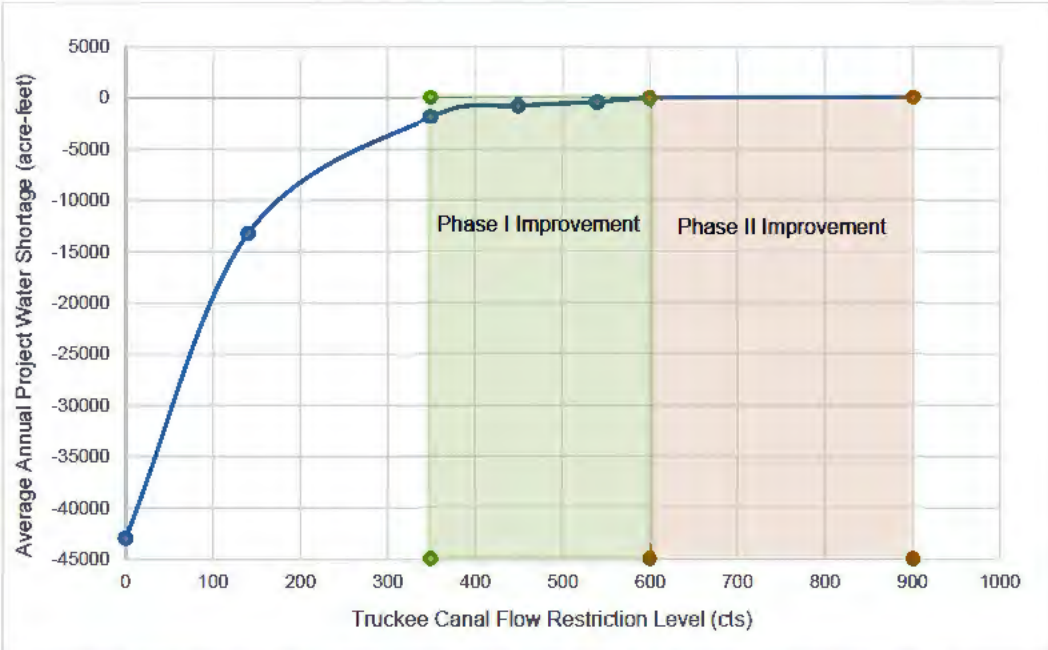

Figure XI-9.—Average Annual Modeled Project Water Shortage for Each of the Water Supply Scenarios Developed in the Operations Model Relative to the Modeled Historical Water Supply Reliability Scenario

Auxiliary benefits (recreation and hydropower) are calculated for each alternative based on the additional benefits provide by Truckee Canal diversions relative to the hypothetical without canal scenario, as modeled by the Operations Model. This comparison helps evaluate the benefits assigned to the no-action alternative (140 ft^3/s). Figure XI-10 displays the benefit of the Truckee Canal - average annual Lahontan Reservoir level relative to the without canal scenario. Figure XI-11 displays the average annual hydropower generated within TCID level relative to the without canal scenario. Again, the marginal increase in Project water availability above a maximum Truckee Canal flow level of 350 ft^3/s diminishes rapidly.

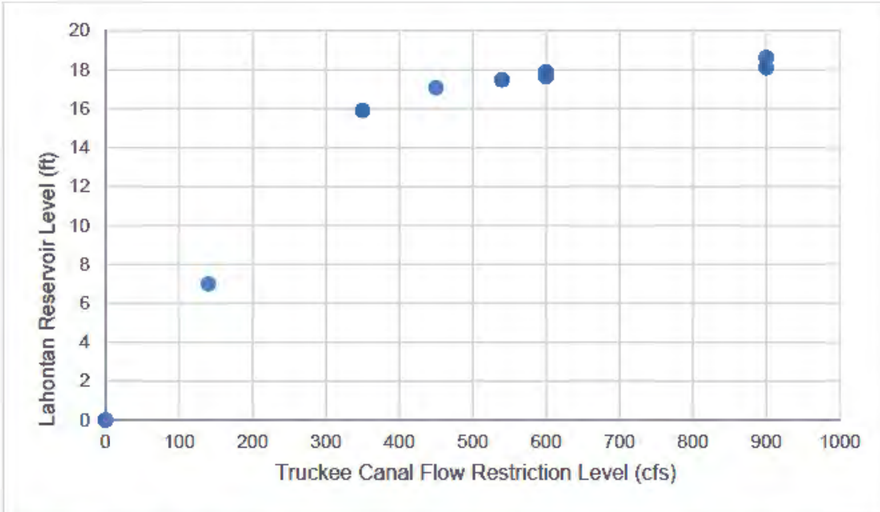

Figure XI-10.—Lahontan Reservoir Level (ft) Relative to the Baseline

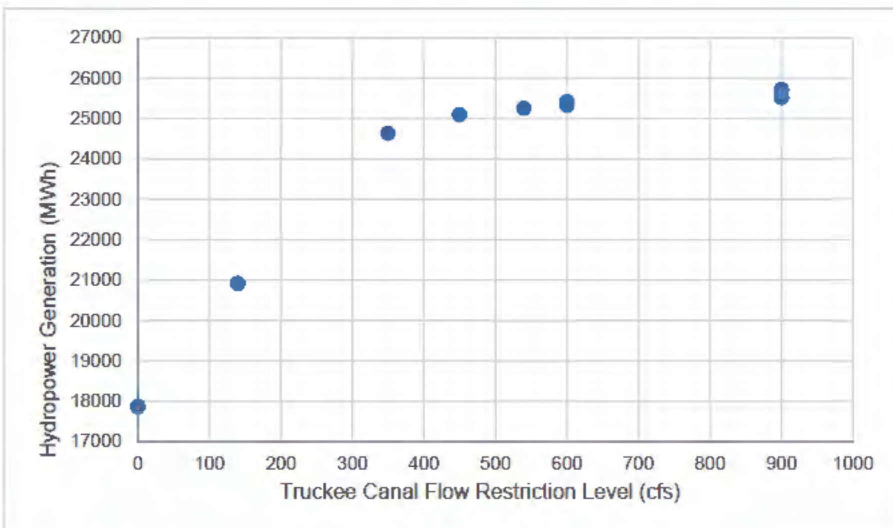

Figure XI-11.—Average Annual Hydropower Generation (MWh)

The benefits and costs of the CAS erosion risk reduction alternatives are evaluated in four steps. First, for each erosion risk reduction alternative, the results of the Operations Model are used to 1) derive the average annual water supply shortages expected in any given year to water users in TCID relative to the modeled historical water supply reliability scenario, and 2) derive the average annual changes to Lahontan Reservoir levels and hydropower generation relative to the without canal scenario in any given year. Second, the average annual lost water supply benefits due to water supply shortages (e.g., reduction in agricultural production) and the additional auxiliary benefits (recreation & hydropower) associated with each of the risk reduction alternatives are estimated as the product of benefit values established in the National Economic Benefit Valuation Section and the quantities derived in step one. Third, the total average annual water supply benefits (avoided costs) and auxiliary benefits provided by Phase II and the lined alternatives relative to Phase I and unlined alternatives are calculated. Fourth, the net benefits and benefit cost ratio are calculated of Phase II versus Phase I and lined versus unlined risk reduction alternative plans by comparing the present value of benefits estimated in step 3 with the estimated construction costs of the risk reduction plans.

Step 1: Derivation of Water Supply Shortages, Lahontan Reservoir Levels and Hydropower Generation Relative to Baselines

Table XI-14 provides the derived average annual water supply shortages, average annual changes to Lahontan Reservoir, and the average annual hydropower generation relative to their respective baseline scenarios (modeled historical water supply reliability and without canal scenarios). This comparison helps evaluate the benefits assigned to the no-action alternative (140 ft^3/s). Note that the modeled historical water supply reliability scenario is equivalent to the unlined Phase II scenarios and therefore the derived water supply shortage is zero for both of those scenarios.

Table XI-14.—Average Annual Operations Model Data for Each Alternative

	Average Annual Water Supply Shortage Relative to the Modeled Historical Reliability Scenario			Average Annual Quantity Relative to the Without Canal Scenario	
Internal Erosion Risk Reduction Alternative	Irrigation Water Supply (AF)	M&I Water Supply (AF)	Wetlands & Wildlife Water Supply (AF)	Lahontan Reservoir Level (ft)	Hydropower Generation (MWh)
Full Prism Canal Lining-Phase I	-111	5	-29	17.88	25,424.65
Full Prism Canal Lining-Phase II	553	18	95	18.61	25,716.96
Partial Prism Canal Lining-Phase I	-364	-4	-73	17.65	25,330.10
Partial Prism Canal Lining-Phase II	0	0	0	18.11	25,524.78
Embankment Cutoff Wall-Phase I	-364	-4	-73	17.65	25,330.10
Embankment Cutoff Wall-Phase II	0	0	0	18.11	25,524.78

Step 2: Calculation of Average Annual Lost Water Supply Benefits and Additional Auxiliary Benefits

Table XI-15 reports the estimated average annual lost water supply benefits due to water supply shortages and the additional auxiliary benefits associated with each of the risk reduction alternatives. The benefits are estimated as the product of net benefit values established in the National Economic Benefit Valuation Section and the quantities derived in step 1.

Table XI-15.— Average Annual Water Supply Benefits and Additional Auxiliary Benefits (2015$)

	Average Annual Benefits in TCID Relative to the Modeled Historical Reliability Scenario				Average Annual Auxiliary Benefits Relative to the Without Canal Scenario		
Internal Erosion Risk Reduction Alternative	Irrigation Water Supply	M&I Water Supply	Wetlands & Wildlife Water Supply	District Total	Lahontan Reservoir Recreation	Hydropower Generation	Total
Full Prism Canal Lining-Phase I	-$12,030	$179	-$2,316	-$14,167	$1,031,801	$306,382	$1,338,183
Full Prism Canal Lining-Phase II	$59,128	$680	$7,622	$67,430	$1,073,927	$318,326	$1,392,253
Partial Prism Canal Lining-Phase I	-$39,068	-$158	-$5,823	-$45,049	$1,018,529	$302,513	$1,321,042
Partial Prism Canal Lining-Phase II	$0	$0	$0	$0	$1,045,074	$310,462	$1,355,536
Embankment Cutoff Wall-Phase I	-$39,068	-$158	-$5,823	-$45,049	$1,018,529	$302,513	$1,321,042
Embankment Cutoff Wall-Phase II	$0	$0	$0	$0	$1,045,074	$310,462	$1,355,536

Step 3: Calculation of the Average Annual Benefits Attributable to the Phase II and Lined Alternatives

Table XI-16 reports the calculated total average annual water supply benefits in terms of avoided costs and the auxiliary benefits provided by Phase II relative to the Phase I. The total average annual benefits provided by Phase II relative to phase I are calculated as the difference between the two phases for each alternative.

Table XI-16.— Phase II Total Average Annual Benefits Relative to Phase I (2015$)

	Total Avg. Annual Water Supply Benefits under Phase II	Total Avg. Annual Water Supply Benefits under Phase I	Total Avg. Annual Water Supply Benefits: Phase II relative to Phase I	Total Avg. Annual Auxiliary Benefits under Phase I	Total Avg. Annual Auxiliary Benefits under Phase II	Total Avg. Annual Auxiliary Benefits: Phase II relative to Phase I
Full Prism Canal Lining	$67,430	-$14,167	**$81,597**	$1,338,183	$1,392,253	**$54,070**
Partial Prism Canal Lining	$0	-$45,049	**$45,049**	$1,321,042	$1,355,536	**$34,494**
Embankment Cutoff Wall	$0	-$45,049	**$45,049**	$1,321,042	$1,355,536	**$34,494**

Table XI-17 reports the calculated total average annual water supply benefits in terms of avoided costs and the auxiliary benefits provided by seepage reduction alternatives under Phase I and Phase II. The total average annual benefits provided by the seepage reduction alternatives relative to the non-seepage reduction alternatives are calculated as the difference between each alternative for Phase I and Phase II.

Table XI-17.—Total Average Annual Benefits Under Seepage Reduction Alt. Relative to Non-Seepage Reduction Alt. (2015$)

	Total Avg. Annual Water Supply Benefit: Seepage Reduction Alt.	Total Avg. Annual Water Supply Benefit: Non-Seepage Reduction Alts.	Total Avg. Annual Benefits: Seepage Alt. Relative to Non-Seepage Reduction Alt.	Total Avg. Annual Auxiliary Benefits: Seepage Reduction Alt.	Total Avg. Annual Auxiliary Benefits: Non-Seepage Reduction Alt.	Total Additional Avg. Annual Auxiliary Benefits: Seepage Alt. Relative to Non-Seepage Reduction Alt.
Phase I	-$14,167	-$45,049	**$30,882**	$1,338,183	$1,321,042	**$17,142**
Phase II	$67,430	$0	**$67,430**	$1,392,253	$1,355,536	**$36,718**

The seepage reduction benefits do not take into account the value of any potential increased efficiency to entities outside of TCID. Note that in non-water short years, due to the increased efficiency of the Truckee Canal through reductions in seepage, there would be an additional benefit provided by water remaining in the Truckee River that would otherwise have been diverted and lost through seepage. However, in water short years, in general, any increases in efficiency gained from the seepage reduction alternatives would only act to decrease shortages to water users in TCID.

Step 4: Calculation of the Net Benefits and Benefit Cost Ratio

Table XI-18 reports the calculated net benefits and benefit cost ratio of Phase II of the alternatives relative to Phase I. The net benefits are calculated as the difference between the present value of the average annual benefits in each benefit category and the estimated construction costs of Phase II for each of the alternatives. For each of the proposed CAS erosion risk reduction alternatives the construction costs of Phase II exceed the estimated benefits generated by Phase II relative to Phase I and the benefit cost ratio is below one; Therefore, Phase II is not economically justified for any of the alternatives evaluated.

Table XI-18.—Net Benefits of Phase II of the Alternatives Relative to Phase I (2015$)

	Phase II Constructio n Costs	Present Value of Annual Water Supply Benefits: Phase II relative to Phase I	Present Value of Annual Auxiliary Benefits: Phase II relative to Phase I	Present Value of Total Annual Benefits: Phase II Relative to Phase I	Net Benefits of Phase II (benefits-costs)	Benefit Cost Ratio (benefits/ costs)
Full Prism Canal Lining	$27,000,000	$2,150,227	$1,424,387	$3,574,614	**-$23,425,386**	0.13
Partial Prism Canal Lining	$16,000,000	$1,187,122	$908,696	$2,095,818	**-$13,904,182**	0.13
Embankment Cutoff Wall	$14,000,000	$1,187,122	$908,696	$2,095,818	**-$11,904,182**	0.15

Table XI-19 reports the calculated net benefits and benefit cost ratio of the seepage reduction alternative (Full Prism Canal Lining) relative to the non-seepage reduction alternatives (partial prism canal lining and embankment cutoff wall). The net benefits are calculated by computing the present value of the average annual benefits in each benefit category over the POA and then comparing them with the estimated construction costs of the seepage reduction alternative for each phase. For both Phase I and II of the proposed seepage reduction alternative (full canal prism lining) the construction costs for a full geomembrane/concrete lined prism do not exceed the estimated benefits generated (negative net benefits) and the benefit cost ratio is below one; Therefore, a full geomembrane/concrete lined prism is not economically justified for either Phase I or Phase II.

Table XI-19.—Net Benefits of the Seepage Reduction Alternative Relative to the Non-Seepage Reduction Alternatives (2015$)

	Seepage Reduction Alt. Construction Costs	Present Value of Annual Water Supply Benefits: Seepage Reduction Alt.	Present Value of Annual Auxiliary Benefits: Seepage Reductio n Alt.	Present Value of Total Annual Benefits: Seepage Reduction Alt.	**Net Benefits of Seepage Reduction Alt. (benefits-costs)**	Benefit Cost Ratio (benefits /costs)
Phase I	$28,000,000	$813,796	$451,568	$1,265,364	**-$26,734,636**	0.05
Phase II	$27,000,000	$1,776,901	$967,258	$2,744,160	**-$24,255,840**	0.10

a. Benefit-Cost Analysis Summary of Findings

The above benefit-cost analysis determined that the construction costs of Phase II for all of the proposed CAS erosion risk reduction alternatives exceed the benefits generated by Phase II relative to Phase I. Therefore, Phase II of all of the risk reduction alternative plans are not economically justified and are not carried through to the cost effective analysis.

Further, the benefit-cost analysis also determined that the benefits generated to water users within TCID by the seepage reduction alternatives do not exceed the construction costs of the seepage reduction alternative. Therefore, the seepage reduction alternative (a full geomembrane/concrete lined prism) is shown to not be economically justified and is not carried forward to the cost effective analysis.

3. Cost Effective Analysis

In this Section, the potential future benefits and costs of the Newlands Project are analyzed over the POA—in which the potential future benefits and costs of the Newlands Project are estimated based on its impact to the Nation's welfare (e.g., the reduction in the national level of agricultural production due to an anticipated future loss of project water). All future project costs and benefits expressed as monetary values are *discounted* back to their present value in the designated 2015 base year using the Planning Rate.

A cost effective analysis is used to determine the risk reduction alternative that meets the long-term risk reduction goals at the least cost. The costs of the risk reduction alternatives are calculated as the sum of the present value of the construction costs and any lost authorized Project purpose benefits under the proposed risk reduction plans and the non-structural long-term no action alternative. Lost Project benefits are calculated as the product of any reduction in Project water supply to irrigators, M&I, and W&W water users under the proposed risk reduction alternatives relative to the modeled historical water supply reliability scenario and the unit values established in Section XI.B.1.

Table XI-20 provides the summary results of the cost effective analysis. Phase I of Risk Reduction Plans No. 3 and 5, and the long-term no-action alternative were evaluated. All of the evaluated risk reduction alternative plans meet the long-term risk reduction goals of the Newlands Project (i.e., all provide a similar level of residual risk for the associated flow/stage-level restrictions). Phase II of the risk reduction plans and the full canal lining alternative were found to be economically unjustified in XI.B.2 and, therefore, are not considered in the cost effect analysis. Of the three risk reduction plans analyzed, Risk Reduction Plan No. 3 Phase I is found to be the least cost alternative as well as provide the largest amount of auxiliary benefits. However, Risk Reduction Plan No. 5 Phase I is only slightly more expensive and does not reduce auxiliary benefits by a significant amount.

Table XI-20.—Cost Effective Analysis Summary Results

Risk Reduction Plans	Long-Term No Action Alternative	Risk Reduction Plan No. 3 Phase I	Risk Reduction Plan No. 5 Phase I
Costs			
Construction Costs	$21,200,000	$37,000,000	$37,700,000
Present Value: Construction Costs	**$19,785,000**	**$29,241,000**	**$29,569,000**
Lost Irrigation Benefits	$71,250,000	$2,161,000	$2,184,000
Lost M&I Benefits	$417,000	$9,000	$9,000
Lost W&W Benefits	$9,725,000	$320,000	$323,000
Total Lost Benefits	$81,392,000	$2,490,000	$2,516,000
Present Value: Total Lost Benefits	**$43,652,000**	**$1,391,000**	**$1,408,000**
Grand Total Costs	$102,592,000	$39,490,000	$40,216,000
Grand Total Present Value: Total Costs	**$63,436,000**	**$30,632,000**	**$30,977,000**
Auxiliary Benefits			
Recreation Benefits (Baseline)	$20,563,000	$51,776,000	$51,757,000
Hydropower Benefits	$6,313,000	$15,384,000	$15,378,000
Total Auxiliary Benefits	$26,876,000	$67,160,000	$67,135,000
Present Value: Total Auxiliary Benefits	**$14,414,000**	**$35,963,492**	**$35,947,000**

C. Ability-to-Pay Study

The goal of this ability-to-pay study (ATP Study) is to assess the financial capability of TCID to pay for existing and additional federal and non-federal Project costs over the specified POA. Specifically, this ATP Study assesses TCID's ability to generate revenue in order to pay for current Project costs and any additional costs attributable to the proposed risk reduction plans. The ATP Study is conducted in accordance with general accounting principles and Reclamation's *Technical Guidance for Ability to Pay and Payment Capacity*, dated May 2004.

1. District-level Payment Capacity

District-level ATP is defined as the sum of all revenue streams plus any excess district-level reserves. TCID generates revenue from the following two primary sources: assessments on water users and the production of hydropower. TCID also realizes revenue through other means including interest revenue and general assessments on non-water users.

a. Payment Capacity

TCID's primary source of revenue comes from assessments on water users within the District. The maximum amount of assessments that water users are financially capable of paying to the District is determined by their payment capacity. In what follows, a payment capacity analysis is conducted for each category of water user within the District and summed to determine the total water users payment capacity.

(5) Irrigation Water Users Payment Capacity

The per-acre payment capacity of commercial agricultural operations within the District is estimated by conducting a farm budget analysis (FBA) of the representative farms identified in Section XI.B.1.a.(1) *Approach to Valuing Irrigation Water Supply Benefits*. The total district-level payment capacity of irrigation water users is calculated as the product of the estimated per-acre payment capacity and the number of District acres of each representative farm type.

The FBA approach has three main steps. First, the agricultural operating and non-operating revenues and expenses of each farming operation are estimated. Second, the average annual net farm income (NFI) of each operation is computed as the difference between the estimated revenues and expenses. Third, allowable returns to the farm family for labor, equity, and management are determined and subtracted from the computed NFI of an operation to derive the farm-level payment capacity per acre of a representative operation.

The farm budgets developed for the payment capacity study are very similar to those constructed for the estimation of irrigation benefits in Section XI.B.1.a.; however, they differ in several key areas. The differences between farm budget inputs used in a payment capacity analysis versus a benefit analysis are described below and displayed in Table XI-21.

First, the two studies are undertaken for different objectives. In general, an economic benefit analysis is conducted from a national perspective and attempts to estimate the total benefits of a project to the Nation. A payment capacity study is a financial analysis that endeavors to assess the financial situation of representative farming operations within a district. This difference in objectives necessitates the use of a different conceptual approach to arrive at the results. In a payment capacity study, the financial situations of agricultural operations are estimated under current with-conditions only. Conversely, in a benefit analysis, the economic effects of a proposed action are calculated as the difference in the net farm returns of agriculture operations under with- and without-conditions.

Second, a payment capacity study attempts to differentiate and approximate the debt and equity (operator-owned) portions of on-farm capital investments and annual operating capital. On the debt portion, operators face interest costs associated with the financing of capital for investment in the operation. On the equity portion, operations are allowed to deduct an allowable return based on the amount of owner invested capital (non-debt). Thus, estimates developed for a payment capacity study reflect both the average annual interest costs incurred by area farmers on the debt portion and an annual return on equity on the remainder. Conversely, in a benefit analysis, the debt and equity portions of representative operations are not calculated separately, but rather the opportunity cost to the nation of investing capital in an agricultural operation is taken into account. This opportunity cost reflects the potential rate-of-return forgone to the nation by operators not investing capital in an alternative venture.

Third, to the extent possible, a payment capacity study attempts to reflect local commodity prices within the study area for the study year using a detailed price forecasting approach. A benefit study, on the other hand, estimates commodity prices as the three year average of state or national level prices.

Fourth, in both benefit and payment capacity studies, a return-to-management is permitted; however, the calculation for determining a return-to-management differs. In a benefit study, return-to-management is calculated as 6 percent of variable costs and in a payment capacity study as 10 percent of NFI.

Table XI-21.—Differences between Benefit and Payment Capacity Farm Budgets

Category	Benefit Budget	Payment Capacity Budget
Debt Load	100% (Opportunity Cost)[6]	Real Estate 5.64% Non-Real Estate 14.88%
Interest Rates	2.875% (Planning Rate)	Real Estate 4.82% Non-Real Estate 5.00%
Commodity Prices	3-yr Avg of State & National	Detailed Commodity Price Forecast
Return to Management	6% of Variable Costs	10% of NFI
Return to Equity	None	3% of Equity Portion of On-Farm Capital Investments

[redacted]

[6] In a benefit study, in order to reflect the opportunity costs of capital, operations are modeled as carrying a debt load of 100% that is financed at the Evaluation Rate.[12]

[7] (156/200)*(.9) ≈ 70% , (44/200)*(.9) ≈ 20%

[8] (160/200)*(1) ≈ 80% , (40/200)*(1) ≈ 20%

[redacted]

[redacted]

[redacted]

[redacted]

[9] Weighted district-level irrigation payment capacity per acre = (total district-level payment capacity) / (total irrigated commercial agriculture acreage in district)

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[11] No data was available for a break out of administration fees for 2011, so the average value over 2012-2015 was used as a proxy value. In 2015, administration fees are charged at $110 per parcel.

[redacted]

3. *Ability-to-Pay Results*

In summary, the goal of this ATP study is to forecast, on an annual basis, TCID's financial ability to cover all existing commitments and any additional costs incurred under the proposed risk reduction alternatives. In general, a district's annual average ATP is calculated by subtracting the estimated annual district-level expenses from the sum of the estimated annual district-level payment capacity and non-operating income. For this TM, any available excess reserves were not included in the assessment of the District's ATP due to uncertainties surrounding the financing mechanism and preferred risk reduction alternative.

This ATP study does not take into account potential changes in Project water supply conditions under risk reduction alternatives other than Phase I of risk reduction plans No. 3 and 5, and holds all assumptions constant to those in the study year. Therefore, the results of the ATP should not be relied on for forecasting TCID's ATP under conditions other than those present in the study year, including risk reduction alternatives that reduce Project water supplies.

[redacted] A positive annual ATP indicates that the District as a whole has the financial capability of paying all existing annual expenses associated with operating the Newlands Project. Further, any of the RRAPs with an annual cost of less than the estimated ATP are considered financially feasible.

[redacted]

XII. Constructability Review

A constructability review was completed to make a preliminary estimate of the construction schedule, canal outage duration, site access, and extent of the disturbance area(s). Alternatives 1, 2, 3, and 5 require work within the canal's prism for reshaping, fill placement and liner installation. A canal outage will be required for the construction duration. Ideally all work requiring a canal outage would begin in mid to late August following deliveries in the Fernley Division. Lahontan Reservoir storage should also be considered when scheduling canal outages. Each implementation Phase has been subdivided into segments about 1 to 2 miles long (see Figure IX-1). The canal lining alternatives will likely be constructed with a chain of automated equipment that shape and prepare the channel, install the geomembrane, and place the concrete or soil cover materials similar to what is shown in Figure V-2. Some hand placements will be required at tie-ins with the checks, at bridge crossings and where segments adjoin. Much of the work will be done from within the canal's prism with material deliveries made from the embankment crest. Multiple access points will be required for material deliveries. The installation rate is expected to be about 1 mile per month, with potentially faster rates in longer reach sections. Construction of Phase I and II are expected to take about 6 months each. The cumulative construction duration will be much longer if the Phases are further divided into smaller contracts.

Reconstruction of the embankment (Alternative 5) will likely include excavation of the embankment in one location and then placement of that material in an adjacent area where the excavation had already been made. This would be done to avoid double handling. Disturbance would be limited to the existing embankment and canal prism footprint. Multiple access points will be required for imported fill deliveries. Stockpile locations adjacent to the canal every few thousand feet would also be required. Embankment reconstruction is also expected to have an installation rate of about 1 mile per month.

Alternative 4 – Embankment Cutoff Wall can be constructed from the crest of the left embankment. A canal outage will not be required during installation. Deliveries of materials and equipment can be done from the embankment crest. The installation rate has been estimated to be about 250 feet per day (about one mile per month). A slower installation rate is expected where pre-driving or pre-trenching is required due to cobbly soils. This alternative is expected to have the least disturbance and canal outage impacts.

A preliminary constructability review for replacement of the check structures was completed in September, 2015. That study indicated replacement of the check structures would take about 5 to 6 months to complete. Fabrication of the gates and other mechanical equipment would start about 6 months before mobilization. Three canal outage options were considered: 1) full outage for the duration of construction, 2) periodic outages during construction of coffer dams and diversion channels around the work site, 3) and periodic outages during construction of coffer dams and routing pumps/pipes around the work area. In the second and third scenarios, the canal outage would be about 1 to 3 months. Multiple checks could be replaced at the same time and ideally be done during an outage for liner installation or embankment reconstruction.

Construction of Alternatives 7 – Drainage Crossings and 9 – Passive Spillways will require a canal outage. The cross drainage and passive spillway structures will have foundations which extend below the normal canal water surface. A canal outage of about one month will be required to construct the foundations. The remainder of the work could be done with water in the canal. Construction of the drainage channels downslope of the canal would not require an outage. Ideally the channels would be completed before the crossings or spillway structures are installed. It is expected each of the drainage structures will take about 2 to 3 months to construct. The downslope drainage channels are each expected to take 3 to 6 months to construct. Construction of the downslope drainage channels will require right-of-way purchases and a number of road and railroad crossings.

Construction of the gated wasteway structures will have a similar duration as the check structure replacement. A 3 to 4 month canal outage will be required until the gates have been installed. Construction of the downslope drainage channels will be similar to those described for Alternatives 7 and 9.

The detention/infiltration pond construction includes about 125,000 cubic yards of excavation. This work is expected to take about 1 to 2 months to complete. Construction of the perimeter berms, training dikes, turnout structure, piping and spillway channel can be done during this period. The total construction duration for the detention pond construction is estimated to be about 3 to 4 months each. Each of the detention ponds will require about 12 acres of land to be acquired.

XIII. Corrective Action Study Findings

The CAS evaluated a number of safety improvement alternatives to lower the risk of internal erosion through the embankment and overtopping during a flood event while providing a peak operating flow ranging from 600 to 900 ft^3/s. The alternatives were evaluated based on the degree of risk reduction they provide, cost, constructability, and impacts to diversions during construction. Combinations of the safety improvement alternatives were developed to achieve the desired risk reduction criteria. The recommended risk reduction alternative plans for feasibility-level development are listed below.

A. Recommended Risk Reduction Alternative Plans for Feasibility-level Development

Results of the CAS studied have been used to identify two risk reduction plans for feasibility-level development. These include:

- Risk Reduction Plan No. 3: Replace the check structures, Phase I and Phase II embankment cutoff wall, and two new upslope detention ponds.

- Risk Reduction Plan No. 5: Replace the check structures, Phase I and Phase II partial geomembrane/concrete cover liner system, and one new wasteway and drainage channel in the Lahontan Reach.

These two risk reduction plans provide the required risk reduction and cost about 20 to 60 million dollars less than the other risk reduction plans. Should the ongoing hydrologic loading analysis indicate the flood loadings are lower, then the costs for the hydrologic protective features could be less.

A modification to both Risk Reduction Plans 3 and 5 could include addition of lining in the Lahontan Reach to reduce seepage losses and improve efficiency. The recommended lining system for the Lahontan Reach is Alternative 3 – geomembrane/soil cover, as is about half the cost of the geomembrane/concrete cover alternative. Fully lining the Lahontan Reach will also lower internal erosion risks from PFM11 during flooding. Benefits of lining the Lahontan Reach should be further developed during the feasibility-level study.

Results of the hydrologic analysis indicates implementation of Phase I with a peak operating flow ranging from 350 to 600 ft^3/s will achieve the historic water supply reliability. Should decision makers choose to only implement Phase I of the linear canal embankment improvements, then additional hydrologic protective features will be required in the Fernley Reach (not presently included in risk reduction plan No. 3 and 5). This might include a combination of a detention pond at pour point No. 8 and a wasteway near the Farm District Road Seep. Without the Phase II improvements, there would remain unimproved segments that would continue to be vulnerable to internal erosion during flood induced stage level rise. Additionally, the limited conveyance capacity in those areas not improved would limit the ability to convey flood inflows through the Fernley Reach for discharge further downstream leading to potential for overtopping.

The economic and financial feasibility analyses indicated negligible increase to the economic benefits when the canal is improved to provide a peak operating flow ranging greater than 350 to 600 ft^3/s (Phase I), and that Phase II is not economically justified. The financial feasibility analyses also indicated the higher costs for the full geomembrane/concrete cover alternatives are not outweighed by the economic benefits from the seepage reduction/water savings. The costs/benefits analysis indicated Risk Reduction Plan Nos. 3 and 5 (Phase I only) are financially feasible and TCID would have the ability to repay the government's loan installments of $1,000,000 per year.

B. Canal Efficiency Modifications

The feasibility-level study should continue to incorporate design features or operations controls that improve efficiency and minimize seepage losses. During the feasibility-level study the Risk Reduction Plan Nos. 3 and 5 should be considered with the expanded use of Alternative 3 (geomembrane/soil cover lining) in the Lahontan Reach. Both Risk Reduction Plan Nos. 3 and 5 include geomembrane/soil cover lining at the Steam Pad and Red Barn Seep areas. The use of geomembrane/soil cover lining throughout the remainder of the Lahontan Reach will further reduce seepage losses and reduce internal erosion risks. The addition of about 10 miles of geomembrane/soil cover lining the Lahontan Reach would add about $23,000,000 to the total project costs for Risk Reduction Plan Nos. 3 and 5 listed in Table ES-3. Justification for addition of the geomembrane/soil cover lining in the Lahontan Reach would need to be evaluated during the feasibility-level study.

XIV. Recommendations and Additional Data Needs

A. Survey Data

The latest aerial survey of the Truckee Canal was done in 2008. Since then, TCID has removed about 2 to 3 feet of sediment from the Fernley and Lahontan

Reaches. A cross section survey was made by the MP survey branch in 2014 but does not provide complete coverage. An updated aerial survey of the canal in its current condition is recommended for use during the feasibility-level design. The aerial survey will need to be scheduled during a planned canal outage. This will allow for a better estimation of the earthwork volumes for each of the alternatives.

The work should also include a detailed survey of the right-of-way limits and all utilities within 200 feet of the canal centerline. This information will be needed to identify construction disturbance outside of the existing right-of-way and any utilities requiring relocation.

B. Field Trials

A brokered project proposal has been issued to Reclamation’s Research and Technology Office to investigate the viability of synthetic sheet piles to lower the internal erosion risks at canals. The Truckee Canal has been selected as the field trial location. The objectives of the study are to investigate the installation rate in a range of soil conditions, monitor seepage reduction where a positive cutoff can be achieved and to evaluate end effects. This work will benefit the development of synthetic sheet piles for the Truckee Canal and for use at other Reclamation canals.

A field trial is also recommended to evaluate the viability of a detention/infiltration pond. A subsurface investigation and percolation testing program should be considered to evaluate whether the infiltration well system within the detention pond could be used as part of the City of Fernley’s plans for an aquifer recharge system (ARS).

C. Land Acquisition Evaluation

Alternatives 7, 8 and 9 require downslope drainage channels to convey flood runoff away from the canal. The drainage channels will have a capacity of about 600 ft^3/s with a channel width of about 50 feet. The channel alignments shown on Figure IX-1 generally follow existing irrigation drain rights-of-way. The existing rights-of-way are about 40 feet wide and will need to be expanded. A survey of the drain alignments will be required and plans for land acquisition should be investigated.

The detention/infiltration pond alternative will also require land acquisition. About 12 acres will be required for each site. The site near Pour Point No. 8 is currently undeveloped but is apparently slated for residential development. The potential use of this site should be communicated to the City of Fernley and considered when reviewing plans for future development. The potential detention pond sites at Pour Points No. 16 and 19 are undeveloped but appear to be private land.

D. Hydrologic Hazard Analysis Update

The 2016 HHA [6] used an aerial reduction factor to account for variations in the rainfall totals throughout the contributing basins. It was also assumed that each of the contributing basins would be impacted at the same time. Since the critical design storm is a thunderstorm, the 2016 RET judged it was unlikely that the thunderstorm would be of sufficient size to impact all of the drainages, and if it did there would be some time lag as it traveled along the south side of the canal. While there remains uncertainty in the 2016 HHA findings, the RET judged the latest study does a good job at estimating the potential runoff from the individual drainages but recommended that further work be done to understand the aerial extent and expected travel patterns for the design storm event.

The LBAO has contracted with a local engineering firm and staff from NOAA's Meteorologic Group in Reno, Nevada to further refine the hydrologic loadings. This study will then be used to refine the hydrologic protective features designs during the feasibility-level design phase.

The existing HEC-RAS model for the Truckee Canal should be updated to reflect the current condition and operations. A survey of the canal prism should be made to reflect the full scope of the recent sediment cleaning activities. Flow/discharge measurements and a water surface profile survey should be completed to "re-calibrate" the updated HEC-RAS model. The calibration efforts should capture multiple flow conditions (low versus high) and seasonally (vegetated versus unvegetated). The updated HEC-RAS model should then be used to complete flood routings with the updated hydrologic loadings.

E. Additional Concepts to be considered during the Feasibility-Level Study

During completion of the CAS, additional concepts were identified that have the potential to add value and reduce costs. They include:

- Construction of detention ponds downslope of the canal in combination with wasteways and passive spillways. The detention ponds could be used to regulate releases downslope and avoid the need to enlarge the drainage channels and costly roadway and railroad crossings. Water stored in the downslope detention pond could also be pumped back to the canal. There is an existing pond about 1,800 feet northeast of the canal near the Mason Check Structure (site of the proposed Hazen Wasteway). This site appears to lie with Reclamation owned lands.

- In areas where the right canal bank is in fill, the linear embankment improvements measures used to improve the left embankment should be used to improve the right embankment.

- During a rain event in January, 2017 runoff impounded against the right canal bank in the area just upstream of the SH-95 bridge crossing in the Fernley Reach. The impounded water led to a failure from both internal erosion and overtopping. The feasibility-level design should include measures to either prevent water from impounding against the right canal bank (i.e. include drainage notches), include surface erosion protection to minimize the effects from overtopping, and include internal erosion protective features.

- Instead of replacing the Mason Check Structure, investigate the viability of replacing the Lahontan Reach turnouts configured to make the needed releases without checking the water surface. Costs for replacing each check structure was estimated to cost about $3,000,000. Replacing the turnouts could be significantly less.

- Further analysis of the water supply reliability may be required to define an acceptable alternative and its parameters. The CAS reflects the primary criterion of 9 water short years out of 100 years as modeled for the 1997 Adjusted OCAP in defining water supply reliability. The peak operating range of 350 ft^3/s to 600 ft^3/s encompasses the case for meeting the nine water short years out of 100 years at 540 ft^3/s, but that peak flow condition does not meet the other 1997 Adjusted OCAP modeling criterion of shortage magnitude. Future analysis may be needed to determine what balanced operating conditions will be used to define water supply reliability and the impact on final project design and economic analyses.

- The next economic analysis should include all applicable criteria to support the requirements of the EIS. Additionally, the comparison for the water supply reliability in alternative selection should consider an alternate baseline of the no action alternative (140 ft^3/s). This supports the selection of maximum flow in the canal that is economically supported.

F. Interim Actions to Manage Risks

The highest risks at the Truckee Canal are from internal erosion during an elevated stage level and from flood overtopping. The internal erosion risks should continue to be managed through the use of a flow/stage level restriction. Recommendations listed in the Decision Document report titled: Flow/stage level

Restriction Recommendations for the Truckee Canal, [17] should be implemented until the risk reduction alternative plans can be fully implemented.

An interim activity to address hydrologic risks would be to form a fuse plug (i.e. low area in the embankment crest) location where the consequences are the lowest. Areas that might be considered include canal station 1150+00 near the Steam Pad Seep or near station 1370+00. Both of these areas were characterized as having a consequence Level 0. Once the hydrologic protective features have been installed the fuse plugs could be removed.

The Bango Check has historically been used to check the water surface in the Lahontan Reach extending upstream of the Mason Check location (about 5 miles upstream). Checking the water surface in the lower Lahontan Reach has apparently attributed to sediment accumulation and aquatic vegetation development. The sediment and vegetation load in addition to reduced freeboard when the water surface is checked attribute to a higher risk of hydrologic overtopping. Checking the water surface higher than what is need to make deliveries at turnout TC-13 should be avoided.

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Appendix A

Appraisal-level Quantities and Cost Estimates

included on CD

Appendix B

RiverWare Modeling for the Truckee Canal Corrective Action Study

included on CD

Appendix C

Economic and Financial Feasibility Analysis Data and Assumptions

included on CD