NEWPORT
OREGON

AGENDA & Notice of Work Session for City Council

The City Council of the City of Newport will hold a work session on Tuesday, September 8, 2015, at 12 P.M. The work session will be held in Conference Room A at City Hall, located at 169 S.W. Coast Highway, Newport, Oregon 97365. A copy of the agenda follows.

The meeting location is accessible to persons with disabilities. A request for an interpreter for the hearing impaired, or for other accommodations for persons with disabilities, should be made at least 48 hours in advance of the meeting to Peggy Hawker, City Recorder at 541.574.0613.

The City Council reserves the right to add or delete items as needed, change the order of the agenda, and discuss any other business deemed necessary at the time of the meeting.

CITY COUNCIL WORK SESSION
Tuesday, September 8, 2015 – 12 P.M.
Council Chambers

A. Call to Order

B. Detailed Presentation From HDR Engineering, Inc. – On the Seismic Evaluation of the Big Creek Dams No.1 and No. 2

C. Adjournment

CITY MANAGER'S REPORT AND RECOMMENDATIONS

Agenda#B:
Meeting Date: 9/8/15

Agenda Item:
Council Work Session – Big Creek Dams 1 & 2 Feasibility Study

Background:
On Tuesday, September 8 at noon, the City Council will be reviewing and discussing the Big Creek Dams and Seismic Feasibility Report. This is a continuation of discussions that occurred in July. The City Council had requested additional information following that presentation prior to making a decision on accepting the report and providing any direction relating to the alternatives identified in the report. Public Works Director, Tim Gross, has invited Keith Ferguson, the lead engineer from HDR Engineering on the study, Tia Cavendar of Chase Park Grants regarding funding opportunities moving forward, and Keith Mills from the State of Oregon Water Resources Dam Safety Division.

Please note that no action is scheduled following this meeting. Following the work session, and considering the questions and suggestions from the Council, we will prepare a report for action for a future Council meeting.

Recommended Action:
None.

Fiscal Effects:
None.

Alternatives:
None recommended.

Respectfully submitted,

Spencer R. Nebel
City Manager

Agenda Item # WS.B.
Meeting Date Sept. 8, 2015

CITY COUNCIL AGENDA ITEM SUMMARY

City Of Newport, Oregon

Issue/Agenda Title: Council Workshop: Big Creek Dams 1 and 2 Feasibility Study Discussion

Prepared By: TEG Dept Head Approval: TEG City Manager Approval:

Issue Before the Council:

Discussion of the feasibility study for Big Creek Dams 1 and 2

Staff Recommendation:

None. For discussion purposes only.

Proposed Motion:

None. For discussion purposes only.

Key Facts and Information Summary:

On Tuesday, September 8th we will be discussing the Big Creek Dams and the seismic feasibility report at a workshop at 12:00 p.m. HDR Engineer, Keith Ferguson; Oregon Water Resources Dam Safety Engineer, Keith Mills; and Chase Park Grants President, Tia Cavendar will be in attendance to discuss the report and findings, what the City's obligations are regarding the dams, and what potential funding opportunities are available. HDR has made some modifications to the Executive Summary of the report based upon requests from OWRD. I have attached a revised copy of the executive summary and have also included a hardcopy in the Council Office at City Hall.

There is no proposed action at the September 8, 2015 regular Council meeting, because City Staff would like to make sure the Council has all of their questions answered before considering any proposed actions.

During this work session, we will be discussing three courses of action that are available to the City. First, the City can choose to do nothing. Oregon Water Resources Dam Safety has indicated that there are serious consequences and potential enforcement action that may take place if the City chooses not to address these dam structures. Big Creek Dams 1 and 2 have identified deficiencies and it is the responsibility of the dam owner, the City of Newport, to address these deficiencies. Keith Mills from Oregon Dam Safety will address this option at the work session.

Second, the City can choose one of the alternatives identified within the feasibility study. If this is the case, then the Council will need to consider two actions at a future Council meeting. First, City Staff needs direction regarding what preferred alternative the Council would like to pursue. This does not obligate the City to any course of action, but rather provides direction to Staff and the City's consultants so we can further define the potential costs and impacts of any preferred alternative. The feasibility study presented at this work session provides order of magnitude cost estimates only, to allow the Council to understand the cost differences between the alternatives; not the cost for the entire project from beginning to end. By identifying a preferred alternative, Staff and the City's consultants can focus on one preliminary design and can prepare estimates of cost for the entire project based upon that alternative. To conduct this level of effort for all the alternatives is time consuming and expensive.

Second, the Council needs to approve the feasibility study. The approval can be worded in many ways to the Council's satisfaction to ensure that the City is not locking itself into an un-reversible course of action, but is necessary for the pursuit of State and Federal Funding. A requirement of Oregon Water Resources and US Bureau of Reclamation grant applications is that the project must be part of an approved plan. Without an approval of the feasibility study, these agencies do not have confidence that the grantee has done their due diligence in fully vetting the costs and impact of the project. Tia Cavender with Chase Park Grants will be talking about financing strategies for the Dam project.

A third course of action the City can consider is the development of Rocky Creek. This is an "and" rather than an "or" option because the City will still need to address the Big Creek Dams in some fashion. The recommendation of City Staff is that the City continue to pursue the development of Rocky Creek concurrently with the Big Creek Dams remediation. The potential exists to develop this water source as a regional opportunity rather than for Newport alone, but it will be several years for this process to develop. Currently the City has a water rights application for Rocky Creek in review with Oregon Water Resources. Tia Cavender will talk about a potential basin study in this area that will be funded entirely through the Bureau of Reclamation and OWRD. This study will identify water availability and the impact of climate change on Rocky Creek over a long period of time. On a regional level, this study can serve as a forum to bring our neighboring communities together to talk about the regional possibilities of Rocky Creek.

<u>**Other Alternatives Considered**</u>:

N/A

<u>**City Council Goals**</u>:

N/A

<u>**Attachment List**</u>:

- Seismic Evaluation of Big Creek Dams No. 1 and 2 Engineering Evaluation and Corrective Action Alternatives Executive Summary

<u>**Fiscal Notes**</u>:

None

Seismic Evaluation of Big Creek Dams No. 1 and 2

Phase 3 – Engineering Evaluation and Corrective Action Alternatives
Newport Big Creek Dams

Newport, Oregon

June 30, 2015

Revised August 28, 2015

Keith A. Ferguson, P.E.
Principal Engineer

Contents

Tables

Figures

Appendixes

Acronyms

ac-ft	acre-feet
Alt	Alternative
BC 1	Big Creek Dam No.1
BC 2	Big Creek Dam No.2
BH	Borehole
CIUC	Isotropically Consolidated Triaxial Compression
CPT	Cone Penetration Testing
CSR	Constant Strain Rate
CSTP	Continuous Standard Penetration Testing
CycDSS	Cyclic Direct Simple Shear
DSS	Direct Simple Shear
FLAC	Fast Lagrangian Analysis of Continua
FOS	Factors of Safety
FSV	Field Shear Vane
H	Horizontal
HDR	HDR Engineering, Inc.
LIR	Load-Increment Ratio
MEG	Marine + Earth Geosciences
NEPA	National Environmental Policy Act
NHPA	National Historic Preservation Act
NPDES	National Pollutant Discharge Elimination System
NSHM	National Seismic Hazard Maps
OCR	Overconsolidation Ratio
ODFW	Oregon Department of Fish & Wildlife
PGA	Peak Ground Acceleration
PMF	Probable Maximum Flood
PSHA	Probabilistic Seismic Hazard Analysis

RCC	Roller compacted concrete
SCPT	Seismic Cone Penetration Testing
SCPTu	Seismic Cone Penetration Testing with pore pressure measurements
SHANSEP	Stress History and Normalized Soil Engineering Properties
SPT	Standard Penetration Testing
UC	University of California
USGS	United States Geological Survey
V	Vertical

Executive Summary

HDR Engineering Inc. (HDR) has completed the Phase 3 assessment of the static and seismic stability of Big Creek Dam No. 1 (BC 1) and Big Creek Dam No. 2 (BC 2) for the City of Newport (City). This assessment included 1) an update of the seismic hazard characterization and characteristic earthquake time histories at the site based on the most recent research; 2) additional site characterizations including borings and cone penetration testing, sampling and laboratory testing; 3) analysis and evaluation of the field and laboratory test results; 4) developing a more detailed and comprehensive geologic model of the two dam sites along with generalized profiles and cross-sections for engineering evaluations; 5) an update of the previously completed seepage, static and post-earthquake stability analysis; 6) evaluating the expected seismic response (deformations) of both existing dams to a range of potential earthquakes at the site; 7) developing and evaluating alternatives for corrective actions for BC 1 and BC 2; 8) development of decision level cost estimates for the corrective action concepts; and 9) providing a preliminary environmental permitting overview for the corrective action concepts. The findings from this evaluation are summarized in this report.

Verification of Seismic Response Deficiencies

The static and post-earthquake stability and seismic response analyses presented in this report have confirmed seismic deficiencies at both existing dams (BC 1 and BC 2). The estimated deformation of each dam in response to potential earthquakes suggests a high potential for significant damage and/or failure to occur.

Two methods of evaluation have been used to assess potential deformations including 1) the development of a numerical model based on an industry accepted “Newmark” analysis methodology, and 2) an empirical correlation between seismic loading and observed deformations at a variety of existing dam sites (i.e. case history data) The estimated crest deformations for both dams based on these methods were reasonably similar. The numerical evaluation method results reflect the more rigorous approach and predict larger potential deformations consistent with the unusually long duration of ground shaking that would be associated with a Cascadia earthquake event.

The selection of an appropriate earthquake loading conditions for dam safety evaluations and design represents a critical aspect of the study. The Cascadia Subduction Zone (CSZ) hazard is substantial (Richter Magnitude 9) and the understanding of this magnitude of event, and the corresponding peak ground accelerations, and duration of strong shaking that would result at the Newport dam sites is continuing to evolve throughout the industry. Based on the current standard of practice at both the state and federal levels of jurisdiction in the northwest, ground motions with expected recurrence intervals of up to 4975-years have been used as the basis of our assessment and design presented in this report.

Alternatives for Corrective Actions

Based on the outcome of the stability analysis and evaluation, HDR developed three different alternatives to provide a solution for both dams that would provide adequate dam safety and for a continuous drinking water supply following a significant earthquake event. The repairs for BC 1 would be very costly for the gained benefit as the dam does not hold enough water to pay off the costs of its remediation. A decision was made together with the City to not proceed with any corrective actions for BC 1.

Alternative 1 consists of a raise of BC 2 to include the current water storage from BC 1, recovery of storage in the upper reservoir due to sediment accumulation, and increased storage for future water demands in the city. This alternative presents some challenges as the existing reservoir and outlet works would need to stay operational during construction. The foundation excavation volume for this alternative is very large and sufficient construction material would have to be found to replace the excavated foundation material as well as the new embankment section. Because of the potential for significant deformations of the upstream slope of the dam, a new outlet structure would have to be built through the right abutment of the existing dam. Further, a spillway and fish ladder would need to be constructed. This alternative is doable but does not present the most cost effective and most feasible option.

Alternative 2 consists of a new roller compacted concrete (RCC) dam at a location just downstream of BC 2 where the topography of the valley narrows the most.

Alternative 3 consist of a new embankment (earthen) dam at the same location as Alternative 2.

Both alternatives 2 and 3 are acceptable solutions for corrective actions and represent a “least cost” solution for the project purposes outlined above.

Decision Level Estimates of Probable Costs

Decision level cost estimates were developed for Alternatives 2 and 3. At this time, the costs exclude some important project elements as the extent and dimensions of those elements is unknown at this stage of the project. They also include some significant cost uncertainties and hence are not suitable for establishing project funding. Future preliminary design will be required to provide the basis for a funding level cost estimate. The Preliminary design should include such elements as the spillway for Alternative 3, fish ladder, access road, and pipeline to the water treatment plant.

From a decision making standpoint, the cost estimates show that both Alternatives are similar and that a decision on the preferred dam type and configuration can be based on a number of other considerations such as long term operation and maintenance, owner preference and cost risk uncertainties.. Based on discussions with the City, Alternative 2 is recommended for preliminary design. Should a significant issue be identified with this Alternative during the early stages of preliminary design, Alternative 2 can be pursued as the preferred configuration.

Conclusions and Recommendations

Alternative 2 (RCC dam) provides a number of potential advantages to the City such as a relatively short construction timeline, proven seismic performance of concrete dams, lower cost uncertainty, smaller project impact footprint, and preferred spillway configuration

HDR recommends moving forward with a preliminary design of Alternative 2 (RCC dam). The preliminary design will include both geophysical, and boring characterization of the proposed site, a budget level cost estimate, environmental permit preparation, access road refinement, and additional modeling which is required by the state.

1 Introduction

HDR began working with the City of Newport in 2009 on the design and construction of a new water membrane filtration treatment plant. The water treatment plant is supplied with water stored in two man-made reservoirs in Big Creek, denoted Big Creek Dam No. 1 (BC 1) and Big Creek Dam No. 2 (BC 2). BC 1 reservoir is adjacent to the new treatment plant, and BC 2 reservoir is located approximately 1 mile upstream. These reservoirs were formed by the construction of an earthen dam at each location.

During construction of the new plant, geotechnical explorations were performed for the design of a new intake structure located in the BC 1 reservoir. A single boring drilled in October 2011 by Foundation Engineering, Inc. (FEI) showed foundation material to generally consist of very soft to soft clayey silt and very loose to loose silty sands. The initial boring and engineering evaluation also identified that the loose silty sand soils have a potential for liquefaction during a seismic event and that further dam safety related evaluations were indicated.

BC 1 is 315 feet long with a maximum height of 21 feet. The reservoir normally impounds 190 acre-feet of pool. The dam was designed by CH2M of Corvallis, Oregon and constructed by the City of Newport Public Works Department in 1951. Available design drawings depict the dam as a homogeneous compacted clay dam with embankment slopes of 1 vertical (V) on 3 horizontal (H) upstream and 1V on 2H downstream. Drawings show a 5-foot-thick granular drainage zone at the foundation level of the downstream third of embankment.

BC 2 was originally constructed in 1969 and modified and raised in 1975 and 1976. The dam was to be raised by 17 feet to an overall height of 56 feet and a length of 450 feet. The dam is shown with a central core trench and a downstream drainage system. Foundation materials are described as medium to stiff sandy silts over a weak siltstone. The CH2M-Hill, (CH2M-Hill, Predesign Report for the Raising of Big Creek Dam No. 2, City of Newport, Oregon, 4 Sep 1974), states that a seismic coefficient of 0.1 g was used for a pseudo-static analysis and a bedrock acceleration of 0.18 for a Newmark analysis which was used to estimate potential displacement during a seismic event.

1.1 Project Background

As a result of the potential dam safety-related concerns identified in the initial boring at the site, the City requested HDR perform a seismic evaluation of the embankment dams for both BC 1 and BC 2 reservoirs. This evaluation was completed in 2011 and 2012 and consisted of site investigations to characterize the dams' earthen and foundation materials, a probabilistic seismic hazard analysis (PSHA), a geologic hazard assessment, and geotechnical analyses to determine the stability of the dams in the event of potential seismic events. The initial site investigation and characterization program consisted of borings, cone penetration testing, seismic refraction geophysical testing, and laboratory testing.

1.2 Previous Report and Results

In February 2013, HDR submitted the “Big Creek Dam No. 1 and No. 2 Preliminary Geotechnical Investigation and Seismic Evaluation” report (February 2013 Report). This is subsequently referred to as the Phase 2 investigation program. The report described the site characterization program, the soils testing program, an evaluation of the results, and the engineering analysis for the two dams. The report included regional and site geology, seismic hazards, preliminary models of subsurface conditions, results of the seepage and stability analysis, and recommendations for the two dams.

The recommendations included the following:

- The seismic safety of BC 1 was estimated to be marginal while a significant safety deficiency was identified at BC 2.
- Additional site characterizations were recommended in order to further refine stratigraphic models of the existing structures, confirm the mineralogical origin of the soils and the corresponding reasons for the low densities, further refine the engineering properties and behavior of the foundation and embankment soils, and reduce uncertainties that occurred with the limited data sampling conducted. The additional data would also be used to support alternative design concepts.
- An update of the time histories was necessary as the U.S. Geological Survey (USGS) guidelines and regulations had changed due to the available research data from the most recent Chile and Japan subduction zone earthquakes. This was necessary to create alternatives that comply with the most recent safety standards and available design criteria.
- Additional laboratory testing was recommended to further examine the soil characteristics of the additional site explorations and refine the soil properties.
- Further engineering analyses were recommended to include the newly analyzed data and use it for computer models to simulate the behavior of the dams in case of a seismic event.
- Based on the findings of the additional analysis, corrective actions would be developed to mitigate the stability problems of the two dams. A range of rehabilitation concepts and methods was recommended for the next phase of the project.

The results presented in this report have subsequently been described as the Phase 2 investigation program.

1.3 Scope of Current Phase

Beginning in July 2014, HDR performed additional (Phase 3) site characterization and further engineering evaluations including concept design/alternative evaluations to reduce the risk of a dam failure for BC 1 and BC 2 in case of a seismic event. The original Phase 3 scope for the project included: additional site explorations, sampling and laboratory testing at both the BC 1 and BC 2 sites; updating the seismic hazard characterization of the site; developing site hydrology that would be used to assess spillway requirements for modified dam configurations; establishing analysis parameters through integrated evaluation of both the field and laboratory test data; updating the previously completed seepage, static and post-earthquake stability analyses; evaluating new seismic response with Newmark Sliding (Rigid) Block analysis based on a more comprehensive geologic model of the site; and developing and evaluating alternatives for corrective actions at both BC 1 and BC 2.

HDR performed initial engineering analysis for existing conditions and for alternative configurations involving corrective actions to mitigate the seismic stability problem for both dams in order to develop opinions on the preferred configuration of corrective actions. During the progress of the work, based on input from the City, HDR modified the approach of the corrective action alternatives to include three potential configurations at or near the BC 2 site that each included the following components of water storage along with remediation of dam safety deficiencies:

Upper Reservoir Storage:	970 acre-feet
Lower Reservoir Storage transfer:	200 acre-feet
Upper Reservoir Sediment Recovery:	100 acre-feet
Future Storage Allowance:	1,000 acre-feet
Total Storage:	**2,270 acre-feet**

The original scope of work also included a risk-based assessment to establish the appropriate level of seismic loading to be included in the design, a review of environmental conditions and clearances that would be needed, consultation with the City Engineer and the State Engineer at the Oregon Water Resources Department for dam safety, and preparation of appropriate reports and decision documents.

As a result of the revised storage and configuration requirements for the project described above the risk-based assessment to establish the appropriate seismic design criteria was removed and a preliminary design criteria of a 4,750-year seismic event was used to configure the alternatives. In addition, the scope of engineering analyses was modified in order to complete the engineering analyses within existing budget limits. The approach to engineering analyses was made in order to include evaluation of the concrete dam alternative by: 1) using a Newmark deformation analysis in lieu of a FLAC analysis for the embankment alternatives, and 2) performing a response spectrum evaluation of the concrete dam configuration.

1.4 Project Team

The Project team for the Phase 2 studies presented in this report included HDR as the principal engineer, with support from Cornforth Consultants (Cornforth), the Geotechnical Earthquake Engineering Department of the University of California, Davis (UC Davis), and Marine + Earth Geosciences (MEG).

Cornforth completed the update to the seismic hazards to the most current USGS standards and also supported the field explorations and index property laboratory testing for the samples.

UC Davis provided support to develop the laboratory testing plan and interpretation of field and laboratory testing data based on their research experience.

MEG provided the laboratory testing for all undisturbed samples.

HDR developed and directed the field and laboratory testing program, provided geologic models of the existing dams along with the engineering evaluation of the dams. Based on the outcome of the engineering analysis, HDR developed concept designs for the Alternatives described in this report along with decision level cost estimates. Three alternatives to mitigate the seismic hazard were identified. HDR also provided a preliminary review of project hydrology, and environmental review which entails a list of the necessary environmental permits associated with the proposed alternatives.

Key HDR personnel for this project included the following:

Verena Winter, P.E.	Project Manager
Keith A. Ferguson, P.E.	Principal Engineer
Scott Anderson, P.E.	Senior Geotechnical Engineer
John Charlton, P.G.	Senior Engineering Geologist
Andrew Little, EIT	Project Engineer
Michael Woodward, EIT	Project Engineer
Richard Hannan, P.E.	Technical Review
Farzad Abedzadeh, PE, PhD	Senior Dam Structural Analyst

2 Phase 3 Site Characterization and Evaluation Results

Additional site characterizations and evaluations were performed during Phase 3 and are summarized below.

2.1 Seismic Hazards and Time Histories

A seismic hazard update in support of this phase was performed based on information from recent large subduction zone earthquakes and newly released probabilistic seismic hazard maps as well as the newly released updated regional seismicity and potential ground motions from USGS's 2014 Probabilistic National Seismic Hazard Maps (NSHM) and supporting documentation. The newer information was compared to the results of the February 2013 report and Cornforth provided additional seismic hazard information and acceleration time history parameters for the site evaluation. The revised seismic hazard analyses and updated information are provided in Appendix A.

2.2 Site Explorations

Subsequent to the initial boring completed at the BC 1 site, field investigations to characterize the site subsurface conditions have occurred during two additional phases. The initial boring at BC 1 occurred in 2010 when the problem was discovered. The results of that boring were included in the previous report from February 2013. The second phase of explorations occurred in December 2011 through January 2012. These investigations consisted of mud rotary and hollow stem auger drilling, cone penetrometer testing, and a surface geophysical survey. The results of Phase 2 were included in the report from February 2013 as well. The third phase of investigations occurred in November/December 2013 and is described in this report. This Phase 3 program consisted of mud rotary drillings and cone penetrometer testing, disturbed and undisturbed sampling, and laboratory testing. A detailed discussion of the Phase 3 program of field investigations is presented in Appendix B.

2.2.1 Boreholes and Cone Penetration Testing Results

The 2013 investigations consisted of additional borings, and cone penetration testing at the BC 1 and BC 2 sites. The drilling work was performed by Western States Drilling and the cone testing was done by Northwest Geophysical Associates, Inc. as a subcontractor to Western States. The borings and cone soundings were necessary to better define the stratigraphy at the site including a better definition of the top of rock, and to collect disturbed and undisturbed soil and rock samples. Continuous Standard Penetration Testing (SPT) was performed in all bore holes. In addition to the SPT data, the procedure also allowed for the collection of disturbed soil samples. Further, undisturbed samples were obtained with 3-inch-diameter thin-walled Shelby tube samples at selected depths in the borings using a fixed piston sampler. The disturbed and undisturbed samples were needed for the second phase of laboratory testing.

The subsurface materials encountered in the BC 1 exploratory bore holes generally consisted of approximately 60 feet of silty sand, clayey silt, and silty clay alluvium overlying Nye Mudstone. The subsurface materials encountered in the BC 2 exploratory bore holes generally consisted of approximately 10 to 15 feet of silty sand and clayey silt alluvium, overlying approximately 30 to 35 feet of silty sand, clayey silt, and silty clay alluvium/colluvium, overlying Nye Mudstone.

Two Seismic Cone Penetration Test (SCPTu) soundings with pore pressure measurements were advanced at the BC 1 site and four were advanced at the BC 2 site. The two SCPTs at BC 1 and two SCPTs at BC 2 were advanced near existing borings to provide a comparison between the SCPT data and SPT data. The SCPT tip resistance, sleeve friction, and pore water pressure was measured at 2-inch increments as the SCPT instrument was pushed at a constant rate of 2 centimeters/second. Shear wave velocity and pore water pressure dissipation measurements were conducted at selected depths at all locations. Each of the four SCPTu explorations at BC 2 showed lower permeabilities at the upper elevations and slightly higher permeability with depth. All SCPTs were terminated at refusal. SCPT data is presented in Appendix B.

2.2.2 Laboratory Testing Results

Laboratory testing of soil samples collected from the 2013 site exploration were taken to MEG in Vancouver, British Columbia and, in conjunction with guidance from Dr. Jason DeJong at the University of California at Davis and HDR, a laboratory test program was developed.

The laboratory testing program was developed using Stress History and Normalized Soil Engineering Properties (SHANSEP) framework, which accounts for the stress history and the anisotropy of the soils due to different modes of shearing that are encountered during slope stability analysis. The three modes are triaxial extension near the toe of the slip surface, triaxial compression at the head of the slip surface, and direct simple shear along the base and transitions of the slip surface.

Radiography (x-ray) of the undisturbed samples was performed to evaluate the suitability of the samples for testing and develop a testing plan for the range of samples taken during the exploration. Consolidation testing consisting of load-increment ratio (LIR) and constant strain rate (CSR) consolidation methods were used to evaluate the sample disturbance and stress history profile with depth. Selected samples were then evaluated in shear by direct simple shear (DSS), isotropically consolidated triaxial compression (CIUC) testing. The SHANSEP method assumes that the behavior of the soil can be represented by the undrained shear strength, S_u, divided (normalized) by the effective overburden pressure, σ'_{v0}, with other parameters to take into account the overconsolidation ratio (OCR) and the shape of the curve, the exponent m. To evaluate the suitability of the SHANSEP framework to represent the behavior of the soil, samples were consolidated to three to four times the estimated pre-consolidation pressure identified in consolidation tests corresponding to an OCR of 1 (the soil is considered normally consolidated at this OCR). Several of the test samples were consolidated to three to four times the pre-consolidation stress and then unloaded to an overburden stress that corresponds to a known OCR, typically an OCR of approximately 4. The plots of these tests can be found in Figure D-1.5 in Appendix D. Individual test results are also found in this Appendix D. The result is a framework with which to evaluate the strength of the soil with depth and OCR.

Cyclic DSS (CycDSS) testing was performed to evaluate strength degradation with cyclic loading. Based on the CycDSS testing the soils appeared to have little to no strength degradation to 100 cycles and Post-CycDSS testing yielded soil strengths nearly the same as samples tested in static DSS. A strength reduction was evaluated by using Figure D-1.8 in Appendix D and the average plasticity index from the soils encountered. A reduction of 20 percent was conservatively used to degrade the strength properties from the peak undrained strength to the post-earthquake undrained strength.

2.3 Engineering Parameters and Assessment

The parameters developed in the laboratory testing program and those calculated and estimated based on SCPTu were used for assessing the existing dams with respect to seismic loading. Permeability values were evaluated from SCPTu dissipation testing and laboratory consolidation testing results. A set of upper and lower bound permeability values were used in the seepage analysis and subsequent stability analysis of the dams. The upper and lower bound values did not result in significantly differing Factors of Safety (FOS) for stability.

Based on the laboratory testing program and the in-situ testing which was calibrated to the laboratory testing data, the slope stability models were updated to use the SHANSEP parameters for the alluvial soils in the foundation. A maximum OCR of 4 was used, neglecting the higher OCR values in some samples that were a result of desiccation and shear stress bias at the toes of the dam where samples were collected and SCPTu testing performed. Figure D-1.4 of Appendix D shows the variation of OCR with depth for the free field environment. The dams themselves increase the overburden stress of the foundation soils and thus reduce the OCR of the underlying soils.

Use of the Field Shear Vane (FSV) and SCPTu was complicated by the drainage conditions within the soils encountered. Intermediate types of soils were encountered exhibiting characteristics of both sand-like and clay-like soils. The drainage conditions complicated the interpretation of both the FSV and SCPTu tests; however the use of dissipation testing as part of the SCPTu soundings assisted in identifying the soils that may be experiencing some degree of drainage conditions during the cone penetration testing. This determination was one of the key Phase 3 exploration program findings and helped to limit the use of the parameters estimated from the in-situ testing. Based on the dissipation and laboratory testing, the SCPTu results were subsequently calibrated with the laboratory testing strengths. This allowed the SCPTu test to validate the SHANSEP framework and parameters. As a result, the Phase 3 program found that with the strength of the foundation materials remaining relatively constant across the entire depth of these materials with appropriate consideration of OCR and overburden pressures.

Results of the engineering parameters evaluation are described in more detail in Appendix D.

2.4 Seismic Deficiency Verification

Based on the Phase 3 exploration, laboratory testing and engineering analyses a significant seismic deficiency was verified at BC 1. Analysis results indicated that this dam would be expected to fail by settlement and overtopping under seismic loading for recurrence intervals of 2,475 and 4,975 years. More frequent events, such as the 475-

and 975-year would likely result in significant damage to the dam, outlet works, water supply pump station, and ability to operate the reservoir. The location and configuration of the critical potential failure surface at BC 1 is very deep, making remediation of the site very challenging and expensive. Given the small amount of storage in the reservoir and the very large anticipated remediation costs, rehabilitation of this dam is judged as non-feasible.

The upper dam, BC 2, also has unacceptable deformations (settlement) during the 2475- and 4,975-year recurrence interval seismic events and would also likely fail due to overtopping and/or seepage through transverse cracks that would develop under these loading conditions. Similar to BC 1, the dam would also likely experience significant damage during earthquakes with more frequent return periods. While the upstream slope for BC 2 may be buttressed by some sediment that has accumulated in the reservoir, analysis results indicate that deformations of the upstream slope of BC 2 would be significant for the larger seismic events resulting in damage or failure of the outlet works, intake structure, and discharge pipeline.

A comparison of the estimates of embankment dam deformations using the Newmark analysis numerical methodology presented in this report with case history data and estimated crest deformations using the empirical methodology from Swaisgood (2003) was made to verify results and conclusions. Using the Swaisgood methodology with the range of estimated peak ground accelerations at the Newport sites for different recurrence interval Cascadia earthquake events indicate that for similar embankment dam case histories in the data base, crest deformations ranged from as little as 1.2 inches for the 475-yr return period peak ground acceleration to over 478 inches for the 4,975-yr. return period peak ground accelerations.

Based on the performance of these similar dams, estimated deformations in the range of 24 to 60 inches have a moderate to high potential for very significant damage or failure. When deformations are estimated to be in this range for these recurrence interval earthquake events, the standard of care within the dam engineering community in the US and internationally would suggest that there is dam safety deficiency and justification to take action to mitigate that deficiency. Estimated deformations of over 60-inches have a high to very high likelihood of complete failure of the dam section and not only is there a deficiency, but justification to take more expedited actions to reduce the risk of failure of the dam.

Swaisgood's estimates of percent settlement are based on the combined thickness of the dam height and the thickness of the underlying loose and/or low density alluvial soils. It should be noted that the case histories only include data up to a PGA of approximately 0.71 g and that extrapolation was necessary to project the regression line to the levels of PGA anticipated for the 2,475 and 4,975-year return period events at the Newport sites. A summary of the estimated deformations from the Newmark analyses along with Swaisgood empirical methodology is provided in Table 1 below. Note that the table cells have been colored to represent the deficiency and action categories described above. The orange cells suggest the deficiency and moderate justification for corrective actions. The red cells suggest a deficiency and justification for more expedited corrective actions. The green cells indicate deformations that are below the level associated with a safety deficiency and need for corrective actions.

Results of engineering analyses and seismic deficiency verification evaluations are presented in more detail in Appendix D.

Table 1: Summary of Estimated Embankment Crest/Downstream Slope Deformations at BC-1 and BC-2

Recurrence Interval Event (years)	Estimated Peak Ground Acceleration (PGA – g's)	Est. Deformations - Empirical (Swaisgood, 2003) (inches)			Est. Deformations – Newmark (inches)		
		Lower Bound	Best Estimate	Upper Bound	Lower Bound	Best Estimate	Upper Bound
BC 1							
2475	0.79	15	33	68	50	>76	90
4975	1.12	218	478	>478	116	>160	184
BC 2							
2475	0.79	15	33	68	32	>48	54
4975	1.12	218	478	>478	56	>96	112

3 Alternatives for Corrective Actions

Based on the results of the Phase 3 explorations, laboratory analysis, and the related engineering assessment, it became apparent that rehabilitation of the lower reservoir, BC 1, is non-feasible from an economic standpoint. The location and depth of the critical potential failure surface through the foundation soil underneath the dam makes mitigation of BC 1 very expensive relative to the amount of storage that is in the reservoir. Consequently, based on discussions with the City, HDR evaluated alternatives to mitigate BC 1 by transferring its current storage capacity to the upstream BC 2 remediation alternatives.

3.1 Alternative Options

The decision to not include BC 1 in the corrective action scenario led to increased storage capacity requirements for BC 2. Additional storage for anticipated sedimentation in the reservoirs and for future storage was also included. Future storage was based on the population projection from the 2008 Water System Master Plan (Civil West Engineering Services, Inc.). The Water System Master Plan indicates a need for a 30 percent increase in water supply by 2030. Table 2 lists theoretical storage capacities for the current reservoirs and for the future solution. The maximum theoretical future storage capacity of 2,270 acre-feet (ac-ft) was used for the configuration level layouts and cost estimates for modifications to BC 2.

Table 2. Reservoir Storage Capacities

Description	Upper Reservoir Storage (ac-ft)	Lower Reservoir Storage (ac-ft)	Sediment Storage Allowance (ac-ft)*	Future Storage Allowance (ac-ft)**	Total Storage Allowance (ac-ft)***
Replace Existing Storage	970	200	100	0	1,270
Minimum Future Storage	970	200	100	380	1,650
Maximum Future Storage	970	200	100	1000	2,270

* Future storage allowance equals an increase of 30 percent of current storage capacities combined

** Indicates estimate of current and future sediment in upper reservoir to be recovered by increased reservoir storage

*** Future storage allowance to be based on approximate minimum and maximum estimates of drought and other supply needs over 20- to 50-year planning horizon. These numbers should be appropriate building blocks for an enlargement project Purpose and Need statement that can be approved under appropriate environmental compliance activity

The project team identified five different alternatives upstream of BC 1 to secure the drinking water source for the City. All alternatives were considered but only three remained feasible and underwent an analysis. All alternatives listed below are conceptual and would require further refinement during the next phase of the project.

Figure 1 shows the five different dam axis considered for the alternatives (All figures are located at the end of this report).

3.1.1 Alternative 1: Raising and Modifying the Existing Dam

Alternative 1 includes raising the existing upper dam (BC 2) to achieve the necessary seismic safety and storage capacity. The new crest of this embankment dam would be downstream of the existing crest as the existing reservoir and dam need to stay in operation during construction. The raised dam would be a continuation from the existing upstream slope at a new 3H:1V (Horizontal:Vertical) slope rising to a total dam height of 111 feet at elevation 131 feet. The new water surface elevation would be at elevation 116 feet for a normal water pool. The new crest would be 20 feet wide and the downstream 3:1 slope would extend into the valley downstream of the existing upper dam.

The dam would have an internal filter and drainage system. The foundation soil of the existing dam would remain in place and the foundation soil for the new portion of the dam would be excavated to bedrock and replaced with suitable compacted dam material.

A new outlet structure consisting of a multi-inlet sloping intake structure and a 36-inch discharge pipe installed in a new tunnel system in the right abutment of the dam and discharging through a control structure into a 20-inch diameter treatment plant pipeline, or 36-inch diameter dam safety discharge to the stream channel. The sloping intake structure would have different inlet ports for water quality purposes so water could be drawn from different elevations of the reservoir. The upstream portion of the outlet pipe would be routed through the right abutment of the dam in a micro-tunnel system creating a seal from the reservoir. This pipe would discharge into an outlet vault within the abutment near the dam axis centerline and then through a 10-foot-diameter access tunnel until it daylights at the control structure The spillway and fish ladder would be routed to the north side of the dam. Figure 2 includes details of this embankment alternative.

Advantages of this alternative include reasonably well-defined foundation geometry, the properties of the existing dam materials have been tested and are well understood, the footprint for the addition would be small compared to a new dam, and a cofferdam and dewatering requirements at the downstream side should not be excessive.

Disadvantages include the possibility that construction of a new outlet and spillway may require the existing dam be taken out of service for a period of time (which may cause water supply issues), only the downstream side of the dam is being seismically stabilized and there would still likely be significant damage to the upstream portion of the embankment during a significant seismic event, and the construction schedule for excavating and embankment construction would be limited due to the short construction season for embankment placement.

This alternative would have significant costs associated with construction of the new outlet works described above.

3.1.2 Alternative 2: New RCC Dam

Alternative 2 includes a new gravity dam structure constructed out of roller compacted concrete (RCC) downstream of the existing upper dam (BC 2) at a location where the valley narrows topographically and offers the possibility of a least cost dam project. The new dam would be located within the existing lower reservoir just downstream of the existing upper dam. This dam would have a height of about 100 feet with the crest at elevation 120 feet. The normal water surface elevation would be at 112 feet. The foundation soil would be excavated and the new dam placed on suitable bedrock. The spillway chute and stilling basin would be over the central portion of the dam. The vertical concrete intake tower would be integrated into the upstream face of the dam and would have intake ports at different levels so water can be drawn from different depths for water quality purposes. From the intake tower a 36 inch outlet pipe would be routed through the base of the dam until it daylights at a gate house and forks into the 20-inch raw water pipe which is connected to the water treatment plant, and into the spillway stilling basin to provide a low level dam safety outlet. Structural details would have to be defined at a later point in time but seismic modeling of the new dam showed the need for a conventional concrete shear key and upstream heal section to provide adequate resistance to cracking and sliding in case of the larger seismic events. The facing, spillway portion, stilling basin, and crest road of the dam would also be conventional concrete. Figure 3 includes details of this RCC alternative.

Advantages of this alternative include a more robust structure that is less susceptible to damage from seismic or hydrologic events, a smaller footprint requiring less excavation than a new embankment dam, smaller quantity of material required for the RCC dam, constructed of material that can generally be placed year around, the ability to incorporate the spillway and outlet work into the RCC structure, little maintenance needs, and this alternative that can be constructed while the existing upstream dam remains in operation.

Disadvantages include the location of the structure in the upstream end of the BC 1 pool that would require a cofferdam and increased dewatering efforts, and foundation conditions that have not been defined which may result in some increase in cost.

3.1.3 Alternative 3: New Embankment Dam

Alternative 3 consists of a new embankment structure at the same proposed location as Alternative 2 (RCC dam). The foundation soil would be excavated to bedrock and suitable embankment earthen material would be placed to construct the dam. The height of the dam would be about 108 feet with the dam crest at elevation 128 feet and a new normal water surface elevation of 112 feet. The downstream and upstream slopes of the dam would be 3H:1V. The dam would have an internal filter and drainage system. The outlet works would be placed in either the lower right or left abutment areas on bedrock and include a multi-port sloping intake structure connected to a concrete encased 36-inch-diameter steel outlet pipe through the dam foundation. The multiple intake ports would be placed for water quality purposes. The 36-inch outlet pipe would daylight at a gate house and fork into the 20-inch raw water pipe going to the water treatment plant, and into the 36-inch pipeline discharging to the stream channel for dam safety purposes.

The spillway channel and access road would be north of the proposed dam. Figure 4 includes details of this embankment alternative.

Advantages of this alternative are limited to the ability to continue operation of the upstream dam during construction, and a dam that is less susceptible to seismic and hydrologic events than the Alternative 1 structure.

Disadvantages include the much larger footprint than Alternatives 1 or 2, the geometry for the rock foundation is unknown, there would be a significant increase in the quantity of foundation excavation required compared to Alternative 2. In addition, the downstream cofferdam and foundation dewatering would be significantly larger than Alternative 2. The construction season for embankment placement would be limited and would take the longest to complete of all the alternatives under consideration. This alternative would have the largest risk exposure to floods and other adverse construction conditions of all alternatives under consideration.

3.1.4 Alternative 4: New Dam Option A

Alternative 4 was considered early in the project as a possible new site location for either an RCC or embankment dam. It was thought to be further downstream of the upper dam (BC 2) located in the lower reservoir about 100 yards downstream of proposed Alternatives 2 and 3. This alternative was eliminated from further consideration as the valley is wider at that particular location and the costs for the dam would be much higher than Alternatives 2 and 3 without providing any other benefits. Figure 1 shows the proposed location of this embankment alternative.

3.1.5 Alternative 5: New Dam Option B

Alternative 5 was similar to alternative 4 as it was considered early in the project as a possible new site location for either an RCC or embankment dam. The location was thought to be where the current access road crosses the lower reservoir as the valley narrows the most at that location. This alternative was not considered further as some of the land that the dam would cover does not belong to the City and is outside the city limits. Acquisition and condemnation of the properties and zoning changes did not seem advantageous in relation with providing a better option than Alternatives 1, 2, or 3. Figure 1 shows the proposed location of this dam alternative.

3.1.6 Alternative 6: No Action

Alternative 6 is the No Action alternative and is still an option that the City has to weigh against the possible risk of loosing the only drinking water source for the City in case of a seismic event.

3.2 Other Related Structures

All alternatives include other related structures that would have to be added to make the dam and water supply functional. The intake tower (for RCC dam alternative) or the sloping intake pipe (for embankment dam alternative) would be equipped with three different ports or gates at different elevations. The reservoir stratifies during the summer months and the lower portion of the lake becomes anaerobic and the upper portion becomes aerobic. This influences the water quality of the lake. Different elevated intake gates allow the treatment plant operators to draw water from different depths of the reservoir to avoid the undesired water during the summer. These gates would need the appropriate size of fish screens to avoid fish getting into the pipeline and therefore into the pumps of the treatment plant. The exact size of those screens would be determined during the next phase as it would depend on regulations and requirements for Oregon Department of Fish and Wildlife (ODFW) and other environmental factors.

All dams require a low level outlet for dam safety that acts as an emergency outlet in case the reservoir has to be drawn down rapidly. This outlet would be part of the outlet works for all alternatives and would be located at the downstream toe of the dam. This outlet would have a stilling basin structure at the end to avoid erosion when the water is being released. The RCC dam has a stilling basin at the toe of the spillway in addition to the dam safety outlet.

The embankment dam options would need a separate spillway as the spillway is not part of the actual dam structure as with the RCC dam alternative. This spillway would have to be refined at a later phase as well. The most likely location would be north of the proposed options around the dam running parallel to the access road.

A new fish ladder may have to be built for all alternatives. The exact requirements for sizing and design of the fish ladder would occur during the next phase of the project as it would depend on permit requirements and regulations by the ODFW. Currently, the location of the fish ladder is anticipated to be right next to the spillway for the embankment dams and to the north side near the access road for the RCC dam.

Presently, there is an access road leading from BC 1 to BC 2 and beyond. This road would have to be realigned as it would be blocked and/or flooded by any of the alternatives discussed. A potential new alignment is shown in Figure 1 but further investigation would be necessary during the next phase of the project.

A new raw water pipeline would have to be constructed starting at the outlets works for the dams and continuing to the existing intake pump station where it would tie into the existing pipeline just downstream of BC 1. Preliminary calculations size the pipe to be 20 inches diameter and constructed of ductile iron. The exact alignment would be determined during the next phase but would likely follow the road.

3.3 Comparison of Alternatives

Each alternative provides opportunities and constraints besides the costs of construction. Items that influence the decision making on an alternative are as follows: constructability, excavation volume, construction materials, foundation conditions, spillway design, intake structure, outlet works, necessary dewatering during construction, seismic and hydraulic resiliency of each dam alternative, environmental impacts and permits, operations and maintenance, and most importantly total costs, including geotechnical explorations, design, construction, permitting and contingency for unexpected events. Table 3 summarizes these items for the three preferred alternatives.

Table 3. Summary of Advantages and Disadvantages of Alternatives 1, 2, 3

Opportunity/ Constraint	Alternative 1 Raising Existing Dam	Alternative 2 New RCC Dam	Alternative 3 New Embankment Dam
Constructability	- Requires modifications to existing spillway - Requires temporary outlet works/coffer dam upstream to provide a continuous, uninterrupted water source during construction - Construction season for an embankment-type dam is limited to summer and early fall. - Source of construction materials for the dam have not been identified and may require a significant distance and processing requirements	- Existing reservoir can be in continuous operation - Downstream cofferdam required - Year-round construction possible - Requires construction of a temporary pipeline from the existing dam outlet to the new outlet during construction - Shortest construction prior and smallest construction risk exposure timeframe of all alternatives.	- Existing reservoir can be in continuous operation - Requires construction of a temporary pipeline from the existing dam outlet to the new outlet during construction - Significant increase in required project footprint - Much larger downstream cofferdam required - Construction season for an embankment type dam is limited to summer and early fall
Excavation Volume	- Moderate foundation excavation required at downstream toe	- Smallest foundation excavation required for dam foundation	- Large foundation excavation required for dam foundation; Several times greater than Alternatives 1 and 2
Construction Material	- Need for large amount of suitable foundation and dam material - Would require an off-site source for filter and drainage materials to be used in the dam	- Need for an appropriate off-site source of aggregate for concrete production	- Need for large amount of suitable foundation and dam material - Would require an off-site source for filter and drainage materials to be used in the dam.
Foundation Conditions	- Foundation conditions reasonably well-defined	- Foundation conditions unknown, and could impact final cost of alternative	- Foundation conditions unknown, and could impact final cost of the alternative
Spillway Design	- New spillway would be constructed into abutment with no stilling basin. Potential for significant erosion damage, if used	- Spillway and Emergency spillway co-located in center of dam with stilling basin. Limited potential for significant erosion and downstream channel degradation.	- New spillway would be constructed into upper right abutment which requires more excavation and cost increase once the design is in place
Intake Structure	- Sloping intake on upstream face of dam, requires lowering the water level significantly which would propose a problem to the continuous water supply - Intake pipe routed through the dam via tunnel in lower right abutment - Sloping intake difficult to operate and maintain	- Intake tower included in dam structure with limited footprint - Intake pipe would be short through the narrow dam compared to Alternatives 1 and 3 - Limited susceptibility to seismic damage	- Sloping intake on upstream face of dam - Intake pipe routed through the dam via tunnel - Sloping intake difficult to operate and maintain

Table 3. Summary of Advantages and Disadvantages of Alternatives 1, 2, 3

Opportunity/ Constraint	Alternative 1 Raising Existing Dam	Alternative 2 New RCC Dam	Alternative 3 New Embankment Dam
Outlet works	- Outlet as a combination of the water supply line to the treatment plant and the dam safety outlet.	- Outlet as a combination of the water supply line to the treatment plant and the dam safety outlet.	- Outlet as a combination of the water supply line to the treatment plant and the dam safety outlet.
Dewatering	- Small downstream cofferdam required for dewatering of area covering the new footprint - Moderate dewatering effort	- Significant downstream cofferdam required (dam located in upper part of reservoir BC 1) - Significant quantity of dewatering may be required	- Cofferdam much larger than Alternative 2 (downstream toe of dam located further downstream in reservoir of BC 1) - Dewatering quantity likely significantly greater than Alternative 2
Seismic Resiliency	- Limited damage due to seismic shaking still probable - Upstream portion of dam still susceptible to significant damage	- Low probability of significant damage resulting from seismic shaking	- Moderate potential for damage resulting from seismic shaking
Hydraulic Resiliency	- Potential for erosion damage during design flow	- Reduced potential for erosion during design flow	- Potential for erosion during design flow similar to Alternative 1
Environmental impacts	- Increase in inundation area - Extensive permitting process - Requires smallest footprint of the three alternatives	- Increase in inundation area - Extensive permitting process - Moderate interruption of existing lower reservoir due to footprint of new dam	- Increase in inundation area - Extensive permitting process - Significant interruption of existing lower reservoir due to footprint of new dam
Maintenance	- Requires annual maintenance to manage vegetation, burrowing animals, erosion, and other potential damage - Maintenance cost similar to Alternative 3	- Structure very resistant to damage and deterioration - Least cost maintenance	- Requires annual maintenance to manage vegetation, burrowing animals, erosion, and other potential damage - Maintenance cost similar to Alternative 1
Total costs	- Most costly due to new outlet works requirement	- Similar to Alternative 3	- Similar to Alternative 2

4 Preliminary Environmental Review

Each alternative would require permits from federal, state, and local agencies. Although the alternatives differ, the necessary work for each alternative would require the same permits and approvals as described in detail in Appendix C. Therefore, the preliminary environmental review does not differentiate permit requirements between alternatives. At this point it is difficult to gauge if one alternative would be more challenging to permit than another. To date, no agencies have been contacted to discuss the project in detail. This section provides an overview of anticipated permitting efforts.

4.1 Major Permits and Timelines

There are several major permits required for this project. Those permits and timelines are described in Table 4. Other permits aside from those listed in this table may be applicable but are not anticipated to be as complicated.

Table 4. Overview of Major Permits and Timelines

Required Permit	Timeline	Submittal Occurs at Engineering Design Level (approximate)
National Environmental Policy Act (NEPA)	12-18 months	15-30%
Clean Water Act Section 404/401 and Oregon Removal-Fill permit Other permits processed concurrently with applications: • Endangered Species Act Section 7 • Magnuson Stevens Fishery Conservation and Management Act (Magnuson Stevens Act) • National Historic Preservation Act (NHPA), Section 106 • Migratory Bird Treaty Act • Oregon Fish Passage • Coastal Zone Management Act	6-18 months	30%
Bald and Golden Eagle Protection Act (if required)	4-6 months	30%
Oregon Water Rights	9-12 months	30%
Clean Water Act Section 402 National Pollutant Discharge Elimination System (NPDES) 1200-C	60 days	100%
City of Newport Conditional Use Permit	30 days	60%
City of Newport Building, Electrical, Plumbing, Mechanical, Sewer/Water Permit	30 days	100%
Oregon State Engineer Design Review and Approval	2 months	100%

4.2 Additional Studies and Potential Costs

The project schedule can be influenced by the permitting process due to approval timelines for certain permits and the potential for unanticipated conditions that may arise and delay the permitting process. This can also delay design as well as construction and increase overall project costs.

Risks associated with complex permitting and stringent permit terms and conditions can result from lack of advance knowledge of the potential impact to sensitive environmental resources or public controversy. Early coordination with the agencies and identification of necessary environmental studies upfront would minimize the risk for permitting process delays. Anticipated environmental studies include completing a cultural resource evaluation and wetland and waters delineation, developing mitigation plans, updating the Emergency Action Plan, and preparing a biological assessment.

Depending on the nature of the project, permitting costs can range from 1 to 6 percent of the overall construction costs.

5 Decision Level Estimates of Probable Costs

The three alternatives presented in Section 3 of this report were further investigated in terms of costs for comparison of feasibility between the three alternatives. The cost estimates were prepared for the purpose of comparing alternatives and not for budgeting purposes. Budgetary costs would be provided during the next phase of the project as part of the preliminary design. These costs would include input from contractor estimating methods for the key units and lump sum items as well as further evaluation of construction material sources and costs.

A number of important budget items are not included in this estimate. The costs for those items would have to be added onto the total costs during the next phase of the project. These items would not make a difference in the outcome of the estimates for comparison purposes between the alternatives as they are similar for each alternative. The items purposely left out include: fish ladder, spillway (for embankment option, spillway is included in the RCC dam), access road to the dam, access road around the reservoir to provide access to the forest land and private properties, and the pipeline from the dam to the water treatment plant. Table 5 summaries the items not included in the cost estimate and the reasoning for exclusion.

Table 5. Excluded Items from Cost Estimate

Excluded Item	Alt 2 – RCC Dam	Alt 3 – Embankment Dam
Spillway	n/a spillway included	Exact alignment of spillway is unknown due to lack of survey and geotechnical information of the area
Fish ladder	Type and requirements of fish ladder are unknown at this point. Environmental assessment is necessary to determine the requirements and size for the fish ladder. It is not possible to set a number to this line item.	
Access Road to Dam	Exact alignment of access road is unknown due to lack of survey and geotechnical information of the area.	
Access Road Around Reservoir	Exact alignment of road unknown due to lack of survey in this area.	
Pipeline to Water Treatment Plant	Exact alignment is unknown due to several options for routing of this pipe and unknown access road alignment.	

5.1 Costs Estimate for Alternative 1 – Upper Dam Embankment Raise

Based on discussions with the City, a cost estimate for Alternative 1 was not completed and has been deferred to be updated at a later date if appropriate and necessary. The reasons for this include: the difficulty with constructability and keeping a continuous drinking water source during construction which makes this alternative less favorable; due to the upstream slope deformation concerns of this dam in a seismic event, replacing the outlet works presents a significant risk to the functionality of the system;

and during the last annual dam inspection in spring of 2015, the State Engineer observed some seepage distress in the pipe inside the dam of the current outlet works. These present concern of the overall stability of the existing dam. Experience on other similar projects suggests that the costs for a new outlet works for Alternative 1 are estimated to be disproportionately higher than for Alternatives 2 and 3 and would make this alternative the most expensive by a relatively wide margin.

5.2 Costs Estimate for Alternative 2 – RCC Dam

A planning level cost estimate for comparison purposes was prepared for Alternative 2 RCC Dam. The estimate includes site preparation, work associated with the dam and other structures associated with the dam (spillway and outlet works) and appropriate cost contingencies for a) design elements not included in the current layout b) permitting, c) engineering during construction, and d) a construction change order/claim contingency percentage. HDR developed a concept design as described in section 3.1.2 for the RCC alternative shown in Figure 3. Based on that concept design, quantities were estimated for each line item and an approximate cost calculated. Table 6 presents a summary of the costs providing a range of costs from a lower bound unit cost to an upper bound unit cost. The items listed in Table 5 were excluded in this cost estimate and need to be added to the construction cost estimate for the next phase. The decision level cost estimate for the RCC dam alternative ranges from $13.7 to $19 million. This number includes the spillway for the dam as an RCC dam has the spillway embedded in the structure.

Table 6. Planning Level Cost Estimate - RCC Dam Alternative 2

Bid Item	Description	Quantity	Unit	Lower Bound Unit Cost	Upper Bound Unit Price	Lower Bound Cost	Upper Bound Cost
Prep Work						**$ 306,225**	**$ 400,257**
1	Clearing and grubbing, stripping topsoil, reclamation of disturbed areas	1.4	Acre	$ 20,000	$ 26,000	$ 28,000	$ 36,400
2	Flood control coffer dam downstream	4,329	CY	$ 25	$ 33	$ 108,225	$ 142,857
3	Temporary pipe from existing dam to downstream of new dam	1,000	LF	$ 170	$ 221	$ 170,000	$ 221,000
Main Dam						**$ 7,853,000**	**$ 10,207,600**
4	Excavation - Foundation General	30,000	CY	$ 8	$ 10	$ 240,000	$ 300,000
5	Embankment - Backfill	15,000	CY	$6	$ 8	$ 90,000	$ 120,000
6	Fill - Roller Compacted Concrete	32,200	CY	$ 80	$ 104	$ 2,576,000	$ 3,348,800
7	Conventional Concrete Reinforced	1,000	CY	$ 750	$ 975	$ 750,000	$ 975,000
8	Conventional Concrete Non-Reinforced	12,100	CY	$ 325	$ 423	$ 3,932,500	$ 5,118,300
9	Construction De-watering	1	LS	$ 125,000	$ 162,500	$ 125,000	$ 162,500
10	Foundation Treatment - Grout Curtain	3,000	LF	$ 16.50	$ 21	$ 49,500	$ 63,000
11	Outlet Works Gates - Slide (Fabrication and Construction)	7,500	LB	$ 12	$ 16	$ 90,000	$ 120,000
Other						**$ 175,000**	**$ 228,600**
12	Intake structure and outlet works	1	EA	$ 100,000	$ 130,000	$ 100,000	$ 130,000
13	fishscreen for intake structure	2,500	LS	$ 12	$ 16	$ 30,000	$ 40,000
14	pipeline thru dam 36"	200	LF	$ 225	$ 293	$ 45,000	$ 58,600
Total Base Construction Cost (BCC)						**$ 8,334,225**	**$ 10,836,457**
15	Design Contingency			25.0%	30.0%	$ 2,083,556	$ 3,250,937
16	Mobilization/Demobilization construction			5.0%	5.0%	$ 416,711	$ 541,823
17	Construction, CO/C Contingency			8.0%	10.0%	$ 666,738	$ 1,083,646
Total Construction Cost						**$ 11,501,231**	**$ 15,712,863**
18	Permitting			3.0%	3.0%	$ 345,037	$ 471,386
19	Design and Site Characterization			7.0%	8.0%	$ 805,086	$ 1,257,029
20	Engineering Support during Construction			9.0%	10.0%	$ 1,035,111	$ 1,571,286
Total Cost (Rounded)						**$ 13,700,000**	**$ 19,000,000**

5.3 Costs Estimate for Alternative 3 – Embankment Dam

A planning level cost estimate for comparison purposes was prepared for Alternative 3 Embankment Dam. As for Alternative 2, the estimate includes site preparation, work associated with the dam, other structures associated with the dam, and appropriate contingencies for a) design costs, b) permitting, c) engineering during construction, and d) a construction change order/claim contingency. HDR developed a concept design as described in section 3.1.3 for the Embankment Alternative shown in Figure 4. Based on that concept design, quantities were determined for each line item and an approximate cost was calculated. Table 7 presents a summary of the costs providing a range of costs. The items listed in Table 5 were excluded in this cost estimate and need to be added to the construction cost estimate for the next phase. The option Embankment dam alternative ranges from $12.9 to $17.8 million. These numbers does not include the spillway for the dam as the spillway is a separate structure for embankment dams.

Table 7. Planning Level Cost Estimate - Embankment Dam Alternative 3

Bid Item	Description	Quantity	Unit	Lower Bound Unit Cost	Upper Bound Unit Price	Lower Bound Cost	Upper Bound Cost
Prep Work						**$ 396,225**	**$ 517,257**
1	Clearing and grubbing, stripping topsoil, reclamation of disturbed areas	5.9	Acre	$20,000	$26,000	$ 118,000	$ 153,400
2	Flood Control coffer dam downstream	4,329	CY	$25	$33	4 108,225	$ 142,857
3	Temporary pipe from existing dam to downstream of new dam	1,000	LF	$170	$221	$ 170,000	$ 221,000
Main Dam						**$ 7,085,140**	**$ 9,161,560**
4	Excavation - Foundation General	124,280	CY	$13	$17	$ 1,615,640	$ 2,112,760
5	Embankment Fill	301,000	CY	$14	$18	$ 4,214,000	$ 5,418,000
6	Embankment Filter Material	15,000	CY	$30	$39	$ 450,000	$ 585,000
7	Construction De-watering	1	LS	$480,000	$624,000	$ 480,000	$ 624,000
8	Foundation Treatment - Grout Curtain	3,000	LF	$17	$21	$ 49,500	$ 63,000
9	Riprap and Bedding	4,200	CY	$30	$39	$ 126,000	$ 163,800
10	Conventional Reinforces Concrete	200	CY	$750	$975	$ 150,000	$ 195,000
Other						**$ 362,500**	**$ 472,600**
11	intake structure and outlet works	1	EA	$175,000	$227,500	$ 175,000	$ 227,500
12	Fish screen for intake structure	2,500	LS	$12	$16	$ 30,000	$ 40,000
13	pipeline thru dam 36"	700	LF	$225	$293	$ 157,500	$ 205,100
Total Base Construction Cost (BCC)						**$ 7,843,865**	**$ 10,151,417**
20	Design Contingency			25.0%	30.0%	$ 1,960,966	$ 3,045,425
21	Mob/Demob construction			5.0%	5.0%	$ 392,193	$ 507,571
22	Construction. CO/C Contingency			8.0%	10.0%	$ 627,509	$ 1,015,142
Total Construction Cost						**$ 10,824,534**	**$ 14,719,555**
23	Permitting			3.0%	3.0%	$ 324,736	$ 441,587
24	Design and Site Characterization			7.0%	8.0%	$ 757,717	$ 1,177,564
25	Engineering Support During Construction			9.0%	10.0%	$ 974,208	$ 1,471,955
Total Cost (Rounded)						**$ 12,900,000**	**$ 17,800,000**

5.4 Comparison Costs Estimates for Alternative 2 & 3

As previously stated, the two cost estimates where prepared for comparing alternatives and assisting in the identification of the preferred alternative to move forward. From a decision making standpoint, the costs for Alternatives 2 and 3 are similar. It should be noted that the RCC dam cost estimate includes the spillway, but the embankment dam does not. The preferred alternative decision needs to be based on advantages and disadvantages of the alternatives presented in Table 3.

Based on the cost estimates, advantages/disadvantages, and overall experience of HDR, we recommend that Alternative 2 be selected for preliminary design. Alternative 3 can be further considered should any future investigations of the site indicate a significant challenge or cost increase to Alternative 2.

6 Conclusions and Recommendations

Phase 3 explorations and engineering analyses have confirmed significant seismic deficiencies with both BC 1 and BC 2 dams. Configuration level analyses and design layouts have provided important information about alternatives to remediate the seismic deficiencies of the Big Creek dams and how to move forward in the future in order to provide the City of Newport with a safe and reliable drinking water source after a seismic event.

6.1 Key Conclusions

Phase 3 of site characterization work provided the basis to update the site model and analysis, and increased the confidence in the findings of the study. The analysis indicated that both existing dams are unsafe due to excessive deformations that would occur during a large seismic event. Some form of remediation is needed to provide appropriate dam safety and water supply security for the City.

Based on the Phase 3 findings, the project purpose was modified to provide all current water storage capacity and an increased water supply meeting master planning requirements at the upper site. Decommissioning of the lower dam and reservoir (BC 1) would be required by the state. The storage from the BC 1 reservoir needs to be recovered. Also increased storage due to sediment accumulation and future water storage capacities needs to be provided with the new modifications.

Several alternatives have been identified that would meet the modified project purpose. The chosen alternatives to proceed include either a new RCC dam or embankment dam at a location immediately downstream of the upper dam (BC 2). Configuration level studies have indicated that both types of dam at this location can be designed and constructed to provide safe and secure water supply for earthquake events that have a minimum recurrence interval of about 5,000 years or higher. Such safety is consistent with state requirements and federal projects with similar potential consequences of dam failure.

6.2 Recommendations

The recommendation to move forward to provide the City with a safe and secure drinking water source is to build a new RCC dam (Alternative 2) at the location just downstream of the existing upper dam (BC 2). Based on the results of the current study, the RCC alternative would provide the most secure and stable option in case of a seismic event. Constructability of an RCC dam is less complicated and takes the least amount of time compared to the embankment option. The footprint of an RCC dam is less and provides fewer disturbances in terms of environmental impact compared to the embankment option. The preliminary costs show the RCC dam is a feasible option compared to the embankment dam.

Preliminary designs that include a comprehensive characterization of the new dam site are needed to update the configuration of the dam, to provide budgetary cost estimates, and to provide information required for permitting of the dam. Such preliminary design would be the objective of the next phase of work.

Information necessary for a preliminary design is geotechnical data of the new proposed site to provide the depth of bedrock and to characterize a foundation concept for the new dam.

The environmental permitting process can be started and prepared for the actual permitting process. A concept for the remediation of Big Creek can be developed at the location of the lower reservoir after the BC 1 dam has been removed. Dialog with ODFW should be started about fish ladder requirements and possible remediation opportunities.

A detailed budgetary cost estimate needs to be prepared that represents actual orders of magnitudes of costs. Based on this preliminary design cost estimate the search for funding and finance options can be explored.

Further, the access road to the dam and around the reservoir would be defined with the help of a comprehensive survey that has to take place to develop a preliminary design. The spillway for the embankment option has to be refined as well with the help of a topographic survey.

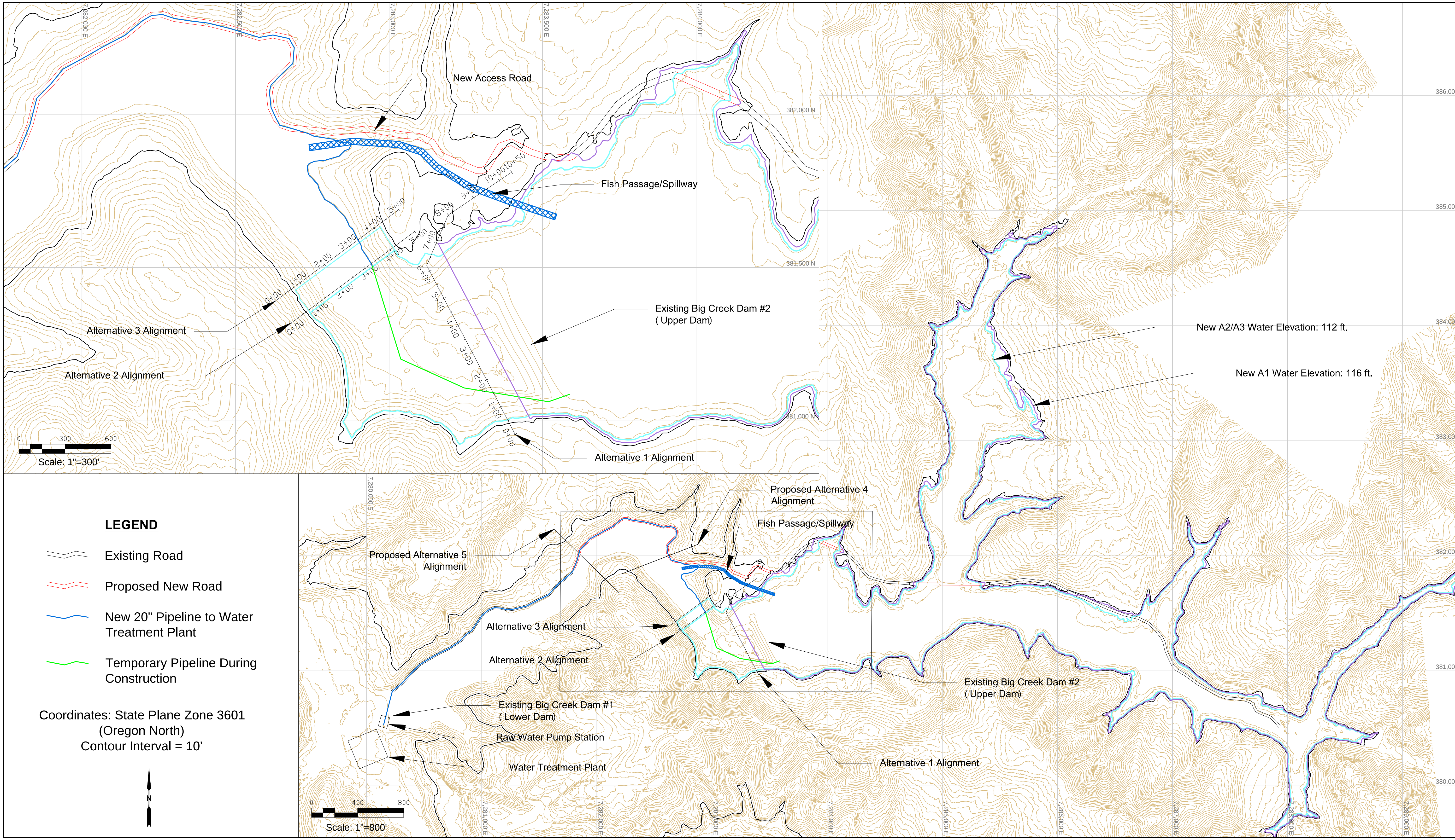
A schedule would need to be developed that presents the next steps of this project.

Some additional modeling analysis for the new dam is necessary during the preliminary design of the dam. This analysis would include two design earthquakes: the biggest crustal and the biggest fault earthquake. Both modeling results would have to be presented to the State to determine the design earthquake requirements for the new dam.

The consequences of a safety related failure of the dam needs to be updated to represent the culvert conditions where Big Creek flows underneath Highway 101 and then into the Ocean. It is likely this culvert would be blocked by debris or damaged in a seismic event. This scenario is not reflected in the current dam breach and inundation limits prepared for consequence evaluations and emergency planning in the Emergency Action Plan report. With the new dam arrangement, a new Emergency Action Plan would also need to be developed once the new dam is in place.

Overall, HDR recommends proceeding with the preliminary design of an RCC dam (Alternative 2) at the identified location. If further explorations show that the foundation soils are not suitable for this option, a refinement of Alternative 3 can be investigated.

Figures

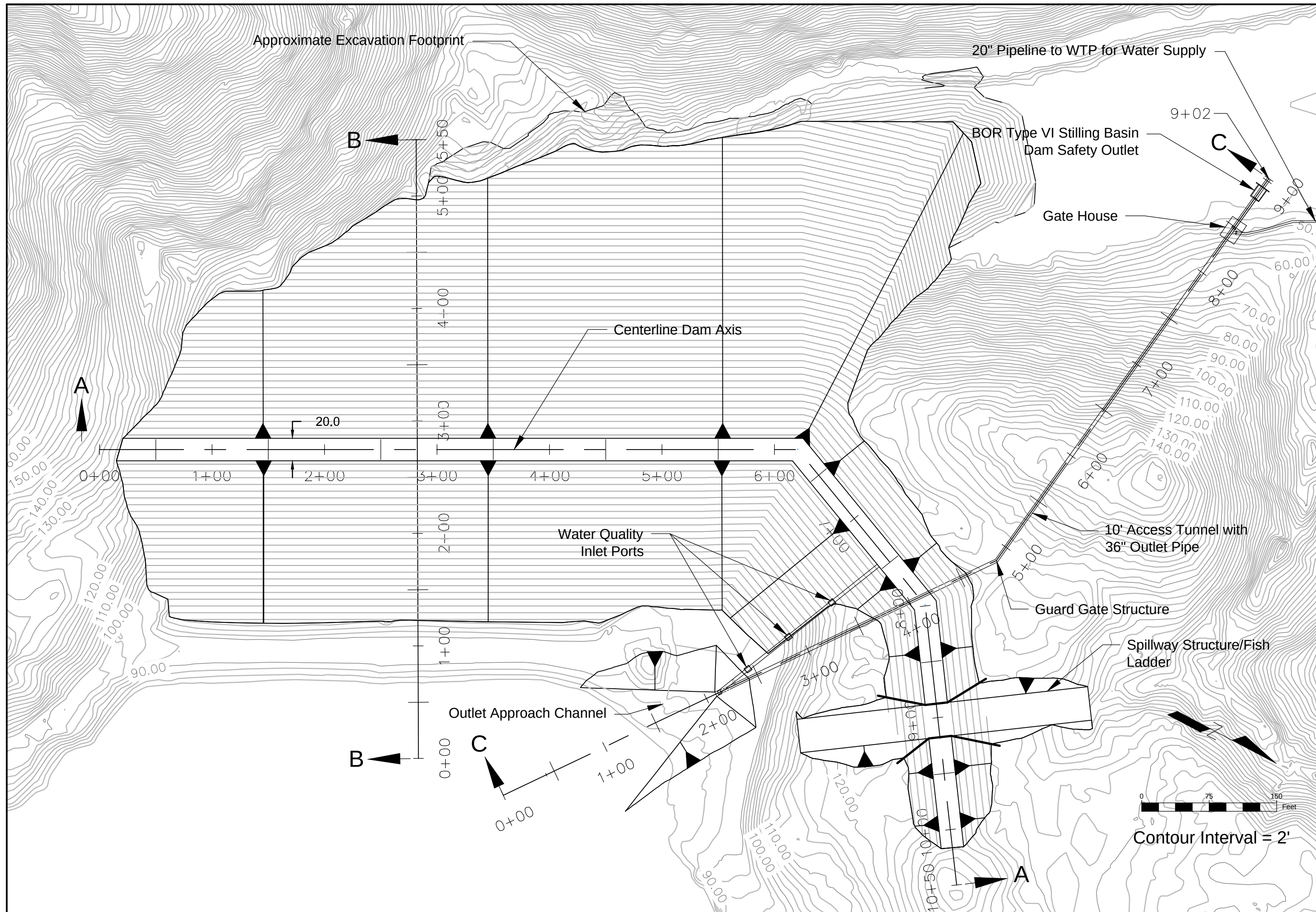


Work Session September 8, 2015

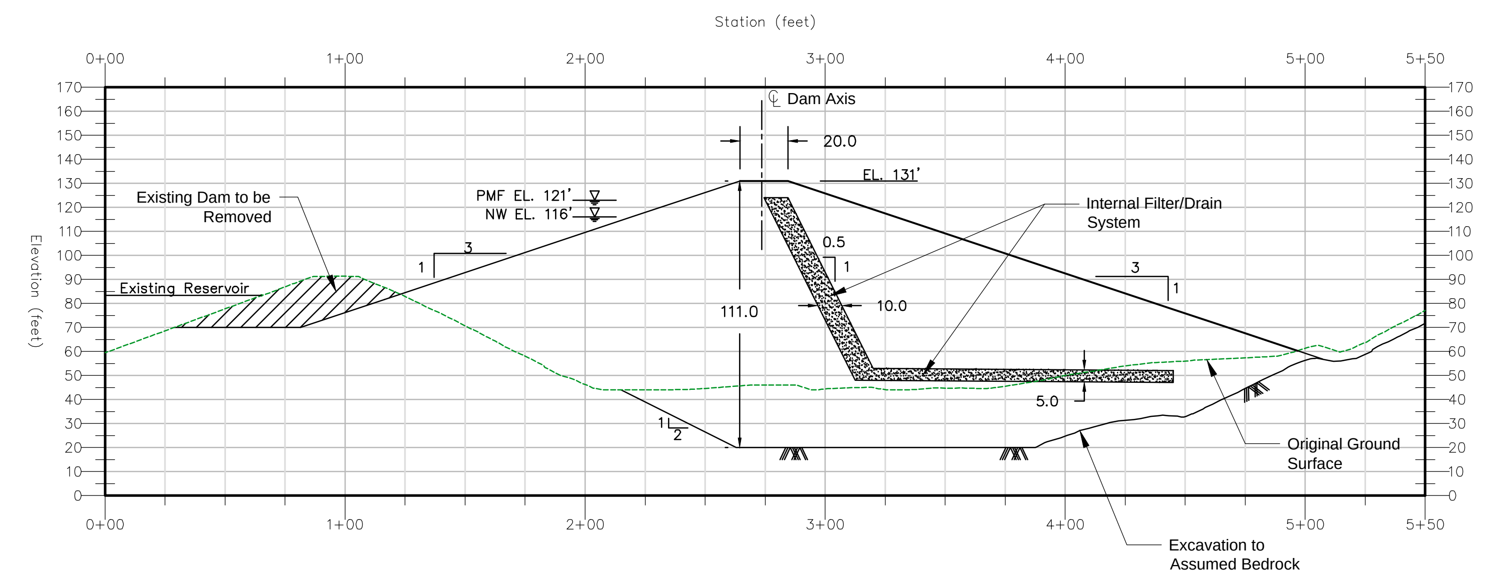
City of Newport
Dam Alternatives Overview

DATE
6-30-2015

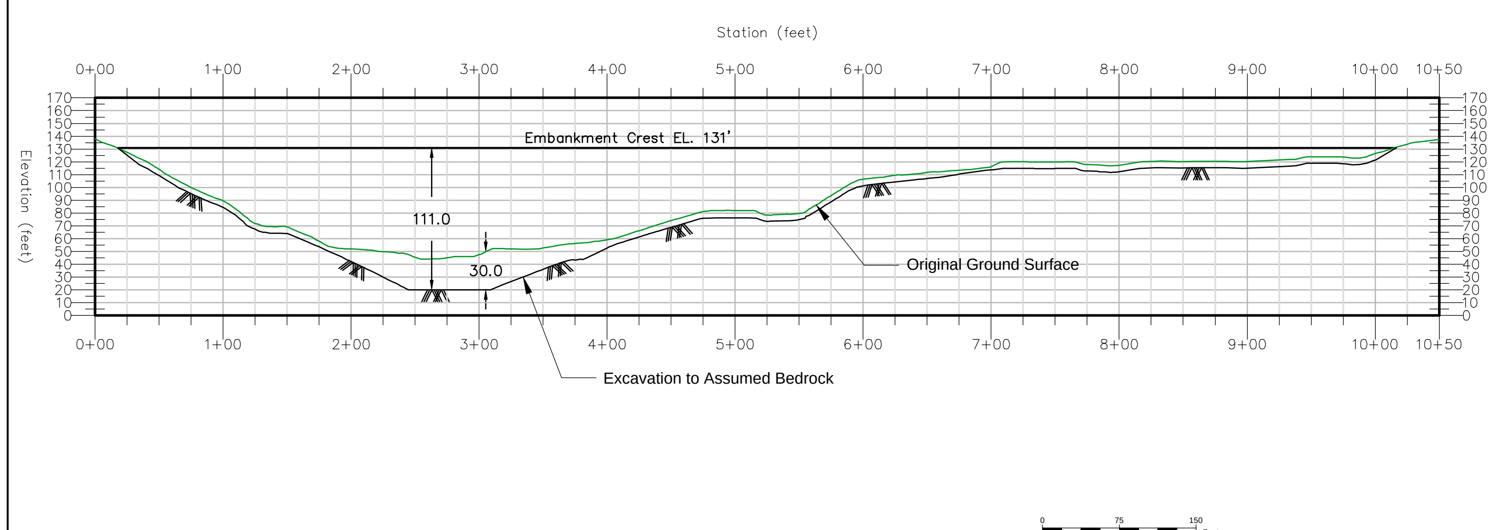
FIGURE
1



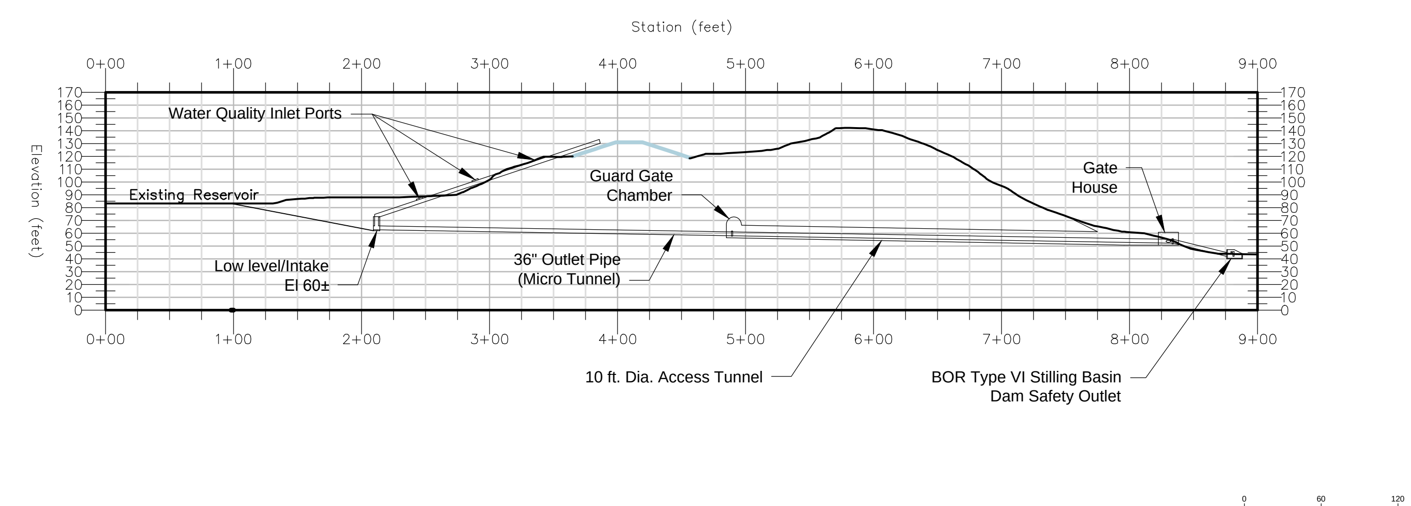
Alternative 1 Embankment Section B–B 2+85



Alternative 1 Embankment Axis Profile A–A



Alternative 1 Embankment Outlet Works C–C



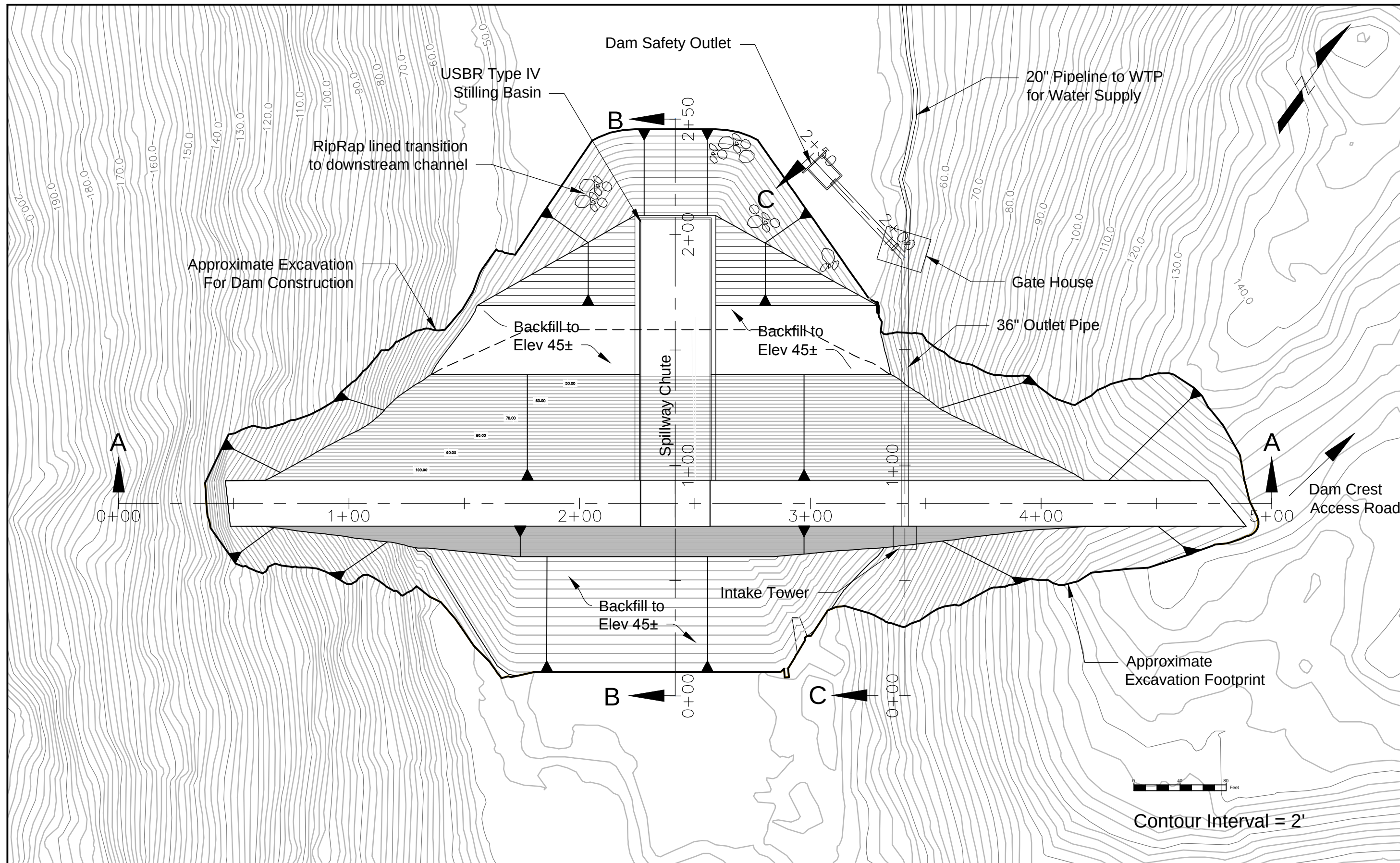
Embankment Volume:	377,000	CY
Excavation Volume:	102,500	CY
Excavation Area:	8.9	AC
Available Storage Volume:	2706	Ac-Ft
Normal Water Elevation:	116	ft
Dam Height:	111	ft

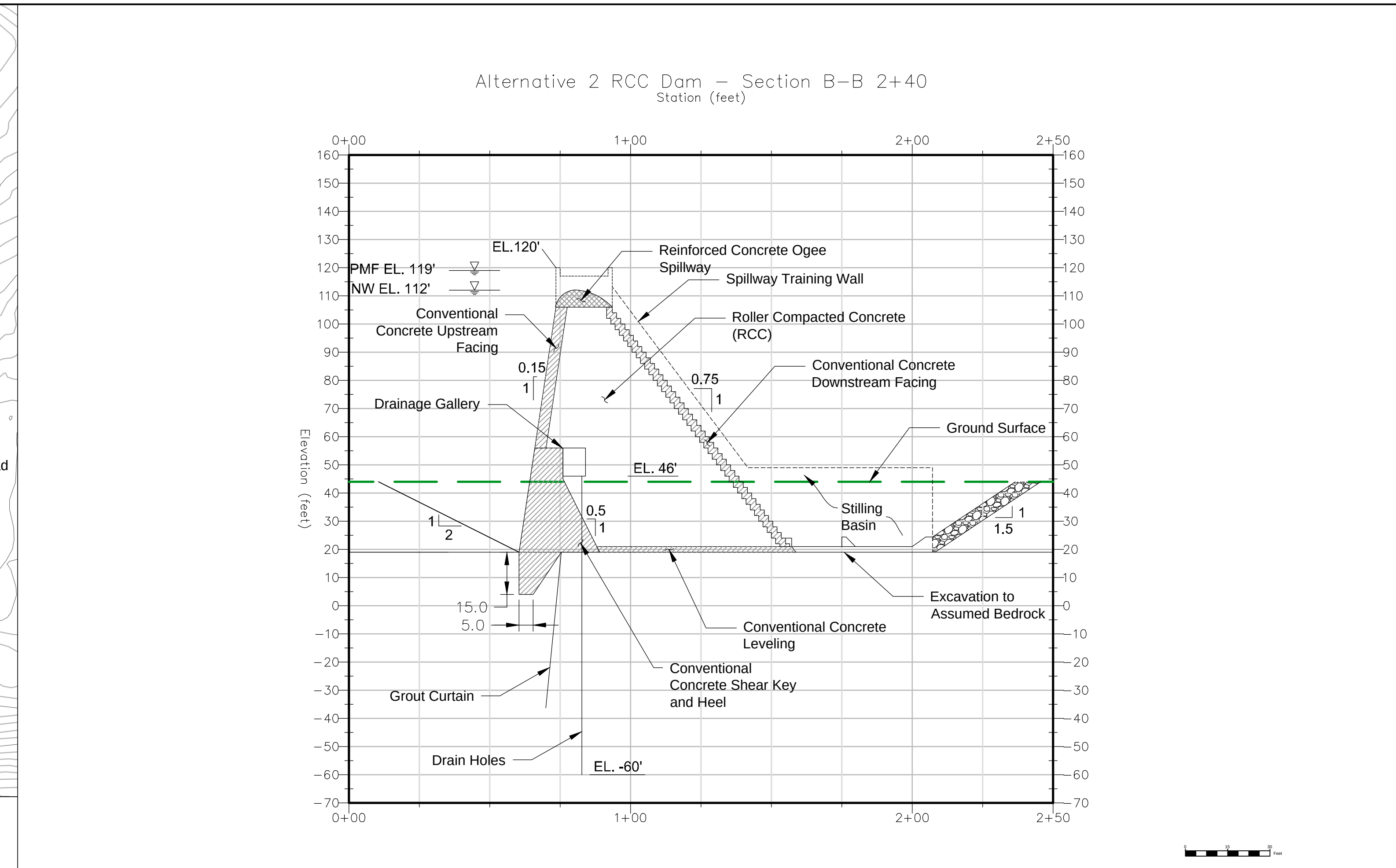
Work Session September 8, 2015

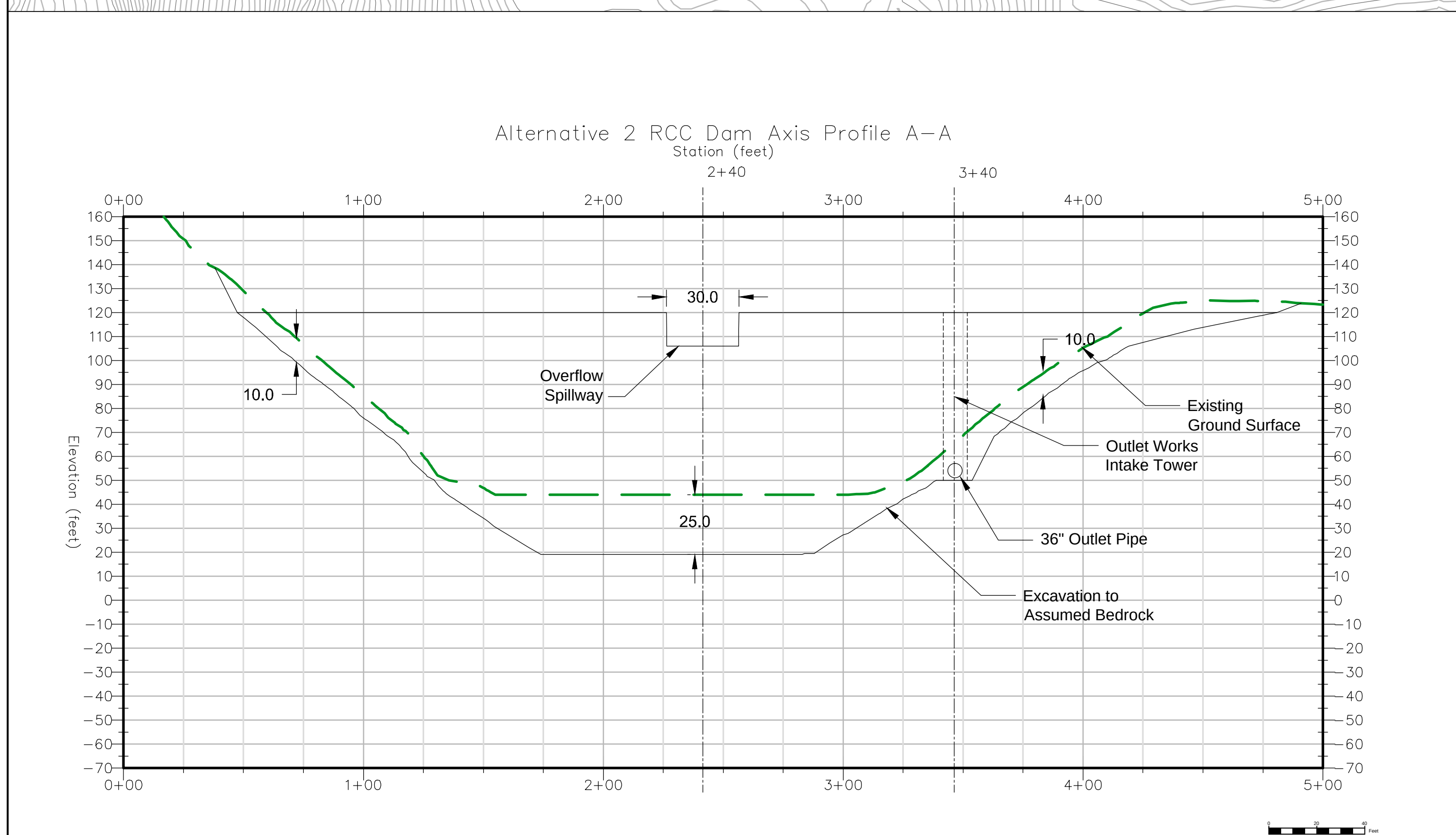
City of Newport
Alternative 1 Upper Dam Embankment Raise

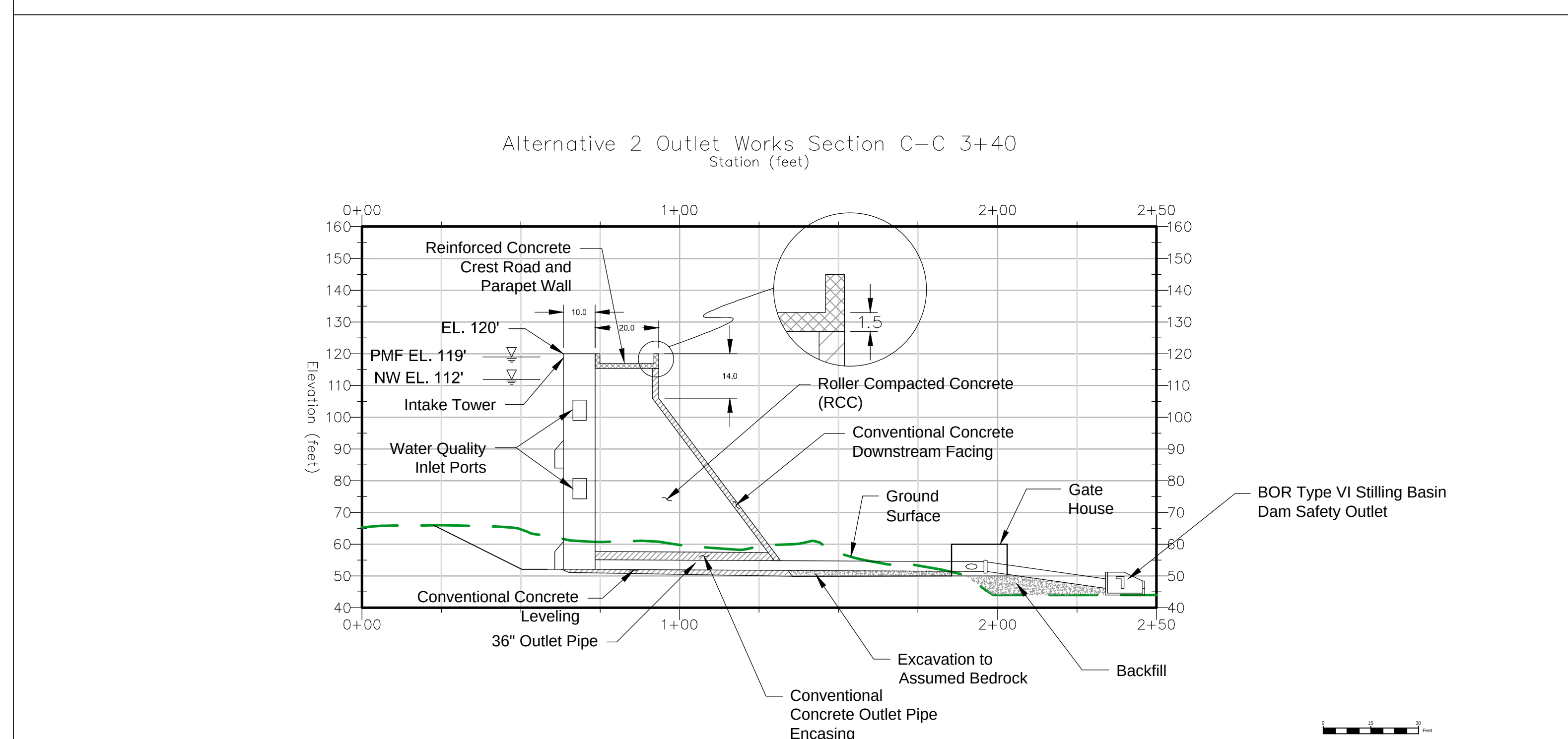
DATE
6-30-2015

FIGURE
2









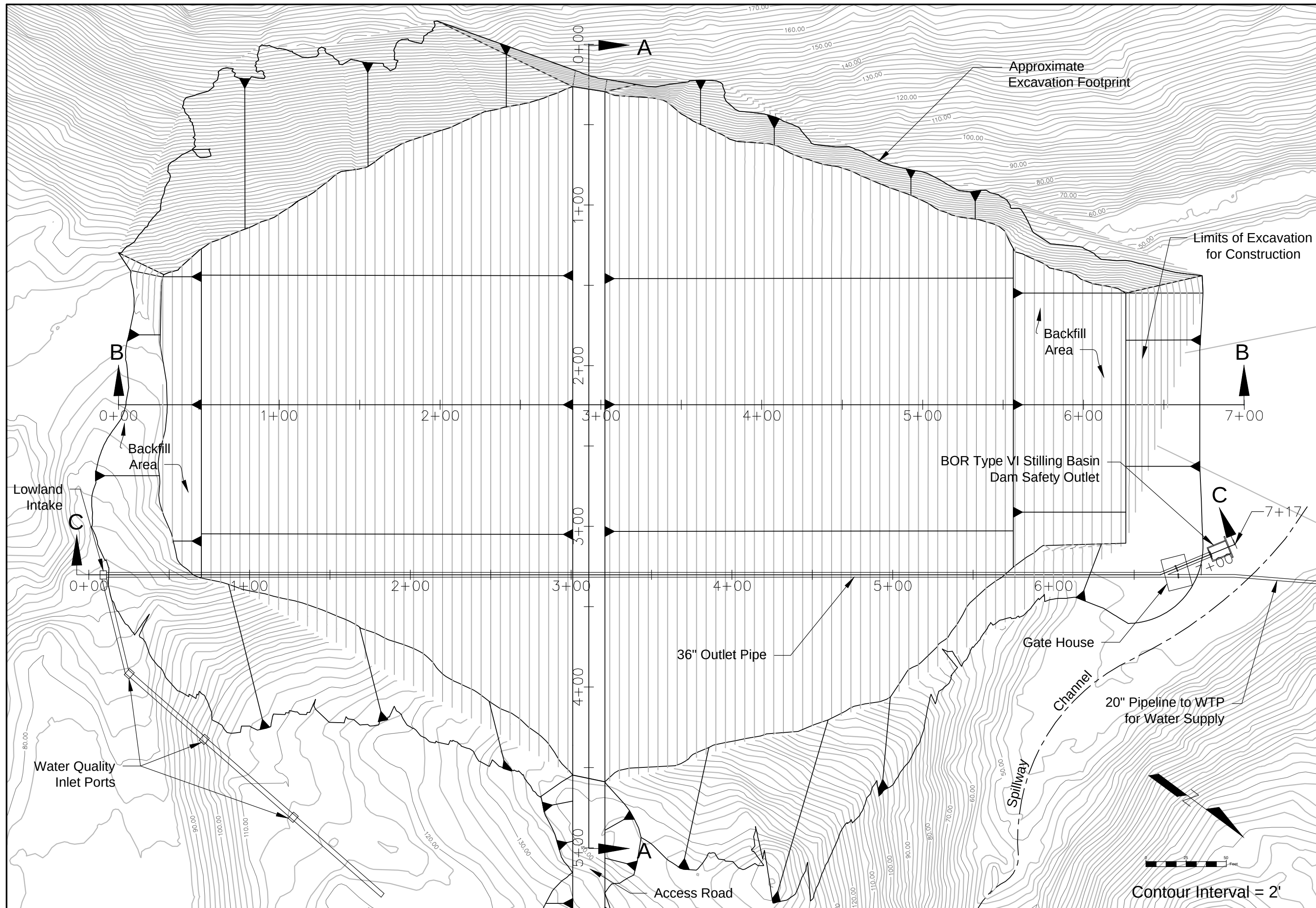
Embankment Volume:	44,723	CY
Excavation Volume:	30,000	CY
Excavation Area:	1.4	AC
Available Storage Volume:	2577	Ac-Ft
Normal Water Elevation:	112	ft
Dam Height:	101	ft

Work Session September 8, 2015

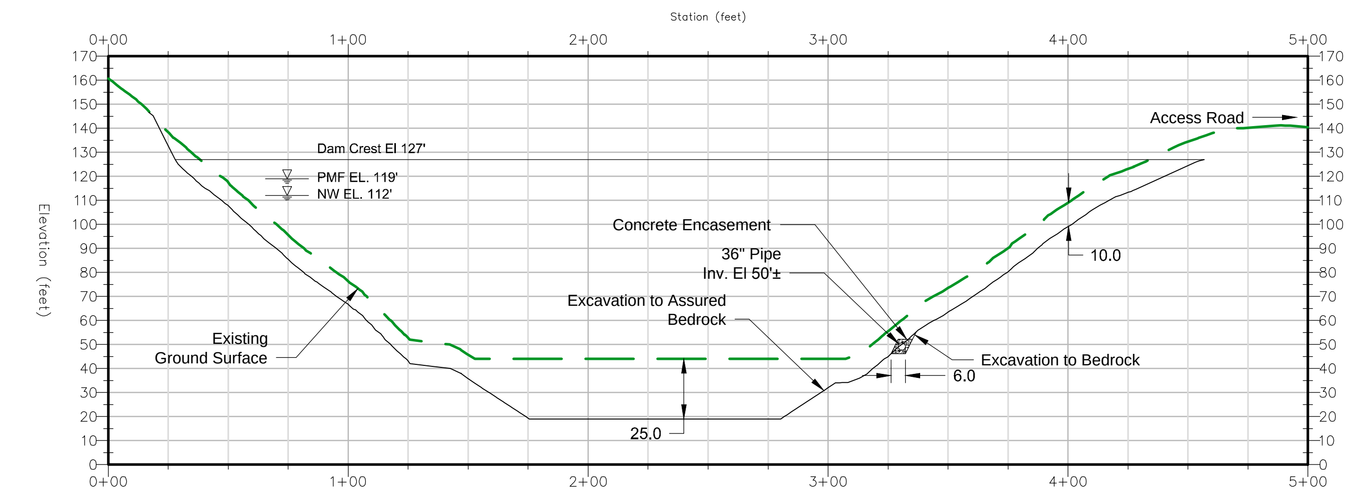
City of Newport
Alternative 2 RCC Dam

DATE
6-30-2015

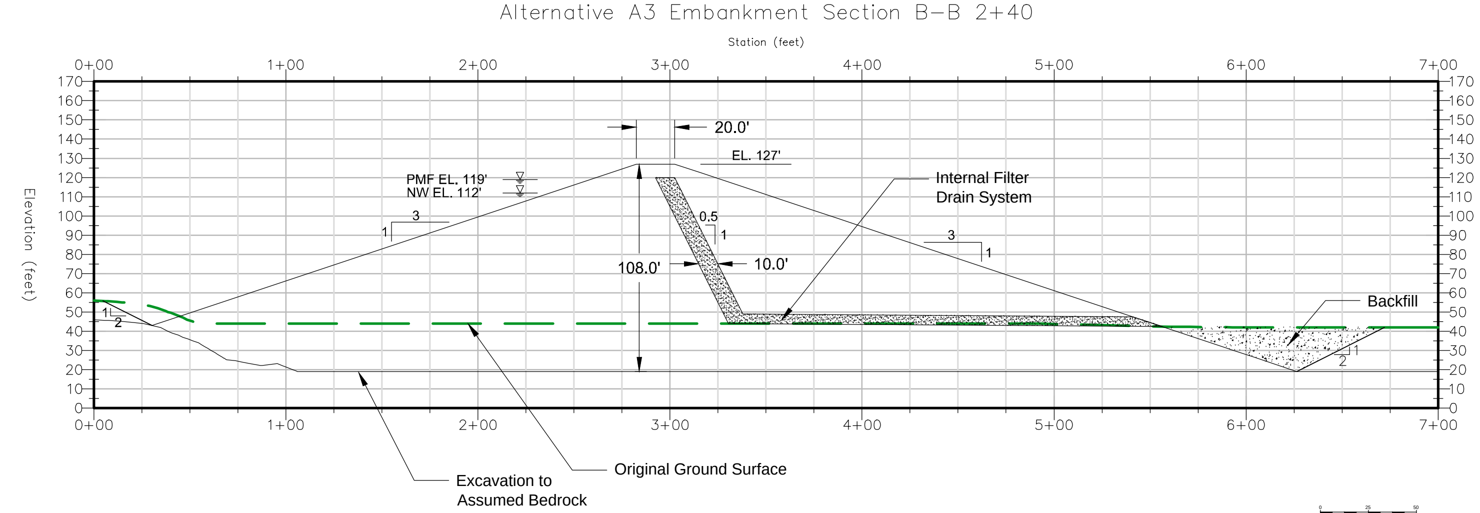
FIGURE
3



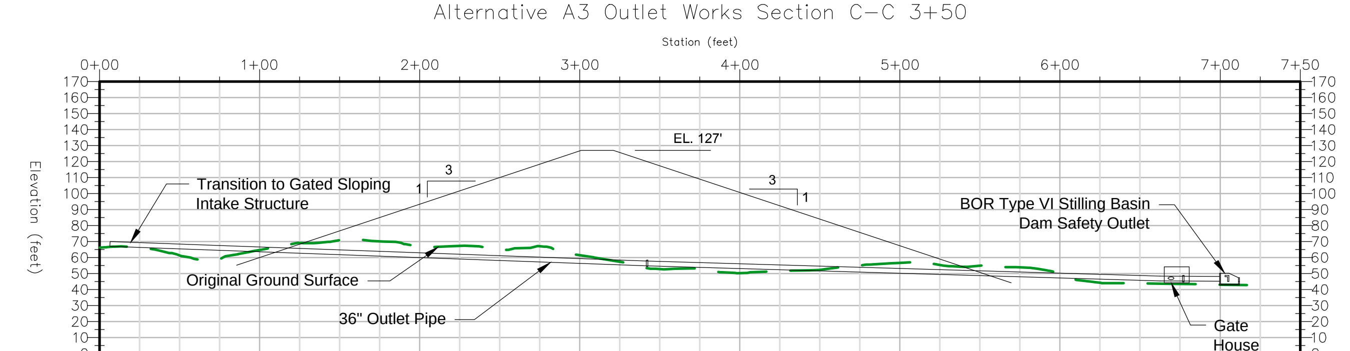
Alternative A3 Embankment Axis Profile A–A



Alternative A3 Embankment Section B–B 2+40



Alternative A3 Outlet Works Section C–C 3+50



Embankment Volume:	301,000	CY
Excavation Volume:	124,280	CY
Excavation Area:	5.9	AC
Available Storage Volume:	2579	Ac-Ft
Normal Water Elevation:	112	ft
Dam Height:	108	ft

Work Session September 8, 2015

City of Newport
Alternative 3 Embankmet Dam

DATE
6-30-2015

FIGURE
4