City of
West Linn

Memorandum

Date: **September 14, 2012**

To: **John Kovash, Mayor**
Members, West Linn City Council

From: **Chris Jordan, City Manager**

Subject: **September 17 Work Session and Council Schedule**

The City Council is scheduled to meet in a work session at 6:00 p.m. on Monday, September 17, 2012. Staff has also invited the members of the Utility Advisory Board and Water System Improvement Task Force to participate in the discussion.

The focus of the discussion will be on our continuing efforts to fund the necessary maintenance and improvements to the City's water system. In February 2012, the Council adopted the following goal regarding the water system:

Water Infrastructure – *The City Council will determine a preferred alternative to secure funding to replace the Bolton Reservoir and/or for ongoing maintenance needs by June 30, 2012.*

• *Meet with representatives of the Utility Advisory Board (UAB) and Finance Department to discuss UAB recommendation (March 2012)*

• *Develop a survey exploring finance and strategic options*

• *Appoint a citizens task force with clear direction to gather additional information, educate the community and provide recommendations to the Council (June 30, 2012)*

All of these bullets have been accomplished and the Council decided that November 2012 was not the best opportunity to successfully place a ballot measure before the voters.

Also, in anticipation of further discussions regarding necessary water system improvements, staff engaged the services of Murray Smith and Associates (MSA) to further investigate the cost of a new 4 million gallon reservoir at the present location of the Bolton reservoir. The MSA report was provided to the Utility Advisory Board last week and is attached to this memorandum.

The September 17 work session is designed to begin to focus the Council on the questions: "What improvements? How much? And, when?" Staff will take this direction, prepare necessary information for review by the group, and would suggest a follow-up work session in approximately two months to determine a final discussion.

In an effort to focus the group on communicating the issue to the public, we have invited a very special guest to the work session. Dr. Lawrence Wallack, Dean of the College of Urban and Public Affairs at Portland State University, will present information on communicating community values.

Future Council Schedule

The Council will next meet in a work session scheduled for October 1. Topics on the agenda for that evening are an update on the police station project and the proposed Robinwood Park trail easement. To ensure staff is fully prepared for the easement discussion, it would be helpful if any requests for information from the Council were received prior to September 21.

The next regular Council meeting is scheduled for October 8.

Attachment

MSA
Murray, Smith & Associates, Inc.
Engineers/Planners
121 S.W. Salmon, Suite 900 ■ Portland, Oregon 97204-2919 ■ PHONE 503.225.9010 ■ FAX 503.225.9022

TECHNICAL MEMORANDUM

DATE: August 31, 2012

PROJECT NO.: 12-1334.405

TO: Mr. Jim Whynot, Water Superintendant
City of West Linn

FROM: Thomas P. Boland, P.E.
Lael L. Alderman, P.E.
Murray, Smith & Associates, Inc.

REGISTERED PROFESSIONAL ENGINEER
59976
OREGON
JULY 13, 1999
THOMAS P. BOLAND
EXPIRES: 12/31/13

RE: City of West Linn, 4.0 Million Gallon Bolton Reservoir Project
Conceptual Siting Analysis

Introduction and Purpose

The City of West Linn (City) is proposing to replace the existing 2.5 Million Gallon (MG) Bolton Reservoir with a new 4.0 MG reservoir. In accordance with the City's authorization, Murray, Smith & Associates, Inc. (MSA) has completed a conceptual siting analysis for the contemplated project, which consists of demolition of the existing reservoir and construction of a new 4.0 MG reservoir within the existing site boundaries. Siting analysis work included confirming floor and overflow elevations and preliminary tank dimensions, evaluating and recommending reservoir type, completing a preliminary geotechnical evaluation of the site, confirming the orientation of the structure on the site and developing budget level cost estimates to allow the City to proceed with project financing. The purpose of this memorandum is to present findings, recommendations and conceptual cost estimates resulting from the siting analysis work.

Background

The Bolton Reservoir site is located northerly of Skyline Drive on City owned property, near the intersection of Skyline Drive and Skyline Circle. The existing reservoir is a concrete slab-on-grade structure with 2:1 (horizontal: vertical) side slopes, constructed in 1913. An interior liner was installed in 1989 and a Hypalon cover was placed over the reservoir in 1995. Previous inspections of the reservoir identified some concrete spalling and some localized cracking. The floating cover for the reservoir appears to be reaching the end of its service life. Based on inspection and recent repairs of holes/tears in the cover in January 2008 and extensive repairs again in 2012, it is beginning to show significant signs of wear, especially at locations where movement occurs, such as around the rain troughs (gutters), and it is likely that the cover will require more frequent repairs in the near term.

Approximately 0.5 MG of the total 2.5 MG volume of the Bolton Reservoir is unusable due to the low elevation of the reservoir floor relative to the higher elevation of the reservoir outlet piping and suction piping of the adjacent Bolton Pump Station, which pumps water from the reservoir into the distribution system's Horton Pressure Zone. Given the reservoir's functional limitations, condition, and age, replacement of the Bolton Reservoir was recommended in the City's previous three Water System Master Plans, including the most recent completed in 2008, in which it was considered a high priority improvement. The new reservoir will be oriented such that its floor and overflow elevations meet hydraulic system requirements and maximize usable storage at the existing site.

Reservoir Elevation

The existing 2.5 MG Bolton Reservoir has an overflow elevation of approximately 442 feet and floor elevation of approximately 426 feet. The invert elevation of the existing reservoir inlet/outlet pipe is approximately 427.75 feet, and the elevation of the existing Bolton Pump Station pump suction piping is approximately 428.75 feet. This 2 to 3-foot difference in elevation between the lower reservoir floor, and the higher reservoir outlet piping and pump station suction piping, results in approximately 0.5 MG of unusable "dead" storage in the existing reservoir.

To maximize usable storage at the site, the overflow elevation of the proposed reservoir should be raised to approximately 450 feet, 8 feet higher than the overflow elevation of the existing reservoir. The floor elevation should be raised to 430 feet, resulting in a water depth of 20 feet. This additional elevation, added to both the reservoir's maximum water surface and floor, will provide more effective service to the Bolton pressure zone from the reservoir when the South Fork Water Board (SFWB) Division Street Pump Station is not in operation, and will provide improved suction pressure to the Bolton Pump Station.

Reservoir Shape/Configuration

The City requested an analysis of two alternative reservoir shapes, circular and rectangular, to determine the most economical configuration of the proposed reservoir based upon estimated construction costs. Site plans and cross sections were developed for a circular prestressed concrete structure (AWWA D110, Type I) and rectangular conventionally reinforced concrete structure (ACI 350) with appropriate dimensions needed to achieve a 4.0 MG reservoir capacity with the proposed 20-foot water depth identified above. The circular configuration shown on the attached Figures 1.A and 1.B, has a diameter of 190 feet, and a water depth of 20 feet. The rectangular (or non-circular) configuration, with dimensions shown on the attached Figures 2.A and 2.B, is a two cell structure with a center dividing wall and water depth of 20 feet. The southeast corner of the non-circular structure was cut back to accommodate existing site constraints. For both alternatives, reservoir site orientation was dependant on geotechnical findings and recommendations described below.

Geotechnical Considerations

A preliminary geotechnical evaluation of the reservoir site was completed by GRI as part of the conceptual siting analysis. This evaluation included a review of existing geotechnical and geologic information; geologic reconnaissance; drilling, logging and sampling of one boring; limited laboratory testing; geologic and engineering analyses; discussions and meetings with MSA and City staff. Findings and recommendations from this evaluation are summarized below and presented in the draft geotechnical report included in Appendix A.

Three significant geotechnical factors associated with the placement of a new concrete reservoir on the Bolton Reservoir site include:

1. The reservoir site is located on a very large, old, or “ancient” landslide, as mapped by DOGAMI.

2. Landsliding has occurred on the slope at the northeast corner of the reservoir. Associated ground cracking occurred very near the reservoir following heavy rainfall in 1996.

3. Borings made by GRI and others indicate that significant portions of the existing reservoir, particularly along the north side, are underlain by fill. The quality of the fill is unknown.

Ancient Landslide -- The reservoir site is located on a very large landslide, as recently mapped by DOGAMI. The landslide was identified based primarily on landforms and topography, as revealed by LiDAR imagery. DOGAMI considers the landslide to have occurred at least 150 years ago and possibly much longer. DOGAMI is not aware of any significant movement of the landslide in modern times. The landslide is likely associated with the Vantage Horizon, which is a soil-like zone interbedded between two basalt flows. Other large landslides in the West Linn area are also associated with the Vantage Horizon. A reconnaissance by GRI did not disclose indications of relatively recent movement, such as cracked streets, sidewalks, or curbs. GRI’s inquiries also did not disclose reports of broken or sheared underground utilities, which could also indicate ground movement. It is GRI’s overall opinion that the risk of significant future movement of the large old landslide mass that could impact a reservoir on the site is probably low, but the risk is not totally absent. Due to the large size of the mapped landslide, GRI anticipates that mitigation measures to improve the stability of the ancient landslide are likely not practical or cost effective.

Localized Landsliding -- A landslide occurred on the slope at the northeast corner of the existing reservoir sometime in the 1970s. Extensive significant ground cracking occurred following heavy rainfall in 1996 and extended to near the reservoir. Based on GRI’s reconnaissance and review of the available topographic information, the undisturbed slopes along much of the north side of the existing reservoir are similar to the slopes at the landslide and likely also have a relatively low factor of safety against landslide.

In June 2012, GRI obtained measurements in the inclinometer that was installed by Landslide Technologies in 1997 at the approximate location of localized landsliding. The inclinometer was installed to provide long-term monitoring of the site with respect to potential slope movement. GRI obtained the baseline measurements collected by Landslide Technologies in 1998 and compared that data with measurements collected from the inclinometer by GRI on June 15, 2012. The comparison of the measurements indicates the upper 5 feet of soil in the vicinity of the inclinometer had moved approximately 6 inches since the 1998 readings were collected.

GRI reported that the risk to a new reservoir by local landsliding can be significantly reduced by setting the reservoir back from the slope, and recommended establishing the reservoir at least 75 feet south of the point at which a horizontal plane at elevation 430 feet (floor level) intercepts the slope. In addition, GRI noted that the stability of the slope can be improved by removing material at the top of the slope and south of the reservoir footprint to reduce the risk of future landsliding or slope movement that could affect the new reservoir.

Existing Fill -- Exploration borings made by GRI and others indicate the presence of significant thicknesses of fill along the north side of the reservoir. The fill likely underlies at least portions of the existing reservoir and footprint of the proposed reservoir. The quality of the fill is unknown; however, there is significant cracking of concrete features along the northwest portion of the existing reservoir. The cracking could be associated with settlement of the fill. The presence of existing poorly compacted or compressible fill beneath the new reservoir could result in excessive and unacceptable total and differential settlement. For this reason, it would be reasonable to assume for preliminary budgeting purposes that ground improvement, such as 20-foot deep rammed aggregate piers, will be needed beneath at least half of the new reservoir footprint, with over-excavation and a minimum 5-foot thickness of crushed rock beneath the remainder of the reservoir. A more conservative approach would be to assume that rammed aggregate piers will be needed beneath the entire footprint to an area 10 feet beyond the reservoir footing.

Seismic Considerations -- Three potentially active faults are located near the site, as described in the attached geotechnical evaluation report. As a result, ground motions during a design earthquake will be larger than for sites located further from the faults. Based on the available subsurface information, the site class is likely D or E. However, a site-specific seismic hazard study will be necessary as part of the preliminary engineering phase to develop criteria for design and evaluation of the reservoir.

Reservoir Orientation and Site Layout

The existing reservoir site has a potential usable area of approximately 250-feet by 250-feet for locating a new reservoir structure, due to existing site constraints which include the 75-foot recommended slope setback, and the existing Bolton Pump Station and utility infrastructure. The area in the vicinity of the existing reservoir and pump station is generally flat and slopes upward to the south at an approximate 10 percent grade, through a grove of large conifer trees to Skyline Drive. The steep slope and area located northerly of the recommended slope setback is considered unsuitable for the tank structure but could be used for temporary construction staging.

Circular Prestressed Concrete Reservoir -- The reservoir site layout for the circular prestressed concrete structure, shown on Figures 1.A and 1.B, was developed to meet minimum siting requirements including a minimum 10-foot circumferential clearance from the reservoir wall for prestressing equipment, maximum 1:1 (horizontal: vertical) temporary excavation slopes, and maximum 2:1 final grading slopes. Excavation shoring may be required in certain areas to provide adequate setback from existing structures and property lines, or may be desired to minimize construction impacts to the existing site. A maximum 5-foot differential in backfill height around the structure was maintained as part of finished grading, and a permanent 15-foot wide perimeter access road was included to provide vehicle access around the tank. The cross section view shown on Figure 1.B shows subsurface ground improvements based on recommendations from the preliminary geotechnical evaluation. These include subgrade over-excavation and replacement with 5 feet of crushed rock to provide a drainage layer and working mat, and rammed aggregate piers extending 20 feet below the mat, in an the area extending 10 feet beyond the reservoir footing. The cross section also shows finished grade slope improvements located to the north of the reservoir, involving removal of material at the top of the slope to improve slope stability.

Non-circular Conventionally Reinforced Concrete Reservoir -- The reservoir site layout for the non-circular conventionally reinforced concrete structure, shown on Figures 2.A and 2.B, was also developed to meet minimum siting requirements including a minimum 10-foot circumferential clearance from the reservoir wall, maximum 1:1 (horizontal: vertical) temporary excavation slopes, and maximum 2:1 final grading slopes. Excavation shoring may be required in certain areas to provide adequate setback from existing structures and property lines, or may be desired to minimize construction impacts to the existing site. A maximum five foot differential in backfill height around the structure was maintained as part of finished grading, and a permanent 15-foot wide perimeter access road was included to provide vehicle access around the tank. The cross section view shown on Figure 2.B shows subsurface ground improvements based on recommendations from the preliminary geotechnical evaluation. These include subgrade over-excavation and replacement with 5 feet of crushed rock to provide a drainage layer and working mat, and rammed aggregate piers extending 20 feet below the mat, in an area extending 10 feet beyond the reservoir footing. The cross section also shows finished grade slope improvements located to the north of the reservoir, involving removal of material at the top of the slope to improve slope stability.

Conceptual Design Elements

Outlined below is a brief discussion and summary of the key conceptual design elements used for the siting analysis and preparation of cost estimates.

Circular Prestressed Concrete Reservoir (AWWA D110, Type I) -- A strand-wound, circular, prestressed concrete reservoir designed and constructed to *AWWA D110, Type I* standards offers a low maintenance option for finished water storage, and may be partially or fully buried. This reservoir type has cast-in-place concrete walls with vertical prestressed reinforcement. Vertical prestressing tendons are cast inside the structure's walls to provide compression that counteracts the effects of differential dryness and thermal loads. To provide an unrestrained connection and to reduce bending moments induced by hydrostatic, thermal, backfill and seismic forces on the tank wall, the roof and floor are separated from the corewall by neoprene bearing pads. This "free-sliding" connection at the wall base and wall top enhances the seismic performance of the tank by allowing the floor, wall and roof to act independently of each other. A continuous PVC bulb waterstop between the floor and wall assures full liquid tightness of the joint. Typical floor designs include a concrete, two-way, flat slab with a slight upward slope to the tank's center, cast monolithically or in the minimum number of pours to minimize joints.

Non-circular Conventionally Reinforced Concrete Reservoir (ACI 350) -- A conventionally reinforced concrete structure, designed and constructed to *ACI 350* standards, was also reviewed as an option for the replacement reservoir. This type of structure may also be partially or fully buried. For cost estimating purposes it was assumed that the conventionally reinforced structure would be designed with a typical spread footing/membrane floor system, a fixed-base configuration, and a flat slab column supported roof. A dual-cell structure with a dividing wall was included, to allow the operational flexibility of taking one cell off-line for maintenance while keeping the other cell operational. Because conventional reinforced concrete resists loads by flexure, or bending in the reinforced concrete, the concrete must crack to engage the reinforcing steel in tension to resist the bending forces. Therefore, it should be expected that the walls and foundation of a properly designed and constructed conventionally reinforced concrete structure will experience flexural cracking and potential leakage at flexural cracks which will require more repairs and future maintenance over its design life than a circular prestressed concrete structure. Generally, the AWWA D110, Type I base isolated reservoir system offers a significantly higher level of life cycle performance than the conventionally reinforced fixed base alternative.

Site Access -- The Bolton Reservoir site has two means of gated vehicle access, one from Skyline Drive to the south, and the second from Skyline Circle to the east. Due to the deep excavation and limited space south of the anticipated construction zone, it is anticipated that temporary construction access will be primarily provided via Skyline Circle, and that access from Skyline Drive may be closed temporarily during some phases of construction. It is anticipated that a total of up to 5,000 to 6,000 truck trips may be required during construction.

Reservoir Piping -- Reservoir piping will include separate inlet and outlet lines, as well as overflow and drain piping. Estimated project costs include passive mixing systems for both tank style alternatives. A separate reservoir drain will extend through the reservoir floor and connect to overflow pipe adjacent to the exterior edge of the reservoir's foundation. Overflow piping will be routed and connected to existing facilities near site's northwestern property boundary. Double ball, flexible expansion joints are recommended for installation on the proposed inlet, outlet and overflow lines, placed adjacent to the exterior edge of the reservoir's foundation to provide the piping with additional flexibility to prevent damage from potential seismic induced settlement or movement. Isolation valving is also recommended on the inlet, outlet and overflow lines between the exterior edge of the reservoir's foundation and flexible expansion joints to provide for isolation of the facility should seismic induced damage occur to site piping.

Reservoir Bypass Facilities -- In order to allow for the decommissioning of the existing Bolton Reservoir and construction of the new Bolton Reservoir, construction of reservoir bypass facilities will be needed to allow for continued supply from the Division Street Pump Station to the Bolton pressure zone (essentially "floating" off of the hydraulic grade line of Oregon City's Mountain View Reservoir) and emergency back-up. These improvements include adding manual valves to isolate/bypass the reservoir and modifying existing pump station piping and valving to provide an emergency connection from the from the Horton Pressure zone to the Bolton pressure zone in the event there is a waterline failure from the Mountain View Reservoir during construction. The cost estimates in this memorandum assume that these facilities will be constructed as permanent bypass facilities to allow for future shut down of the new Bolton Reservoir.

Demolition of Existing Bolton Reservoir -- The existing 2.5 MG reservoir will be demolished and removed. The existing reservoir foundation will be removed as part of the excavation for the proposed reservoir. The existing piping associated with the reservoir will be abandoned in-place or removed.

Demolition of Old Bolton Pump Station -- With the construction of the new Bolton Reservoir, the Old Bolton Pump Station can be completely decommissioned and demolished.

Abandonment of 14-inch Diameter Waterline from Old Bolton Pump Station to Skyline Drive -- With the construction of the new Bolton Reservoir and the demolition of the Old Bolton Pump Station, the existing 14-inch diameter pump station discharge main to the Horton pressure zone distribution piping in Skyline Drive can be decommissioned and abandoned.

PRV Vault and New 8-inch Diameter Main in Skyline Circle -- The aging 6-inch diameter Horton pressure zone water main in Skyline Circle should be replaced as part of the proposed Bolton Reservoir project. This main also supplies a pressure reducing valve (PRV) station, at the intersection of Skyline Circle and Skyline Drive, and dead-end 2-inch diameter main extending east on Skyline Drive. A new 8-inch diameter main and PRV from the discharge of the Bolton Pump Station should be extended down Skyline Circle, approximately 500 feet in length, to replace the 6-inch diameter main. This new main should be extended as 4-inch diameter main, approximately 650 feet in length, from the end of Skyline Circle to replace the 2-inch diameter main.

Replacement 18-inch Diameter Waterline in Skyline Circle with 24-inch Diameter Waterline -- Portions of the 18-inch diameter transmission main supplying the reservoir, from the new reservoir inlet/outlet east in Skyline Circle to Skyline Drive, should be replaced as part of the proposed Bolton Reservoir replacement project. Parts of the existing transmission main are located above the floor elevation of the existing and proposed reservoir, limiting the inlet/outlet capacity of the reservoir. It is recommended that this section of transmission main, approximately 500 to 700 feet in length, be upsized to 24-inch diameter and constructed at a depth below the proposed floor elevation of the new Bolton Reservoir.

Replacement 6-inch Diameter PVC Main from Bolton Reservoir to Barclay Street -- As part of the proposed Bolton Reservoir Replacement project, the existing 6-inch diameter PVC main serving the Bolton pressure zone, extending across the north side of the Bolton Reservoir site should be replaced to the west side of the site where it transitions to cast iron main and continues cross country along property lines to Barclay Street. The existing main is constructed at an elevation near to the normal operating level of the Bolton Reservoir. As such, under peak demand conditions this water main drains when the Bolton Reservoir level drops below the elevation of the main. This section of 6-inch diameter PVC main, approximately 300 feet in length, should be upsized to 8-inch diameter and installed at a depth below the floor elevation of the Bolton Reservoir.

Estimated Project Costs

Budget level project cost estimates were developed for each reservoir shape alternative and are summarized in Table 1. The estimated project costs for the proposed replacement reservoir and associated site improvements, for the circular prestressed concrete structure (AWWA D110, Type I), are presented with a detailed cost breakdown in Attachment 1. The estimated project costs for the proposed replacement reservoir and associated site improvements, for the non-circular conventionally reinforced concrete structure (ACI 350), are presented with a detailed cost breakdown in Attachment 2.

Table 1
Alternative Cost Summary

Cost Component	Circular Prestressed Concrete Reservoir Alternative	Non-circular Reinforced Concrete Reservoir Alternative
Subtotal Estimated Construction Cost (1)	$6,135,000	$6,423,000
+ 20% Construction Contingency	$1,227,000	$1,285,000
Total Estimated Construction Cost	$7,362,000	$7,708,000
+ 20% Allowance for Engineering, Permitting, Construction Management and City Administration	$1,473,000	$1,542,000
Total Project Cost	***$8,835,000***	***$9,250,00***

1. This preliminary construction cost estimate is an opinion of cost based on information available at the time of the estimate. Final costs will depend on actual field conditions, actual material and labor costs, market conditions for construction, regulatory factors, final project scope, method of implementation, schedule and other variables. The Engineering (ENR) Construction Cost Index for Seattle, Washington is 9061.73 (August 2012).

The preliminary construction cost estimates are based on conceptual level design elements, and are considered Class 4 estimates as defined by the Association for the Advancement of Cost Engineering (AACE) with an expected accuracy range of +30% to -50%. The suggested contingency for projects similar in scale and size to the construction of the City's proposed replacement reservoir project is 20 percent of the overall estimated construction cost to allow for flexibility in addressing any possible unforeseen conditions that may arise during construction. An additional 20 percent allowance was included for engineering, permitting, construction management and City administration costs.

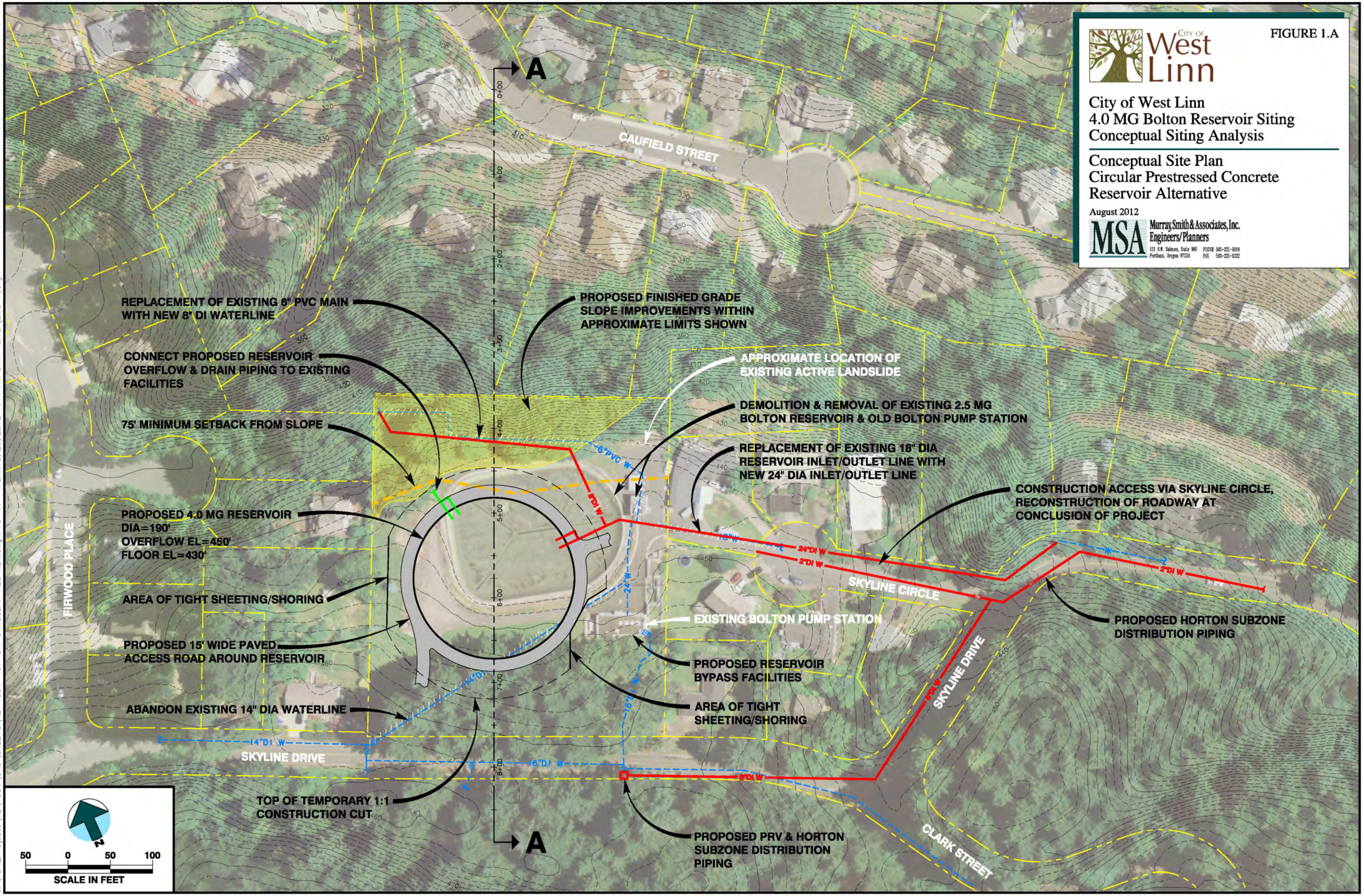
Preliminary Project Schedule

A preliminary project schedule was developed for design and construction of the proposed reservoir replacement project and is included as Attachment 3. It is anticipated that the project schedule will be similar for both reservoir shape alternatives, therefore it is expected that schedule will not be a factor in recommendation of an alternative. The preliminary project schedule identifies the anticipated timeline from preliminary engineering through final design, permitting, bidding and construction completion. Estimates of construction duration indicate that operation of the water system without storage at the Bolton site will be required through one full summer season in 2015. Based on this schedule, the new reservoir would be substantially complete and ready to be put into service by the end of June 2016, following the completion of reservoir backfill and on-site piping. An estimated project start date of October 1, 2013 has been included, with final project completion anticipated by late August 2016, due to an extended allowance for conditional use permitting.

Summary and Recommendations

This technical memorandum is intended to identify the key conceptual design issues associated with the project, recommend an orientation and configuration of the proposed reservoir, and provide a budget level cost estimate for the proposed project. To determine the most cost efficient and constructible layout, two reservoir orientations and configurations were evaluated. The evaluation included consideration of the existing physical site constraints, including an active landslide within the existing reservoir site; the presence of existing water storage and pump station facilities and the impacts of subsurface geological conditions on reservoir foundation conditions; reservoir style, configuration and size; integrating the proposed reservoir with existing facilities; and overall project costs. Based on the relative costs for each alternative, a circular, prestressed concrete reservoir designed and constructed to AWWA D110, Type I standards is the recommended alternative.

FIGURES



G:\PDX_Projects\12\1334\CAD\SHEETS\12-1334-401-OR-CIRCULAR RES ALT.dwg FIG 1A - PLAN 8/29/2012 1:11 PM DAK 18.1s (LMS Tech)

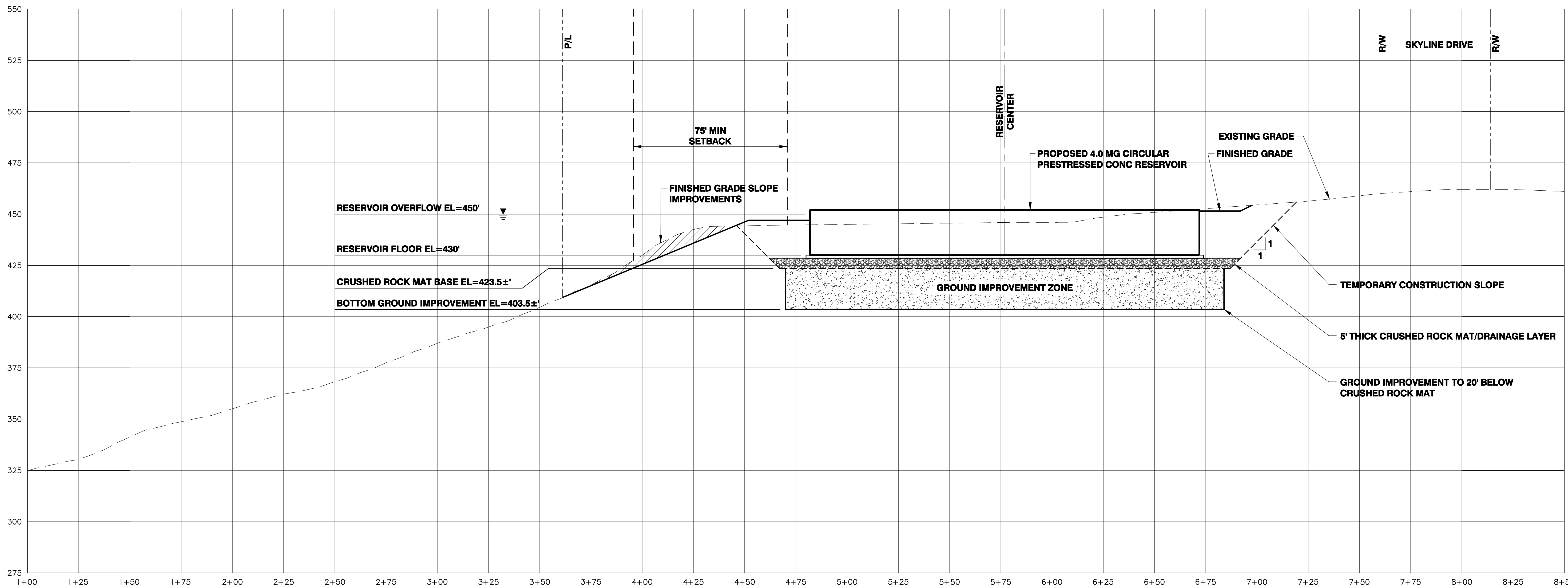
FIGURE 1.B

City of West Linn
4.0 MG Bolton Reservoir Siting
Conceptual Siting Analysis

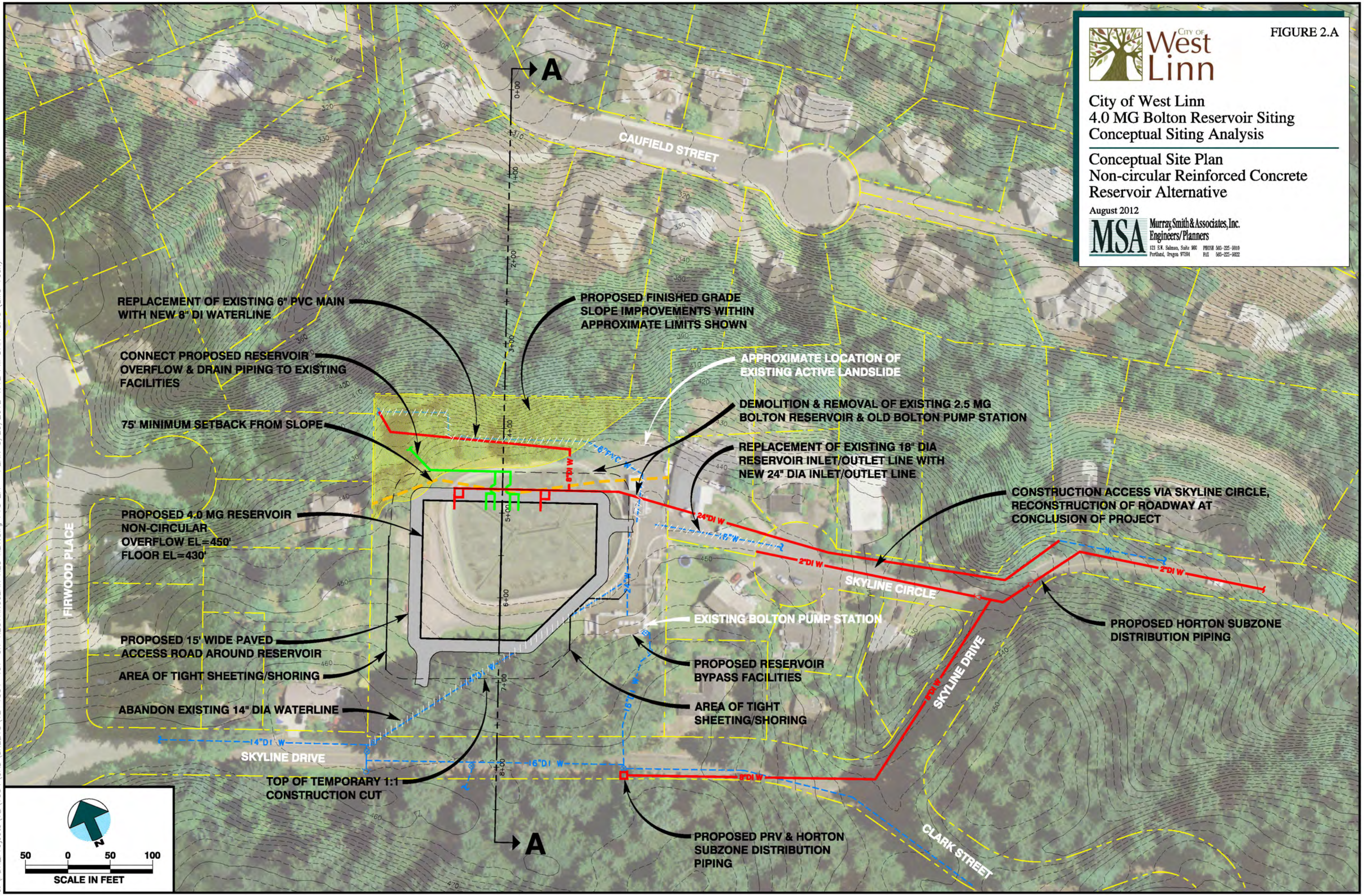
Section A
Circular Prestressed Concrete
Reservoir Alternative

August 2012

Murray, Smith & Associates, Inc.
Engineers/Planners
121 S.W. Salmon, Suite 900 Portland, Oregon 97204
PHONE 503-225-9010 FAX 503-225-9022



SECTION A
SCALE: 1"=50' HORIZ & VERT



G:\PDX_Projects\12\1334\CAD\SHEETS\12-1334-401-OR-RECTANGULAR RES ALT.dwg FIG 2A - PLAN 8/29/2012 1:12 PM DAK 18.1s (LMS Tech)

FIGURE 2.B

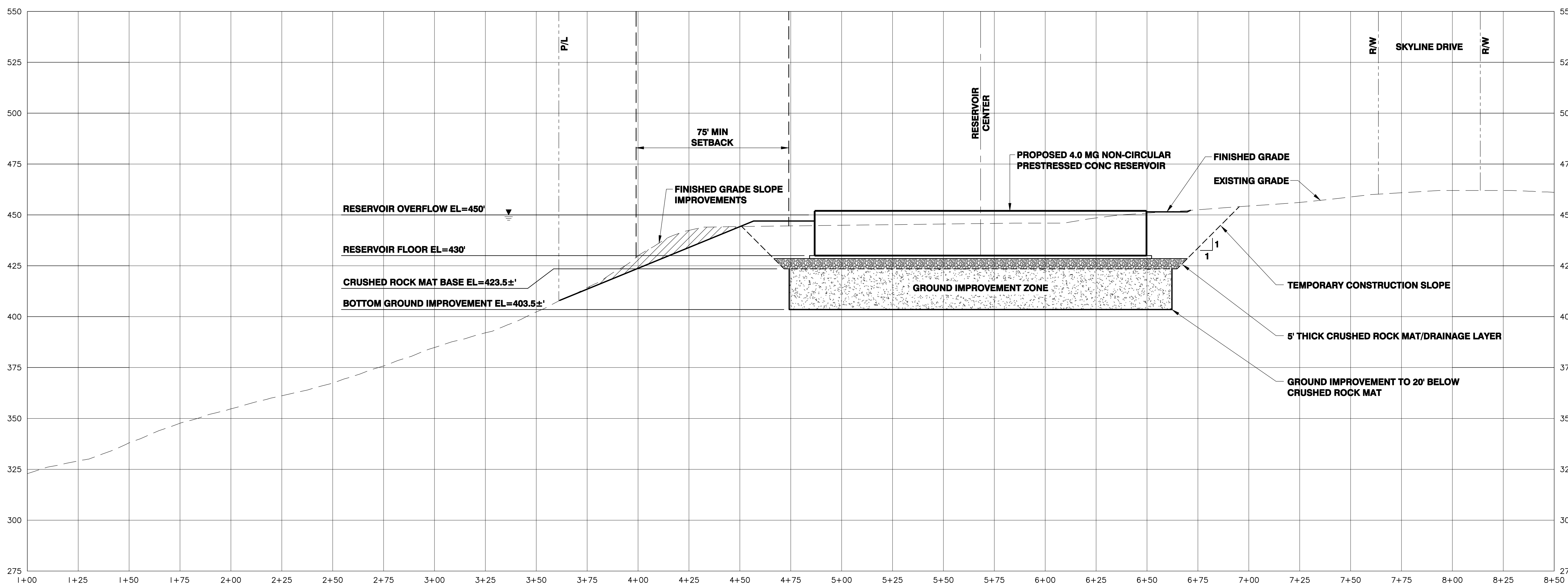
City of West Linn

City of West Linn
4.0 MG Bolton Reservoir Siting
Conceptual Siting Analysis

Section A
Non-circular Reinforced Concrete Reservoir Alternative

August 2012

MSA Murray, Smith & Associates, Inc.
Engineers/Planners
121 S.W. Salmon, Suite 900 Portland, Oregon 97204
PHONE 503-225-9010 FAX 503-225-9022



SECTION A

SCALE: 1"=50' HORIZ & VERT

MSA

ATTACHMENTS

MSA

ATTACHMENT 1
COST FOR CIRCULAR RESERVOIR

Attachment 1

CITY OF WEST LINN
4.0 MG Bolton Reservoir Project
Conceptual Siting Analysis

Preliminary Project Cost Estimate
Circular Prestressed Concrete Reservoir Alternative

Item	Description		Cost
1.	Mobilization, Bonds and Insurance	$	293,000
2.	Construction Surveying and Staking	$	5,000
3.	Tree Removal, Clearing and Grubbing	$	20,000
4.	Demolition of Existing 2.5 MG Bolton Reservoir	$	150,000
5.	Site Preparation, Excavation, Backfill and Grading & Erosion Control (Includes Overexcavation for Unsuitable Foundation Materials)	$	1,080,000
6.	Subsurface Ground Improvements (Aggregate Piers)	$	700,000
7.	4.0 MG Prestressed Concrete Reservoir Complete w/ Interior Piping, Foundation & Appurtenances	$	3,250,000
8.	Miscellaneous On-Site Waterline & Storm Drain Piping and Accessories (Includes Flexible Expansion Joints at Reservoir)	$	130,000
9.	Electrical, Instrumentation Control and Telemetry	$	25,000
10.	Reservoir Testing, Disinfection, Start-up and Clean-up	$	5,000
11.	Slope Stability Improvements North of Reservoir	$	75,000
12.	Surface Restoration & Landscaping	$	20,000
13.	Reservoir Bypass Facilities	$	50,000
14.	Demolition of Old Bolton Pump Station	$	25,000
15.	Abandonment of 14-Inch Dia. Waterline from Reservoir to Skyline Drive	$	2,000
16.	PRV Vault & New 8-Inch Ductile Iron (DI) Dia. Skyline Circle Waterline	$	95,000
17.	Replacement of 18-Inch Dia. Waterline in Skyline Circle with 24-Inch Dia. DI Waterline, Approx. 700 LF	$	160,000
18.	Replacement of 6-Inch Dia. PVC Waterline from Reservoir to Barclay Street with 8-Inch Dia. DI Waterline	$	50,000

Subtotal Estimated Construction Cost(1)	$6,135,000
+ 20% Construction Contingency	$1,227,000
Total Estimated Construction Cost	$7,362,000
+ 20% Allowance for Engineering, Permitting, Construction Management and City Administration	$1,473,000
Grand Total Project Cost	**$8,835,000**

Footnote:

1. This preliminary construction cost estimate is an opinion of cost based on information available at the time of the estimate. Final costs will depend on actual field conditions, actual material and labor costs, market conditions for construction, regulatory factors, final project scope, method of implementation, schedule and other variables. The Engineering (ENR) Construction Cost Index for Seattle, Washington is 9061.73 (August 2012).

ATTACHMENT 2
COST FOR NON-CIRCULAR RESERVOIR

Attachment 2

CITY OF WEST LINN
4.0 MG Bolton Reservoir Project
Conceptual Siting Analysis

Preliminary Project Cost Estimate
Non-circular Reinforced Concrete Reservoir Alternative

Item	Description		Cost
1.	Mobilization, Bonds and Insurance	$	306,000
2.	Construction Surveying and Staking	$	5,000
3.	Tree Removal, Clearing and Grubbing	$	20,000
4.	Demolition of Existing 2.5 MG Bolton Reservoir	$	150,000
5.	Site Preparation, Excavation, Backfill and Grading & Erosion Control (Includes Overexcavation for Unsuitable Foundation Materials)	$	980,000
6.	Subsurface Ground Improvements (Aggregate Piers)	$	775,000
7.	4.0 MG Prestressed Concrete Reservoir Complete w/ Interior Piping, Foundation & Appurtenances	$	3,550,000
8.	Miscellaneous On-Site Waterline & Storm Drain Piping and Accessories (Includes Flexible Expansion Joints at Reservoir)	$	130,000
9.	Electrical, Instrumentation Control and Telemetry	$	25,000
10.	Reservoir Testing, Disinfection, Start-up and Clean-up	$	5,000
11.	Slope Stability Improvements North of Reservoir	$	75,000
12.	Surface Restoration & Landscaping	$	20,000
13.	Reservoir Bypass Facilities	$	50,000
14.	Demolition of Old Bolton Pump Station	$	25,000
15.	Abandonment of 14-Inch Dia. Waterline from Reservoir to Skyline Drive	$	2,000
16.	PRV Vault & New 8-Inch Ductile Iron (DI) Dia. Skyline Circle Waterline	$	95,000
17.	Replacement of 18-Inch Dia. Waterline in Skyline Circle with 24-Inch Dia. DI Waterline, Approx. 700 LF	$	160,000
18.	Replacement of 6-Inch Dia. PVC Waterline from Reservoir to Barclay Street with 8-Inch Dia. DI Waterline	$	50,000

Subtotal Estimated Construction Cost[(1)]	$6,423,000
+ 20% Construction Contingency	$1,285,000
Total Estimated Construction Cost	$7,708,000
+ 20% Allowance for Engineering, Permitting, Construction Management and City Administration	$1,542,000
Grand Total Project Cost	***$9,250,000***

Footnote:

1. This preliminary construction cost estimate is an opinion of cost based on information available at the time of the estimate. Final costs will depend on actual field conditions, actual material and labor costs, market conditions for construction, regulatory factors, final project scope, method of implementation, schedule and other variables. The Engineering (ENR) Construction Cost Index for Seattle, Washington is 9061.73 (August 2012).

MSA

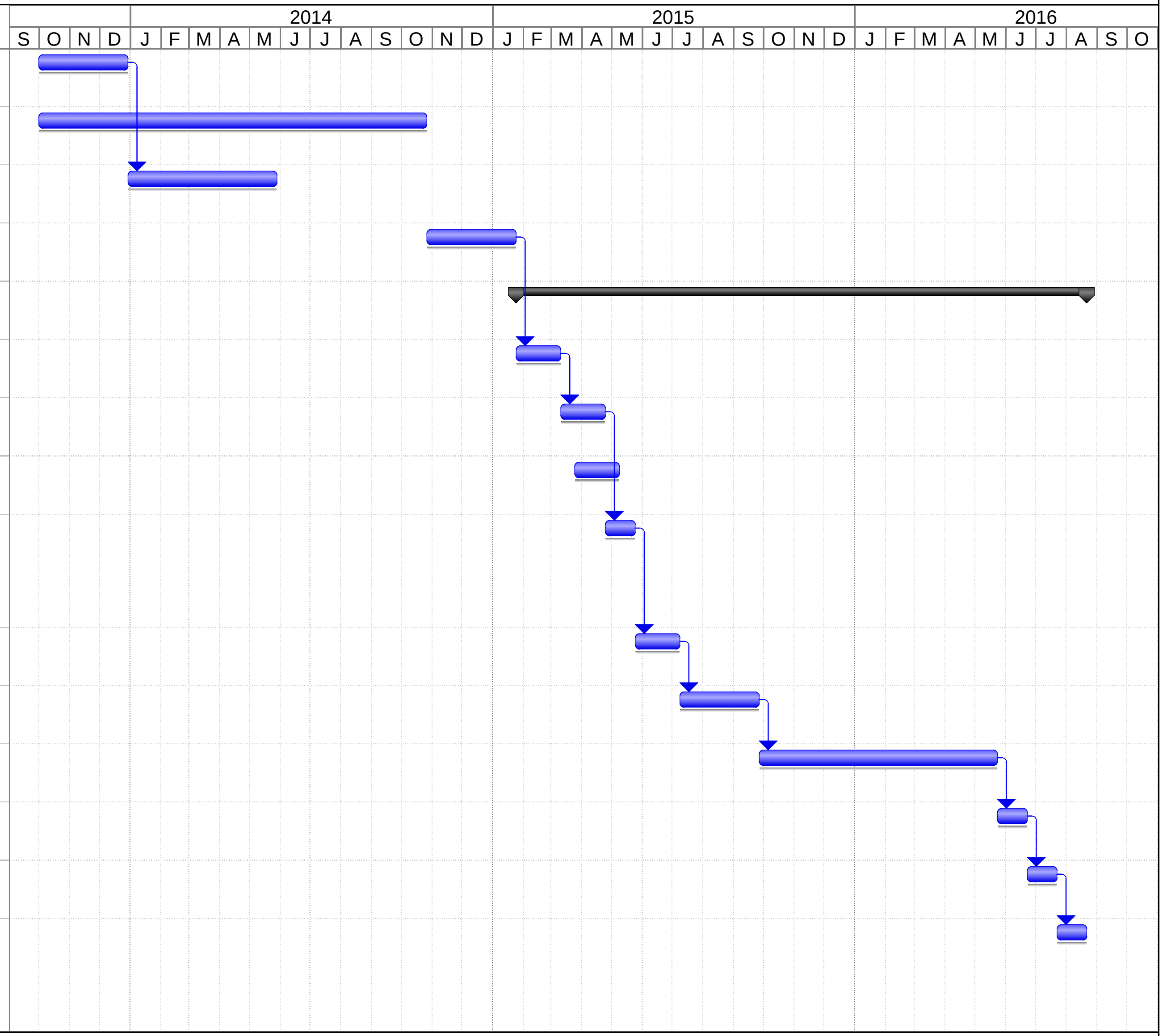
ATTACHMENT 3
PRELIMINARY PROJECT SCHEDULE

CITY OF WEST LINN
4.0 MG BOLTON RESERVOIR PROJECT
CONCEPTUAL SITING ANALYSIS

PRELIMINARY PROJECT SCHEDULE

Task Name

- Preliminary Engineering
- Permitting
- Final Design
- Bidding, Award & NTP
- Construction Phase
  - Submittal & Shop Drawing Review
  - Mobilization & Reservoir Bypass Facilities
  - Off-site Piping
  - Demolition of Existing 2.5 MG Reservoir & Old Bolton Pump Station
  - Site Preparation & Mass Excavation
  - Ground Improvements
  - 4.0 MG Reservoir Construction
  - Reservoir Backfill & On-site Piping
  - Finished Grading & Slope Improvement
  - Final Surface Restoration, Roadway Construction & Landscaping



Attachment 3

APPENDIX

APPENDIX A

GRI
9725 SW Beaverton-Hillsdale Hwy, Suite 140
Beaverton, OR 97005-3364
p| 503-641-3478 f| 503-644-8034

August 31, 2012 5338 GEOTECHNICAL RPT

Murray, Smith & Associates, Inc.
121 SW Salmon Street, Suite 900
Portland, OR 97204

Attention: Tom Boland, PE

SUBJECT: Preliminary Geotechnical Evaluation for Conceptual Siting Analysis
4-MG Bolton Reservoir
West Linn, Oregon

At your request, GRI has completed a preliminary geotechnical evaluation to support the conceptual siting analysis for the planned 4-MG Bolton Reservoir. This evaluation focused on the site of the existing Bolton Reservoir at the approximate location shown on the Vicinity Map, Figure 1.

The purpose of the geotechnical evaluation is to provide our conclusions and preliminary recommendations for siting a new reservoir on the site. Our work included review of existing geotechnical and geologic information; geologic reconnaissance; drilling, logging and sampling of one boring; limited laboratory testing; geologic and engineering analyses; discussions and meetings with Murray, Smith, & Associates, Inc.; and preparation of this report.

PROJECT DESCRIPTION

The City of West Linn (City) is considering construction of a new 4-MG concrete reservoir at the site of the existing open Bolton Reservoir. The two alternatives currently being considered include a 190-ft-diameter concrete reservoir and a near-rectangular concrete reservoir. For both alternatives, the floor would be established at elevation 430 ft and the overflow at elevation 450 ft. The preliminary configurations of the two alternatives are shown on Attachments 1 through 4. As shown, significant excavation will be required to establish either alternative.

SITE DESCRIPTION

Topography

As shown on the Site Map, Figure 2, the proposed reservoir site is located northeast of Skyline Drive at the location of the existing reservoir. The land use in the area surrounding the existing reservoir consists of forested undeveloped land and residential properties. The ground surface slopes down from south to north, based on the topographic information shown on Attachments 1 and 3, and the cross sections on Attachments 2 and 4.

Geology

The site is located on the flanks of the Tualatin Mountains, which form a topographic high point separating the Portland Basin to the northeast from the Tualatin Basin to the west and the Willamette Valley to the south. Geologic mapping completed for the area indicates the site is located in the vicinity of the contact between the Wanapum Basalt and the Grande Ronde Basalt units of the Columbia River Basalt Group

(Madin, 2009). Where fresh and unweathered, these basalt units are typically a light to dark gray, dense volcanic rock. However, the Wanapum-Grande Ronde boundary is characterized in places by an erosional unconformity or an interbed that varies from non-marine sediments to a thick relic soil, and is referred to as the Vantage Horizon (Beeson, et al., 1985). The Vantage Horizon originated during a period of erosion and soil development that occurred between volcanic flow events. Large-scale landsliding is associated at nearby locations where the Vantage Horizon daylights at or near the ground surface.

Several geologic faults are located in the project area. The Bolton Fault is located about 1,000 ft northeast of the site, the Oatfield Fault is about 2.5 miles northeast of the site, and the Portland Hills Fault is about 3 miles northeast of the site (Personius, et al., 2002). These faults do not have historic seismicity, but are considered potentially active. The Cascadia Subduction Zone (CSZ) is located approximately 80 miles west of the site.

SLOPE STABILITY

Previous Reports

Three engineering reports prepared for the Bolton Reservoir site in 1972, 1988, and 1998, were provided to GRI. The first report was prepared by Northwest Testing Laboratories (NTL) for the City in 1972 (NTL, 1972). The report presented the results of a soils and foundation investigation site and provided recommendations for enlargement of the reservoir. The report concluded the slope east of the site could accommodate the additional load of the reservoir.

L.R. Squier Associates, Inc. prepared a geologic reconnaissance report for the City in 1988 (L.R. Squier, 1988). The purpose of the report was to evaluate the slope northeast of the reservoir for a planned residential development where there were concerns of slope stability. The report concluded that steep slopes, weak and locally thin soils, soil creep, and groundwater seepage from springs suggested a high risk for slope instability, and a comprehensive geotechnical investigation was recommended.

In the 1970s, a small earth flow failure landslide occurred at the site along the steeply sloping wooded area northeast of the reservoir. Large ground cracks occurred south of the reservoir in 1996 following heavy rainfall. Landslide Technology conducted an investigation into the stability of the steep slope area in 1997 (Landslide Technology, 1998). The investigation included a reconnaissance, field explorations, laboratory testing, installation of an open pipe piezometer and inclinometer casing. Based on the results of the investigation, the report provided an approach for repair of the landslide.

Oregon Department of Geology and Mineral Industries Landslide Mapping

The Oregon Department of Geology and Mineral Industries (DOGAMI) is the state agency responsible for geologic hazard mapping for the State of Oregon. DOGAMI has indicated in its statewide landslide hazard database that the Bolton Reservoir is located on a prehistoric (>150 years), deep-seated (>15 ft depth), translational rock landslide, referred to as *Canby 133*. Attachment 5 shows the limits of the landslide from the state database. The mapping of landslide deposits are based in part on light detection and ranging (LiDAR) derived elevation data and interpretation of surface topography typical of landslide features. *Canby 133* was mapped using LiDAR and a method protocol outlined by DOGAMI (DOGAMI, 2009) with a 'moderate' level of confidence. The confidence ranking (low, moderate, high) is developed based on desktop work. Bill Burns with DOGAMI was contacted regarding this feature and recalls they did do a vehicle-based reconnaissance from public roads for this feature, but he was not aware of any more data (i.e., reports, borings, anecdotal stories of ground movement) about the feature. He indicated unpublished DOGAMI field mapping from 2004 also indicates the area is a landslide. This information indicates the Bolton Reservoir site is located on a very large, old, or "ancient," landslide.

SITE RECONNAISSANCE

A reconnaissance of the site and surrounding area was conducted by a registered geologist and a certified engineering geologist from GRI in June 2012. The following description of the site is a summary of the observations made during site reconnaissance activities. Private properties located immediately northwest and southeast of the site were not accessed, but observed from the public right-of way for any features of significance. To the northeast, the ground slopes downward at approximately 25° toward Caufield Street. The slope northeast of the reservoir site is wooded with predominantly deciduous trees and occasional conifer tree, and springs. The ground surface is generally covered by English ivy, ferns, and blackberries. Several springs and flowing water were also observed along Caufield Street coming from the slope above. A concrete manhole and pipe valve were observed along the slope near the northern property boundary. The valve appeared to be rusted through and was leaking water. No indications of recent slope instability were observed along the northeast slope during the site reconnaissance. The surrounding neighborhood was also examined from the public right-of-way for indications of slope movement (cracked and separated sidewalks or curbs). The reconnaissance did not disclose obvious indications of relatively recent movement, such as cracked streets, sidewalks, or curbs. Limited interviews with City maintenance personnel did not disclose reports of broken or sheared underground utilities.

The slope failure that occurred in 1996 along the northeast side of the existing reservoir has not been repaired and is essentially covered now with vegetation. Most of the remainder of the slope along the north side of the reservoir has the same general appearance and inclination of the slope adjacent to the landslide. The current reservoir was fully covered with a liner, and could not be examined. However, cracking is present along portions of the north side of the reservoir flatwork and ring wall, particularly in the northwest corner. Whether the flatwork and ring wall cracking is due to slope movement or fill settlement could not be ascertained.

Inclinometer

In June 2012, GRI obtained measurements in the inclinometer that was installed by Landslide Technologies in 1997 at the approximate location shown on Figure 2. An inclinometer casing consists of a plastic pipe with a pair of orthogonal slots, or grooves, that permit a calibrated instrument to be lowered to the bottom of the casing. When the ground surrounding the casing moves, the casing distorts above the zone of movement, and the orientation of the casing changes. The orientation of the casing is measured by lowering the calibrated instrument to the bottom of the casing and reading the instrument at 2-ft intervals as it is withdrawn. The zone and rate of movement can be determined by comparing the results of successive sets of readings. The inclinometer was installed east of the proposed tank footprint to provide long-term monitoring of the site with respect to potential slope movement.

GRI obtained the baseline measurements collected by Landslide Technologies in 1998 and compared those data with measurements collected from the inclinometer by GRI on June 15, 2012. Our comparison of the measurements indicates the upper 5 ft of soil in the vicinity of the inclinometer had moved approximately 6 in. since the 1998 readings were collected. In addition, potential movement at a depth of

28 ft may be interpreted from the inclinometer readings. However, movement appeared to be less than 0.125 in.

SUBSURFACE CONDITIONS

General

Subsurface information previously obtained by NTL and Landslide Technologies along the north side of the reservoir was supplemented with one boring, designated B-1, completed by GRI on June 15, 2012. Boring B-1 was advanced to a depth of 76 ft at the location shown on Figure 2. The field exploration program completed for this study is discussed in detail in Appendix A. The log of the boring is provided on Figure 1A. The terms used to describe the materials encountered in the boring are defined in Tables 1A and 2A.

Subsurface Materials

The materials disclosed by boring B-1 have been grouped into the following categories:

1. **SILT FILL**
2. **Clayey SILT (Possible Landslide Debris)**
3. **BASALT (Columbia River Basalt)**

1. SILT (FILL). Silt fill was encountered at the ground surface and extended to a depth of 8 ft. The silt is brown and contains a trace of sand that is typically fine grained. An N-value of 3 blows/ft for a sample from a depth of about 3.5 to 5 ft indicates the relative consistency of the silt is soft. The natural moisture content of the sample is 30%.

2. Clayey SILT. Clayey SILT was encountered between a depth of 8 and 60 ft in boring B-1. The silt ranges from dark brown to gray and contains scattered sand- and gravel-size fragments of decomposed rock. N-values ranging from 5 to 29 blows/ft indicate the relative consistency of the material ranges from medium stiff to very stiff. The natural moisture content of the silt ranges from about 30 to 68%. This unit is interpreted to be possible landslide debris.

3. BASALT. Basalt rock was encountered between a depth of 60 and 76 ft (total depth of boring) in boring B-1. The rock consists of a hard, gray, vesicular basalt correlative with the Columbia River Basalt. The basalt was cored continuously; core recovery was 100%, and rock quality designation ranged from about 45 to 88%. This unit is interpreted as intact, basalt bedrock that is below the potential depth of past landslide processes.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

General

Our limited, or preliminary, investigation indicates there are three significant factors associated with the placement of a new concrete reservoir on the Bolton Reservoir site.

1) The reservoir site is located on a very large, old, or "ancient" landslide, as mapped by DOGAMI.

2) Landsliding has occurred on the slope at the northeast corner of the reservoir. Associated ground cracking occurred very near the reservoir following heavy rainfall in 1996.

3) Borings made by GRI and others indicate that significant portions of the existing reservoir, particularly along the north side, are underlain by fill. The quality of the fill is unknown.

Ancient Landslide

The reservoir site is located on a very large landslide, as recently mapped by DOGAMI. The landslide was identified based primarily on landforms and topography, as revealed by LiDAR imagery. DOGAMI considers the landslide to have occurred at least 150 years ago and possibly much longer. DOGAMI is not aware of any significant movement of the landslide in modern times. The landslide is likely associated with the Vantage Horizon, which is a soil-like zone interbedded between two basalt flows. Other large landslides in the West Linn area are also associated with the Vantage Horizon.

A reconnaissance by GRI did not disclose indications of relatively recent movement, such as cracked streets, sidewalks, or curbs. Our inquiries also did not disclose reports of broken or sheared underground utilities, which could also indicate ground movement.

It is our overall opinion that the risk of significant future movement of the large, old landslide mass that could impact a reservoir on the site is probably low, but the risk is not totally absent. Due to the large size of the mapped landslide, we anticipate that mitigation measures to improve the stability of the landslide are likely not practical or cost effective.

Localized Landsliding

A landslide occurred on the slope at the northeast corner of the existing corner of the existing reservoir sometime in the 1970s. Extensive significant ground cracking occurred following heavy rainfall in 1996 and extended to near the reservoir. Based on our reconnaissance and review of the available topographic information, the undisturbed slopes along much of the north side of the existing reservoir are similar to the slopes at the landslide and likely also have a relatively low factor of safety against landslide. The risk to a new reservoir by local landsliding can be significantly reduced by setting the reservoir back from the slope. In this regard, we recommend establishing the reservoir at least 75 ft south of the point at which a horizontal plane at elevation 430 ft (floor level) intercepts the slope.

In addition, the stability of the slope can be improved by removing material at the top of the slope and south of the reservoir footprint. Improving the stability of the slope will reduce the risk of future landsliding or slope movement that could affect the new reservoir.

Existing Fill

The exploration borings made by GRI and others indicate the presence of significant thicknesses of fill along the north side of the reservoir. The fill likely underlies at least portions of the existing reservoir and footprint of the proposed reservoir. The quality of the fill is unknown; however, there is significant cracking of concrete features along the northwest portion of the existing reservoir. The cracking could be associated with settlement of the fill. The presence of existing poorly compacted or compressible fill beneath the new reservoir could result in excessive and unacceptable total and differential settlement.

For this reason, it would be reasonable to assume for preliminary budgeting purposes that ground improvement such as 20-ft-deep rammed aggregate piers will be needed beneath at least half of the new reservoir footprint, with overexcavation and a minimum 5-ft thickness of crushed rock beneath the remainder of the reservoir. A more conservative approach would be to assume that rammed aggregate piers will be needed beneath the entire footprint.

Following suitable subgrade preparation, we anticipate reservoir footings could be designed to impose a bearing pressure of at least 3,000 psf that would be in addition to the pressure imposed by the water column.

Seismic Considerations

As discussed previously, three potentially active faults are located near the site. As a result, ground motions during a design earthquake will be larger than for sites located further from the faults. Based on the available subsurface information, the site class is likely D or E. However, a site-specific seismic hazard study will be necessary to develop criteria for design and evaluation of the reservoir.

LIMITATIONS

This preliminary report has been prepared to aid in the evaluation and conceptual design of this project. The findings presented herein are based on the data obtained from the boring made at the locations indicated on Figure 2 and from other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in soil and rock conditions may exist between boring locations. The scope of our investigation was limited by the fact that actual plans for the reservoir are indefinite; hence, only preliminary opinions are presented. Significant limitations are inherent in a study of this type, and additional site investigations should be conducted as specific plans and designs are developed. The information provided in this report is not intended for final design of the project. A more detailed geotechnical investigation, including additional subsurface explorations, laboratory testing, and engineering analyses will be necessary to develop criteria and guidelines for final design.

Submitted for GRI,

REGISTERED PROFESSIONAL ENGINEER 51358PE OREGON July 9, 2001 KEITH S. MARTIN Renews 12/2012

CERTIFIED OREGON GEORGE A. FREITAG E1380 ENGINEERING GEOLOGIST

Expires 1/2013

Dwight J. Hardin, PE, GE	Keith S. Martin, PE, GE	George A. Freitag, CEG
Principal	Project Engineer	Associate

This document has been submitted electronically.

References

Beeson, M.H., Fecht, K.R., Reidel, S.P., Tolan, T.L., August 1985, Regional correlations within the Frenchman Springs Member of the Columbia River Basalt Group - new insights into the middle Miocene tectonics of northwestern Oregon: Oregon Geology, vol. 47, no. 8.

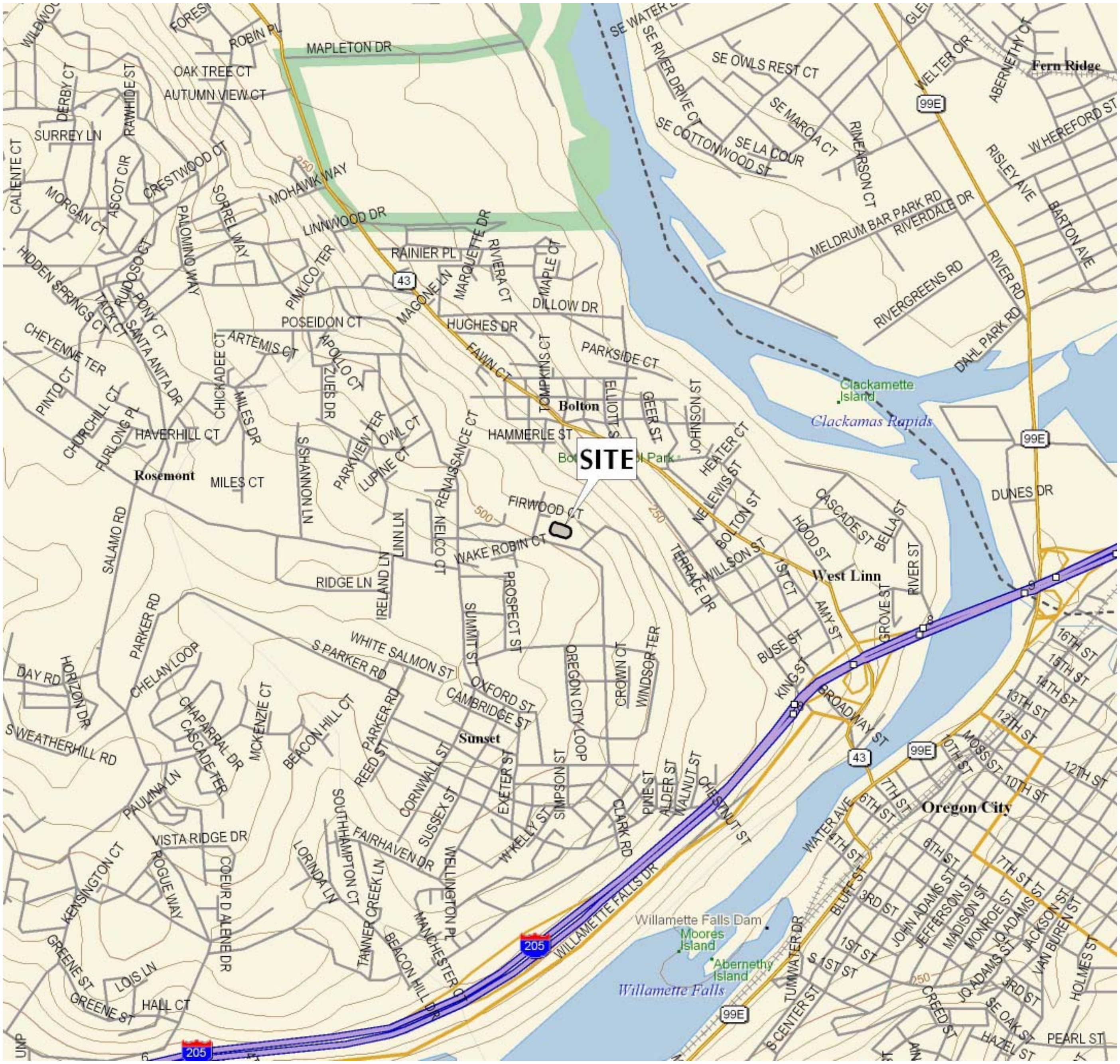
L. R. Squier Associates, Inc., 1988, Geologic reconnaissance, property downslope of 2.5 MG Skyline (Bolton) reservoir, West Linn, Oregon; prepared for City of West Linn.

Landslide Technology, June 19, 1998, Revised report, Phases I and II geotechnical investigation, Bolton Reservoir landslide, West Linn, Oregon; prepared for Montgomery Watson.

Madin, I. P, 2009, Geologic map of the Oregon City 7.5' quadrangle, Clackamas County, Oregon: Oregon Department of Geology and Mineral Industries, GMS-119; scale 1:24,000.

Northwest Testing Laboratories, October 11, 1972, Soils and foundation investigation, City of West Linn, c/o John Cunningham and Associates, City Reservoir, West Linn, Oregon; prepared for City of West Linn.

Personius, S.F., compiler, 2002, Fault numbers 874, 875, 877, in Quaternary fault and fold database of the United States: U.S. Geological Survey website http://earthquakes.usgs.gov/regional/qfaults, accessed 08/03/2012.



DELORME 3-D TOPOQUADS, OREGON
OREGON CITY, OREG. (2bb) 2004

North

0 1/2 1 MILE

GRI

MURRAY, SMITH & ASSOCIATES, INC.
BOLTON RESERVOIR

VICINITY MAP

AUG. 2012 JOB NO. 5338 FIG. 1

● INCLINOMETER INSTALLED BY LANDSLIDE TECHNOLOGIES (1997)

◐ BORING MADE BY GRI (JUNE 15, 2012)

SITE MAP FROM AERIAL PHOTO BY BING IMAGE (UNDATED)

North

0 250 500 FT

GRI MURRAY, SMITH & ASSOCIATES, INC.
BOLTON RESERVOIR

SITE MAP

AUG. 2012 JOB NO. 5338 FIG. 2

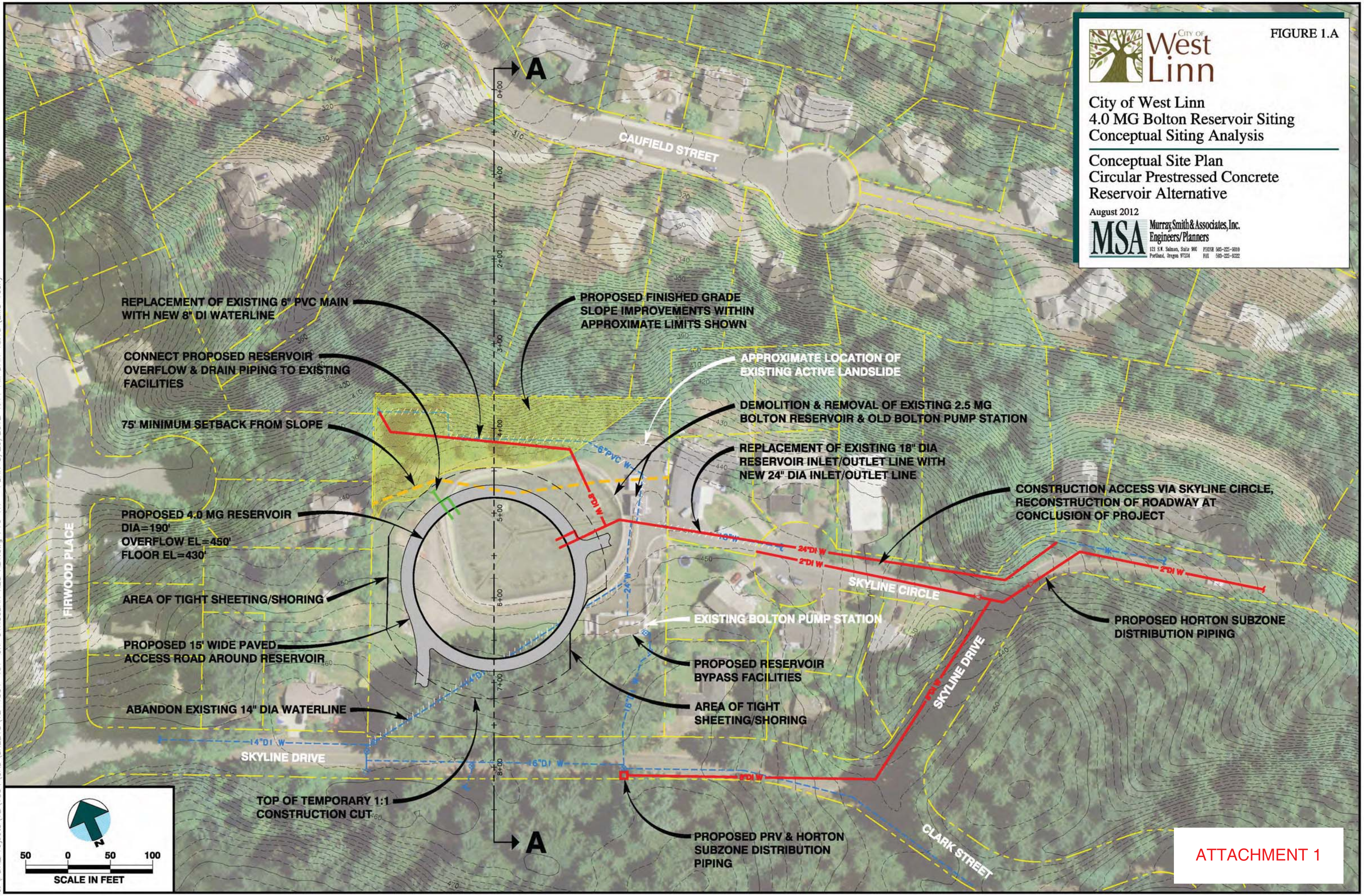


FIGURE 1.A
City of West Linn
City of West Linn
4.0 MG Bolton Reservoir Siting
Conceptual Siting Analysis
Conceptual Site Plan
Circular Prestressed Concrete
Reservoir Alternative
August 2012
MSA
Murray, Smith & Associates, Inc.
Engineers/Planners
CAUFIELD STREET
FIRWOOD PLACE
SKYLINE DRIVE
SKYLINE CIRCLE
CLARK STREET
A
A
REPLACEMENT OF EXISTING 6" PVC MAIN WITH NEW 8" DI WATERLINE
PROPOSED FINISHED GRADE SLOPE IMPROVEMENTS WITHIN APPROXIMATE LIMITS SHOWN
CONNECT PROPOSED RESERVOIR OVERFLOW & DRAIN PIPING TO EXISTING FACILITIES
APPROXIMATE LOCATION OF EXISTING ACTIVE LANDSLIDE
DEMOLITION & REMOVAL OF EXISTING 2.5 MG BOLTON RESERVOIR & OLD BOLTON PUMP STATION
75' MINIMUM SETBACK FROM SLOPE
REPLACEMENT OF EXISTING 18" DIA RESERVOIR INLET/OUTLET LINE WITH NEW 24" DIA INLET/OUTLET LINE
CONSTRUCTION ACCESS VIA SKYLINE CIRCLE, RECONSTRUCTION OF ROADWAY AT CONCLUSION OF PROJECT
PROPOSED 4.0 MG RESERVOIR
DIA=190'
OVERFLOW EL=450'
FLOOR EL=430'
AREA OF TIGHT SHEETING/SHORING
EXISTING BOLTON PUMP STATION
PROPOSED HORTON SUBZONE DISTRIBUTION PIPING
PROPOSED 15' WIDE PAVED ACCESS ROAD AROUND RESERVOIR
PROPOSED RESERVOIR BYPASS FACILITIES
ABANDON EXISTING 14" DIA WATERLINE
AREA OF TIGHT SHEETING/SHORING
TOP OF TEMPORARY 1:1 CONSTRUCTION CUT
PROPOSED PRV & HORTON SUBZONE DISTRIBUTION PIPING
8"DI W
6"PVC W
24"DI W
2"DI W
14"DI W
16"DI W
50 0 50 100
SCALE IN FEET
N
ATTACHMENT 1

FIGURE 1.B

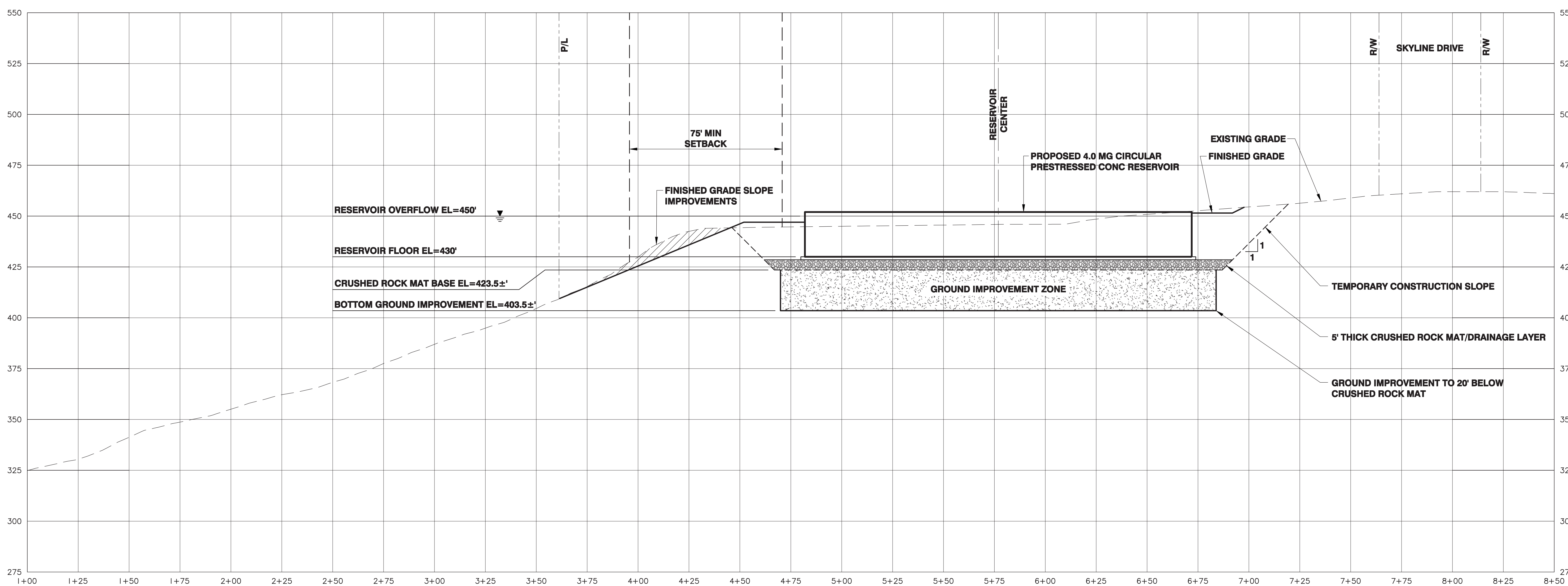
City of West Linn

City of West Linn
4.0 MG Bolton Reservoir Siting
Conceptual Siting Analysis

Section A
Circular Prestressed Concrete
Reservoir Alternative

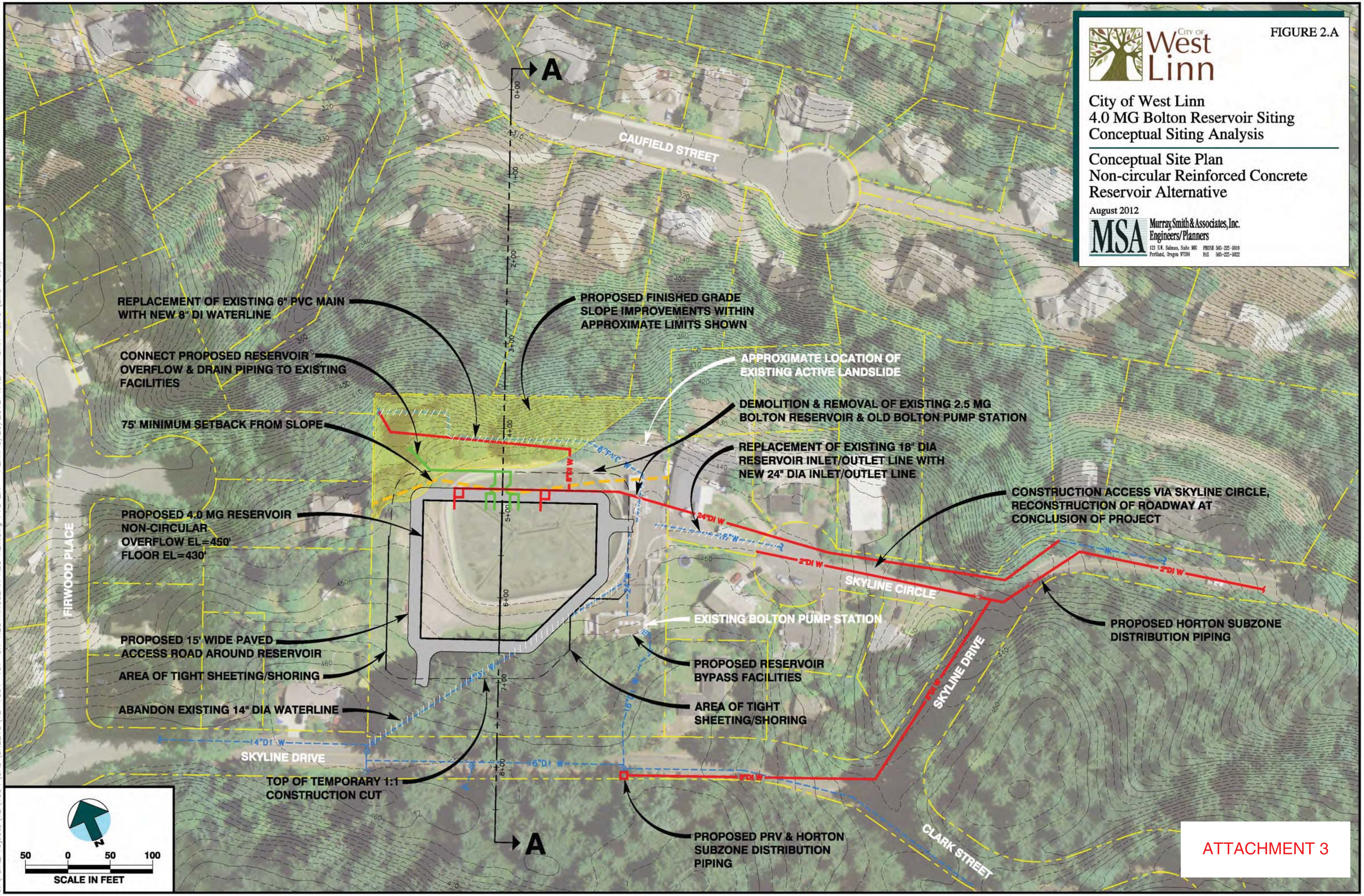
August 2012

MSA Murray, Smith & Associates, Inc.
Engineers/Planners
121 S.W. Salmon, Suite 900 Portland, Oregon 97204 PHONE 503-225-9010 FAX 503-225-9022



SECTION A
SCALE: 1"=50' HORIZ & VERT

ATTACHMENT 2



ATTACHMENT 3

G:\PDX_Projects\12\1334\CAD\SHEETS\12-1334-401-OR-RECTANGULAR RES ALT.dwg FIG 2A -- PLAN 8/29/2012 1:12 PM DAK 18.1s (LMS Tech)

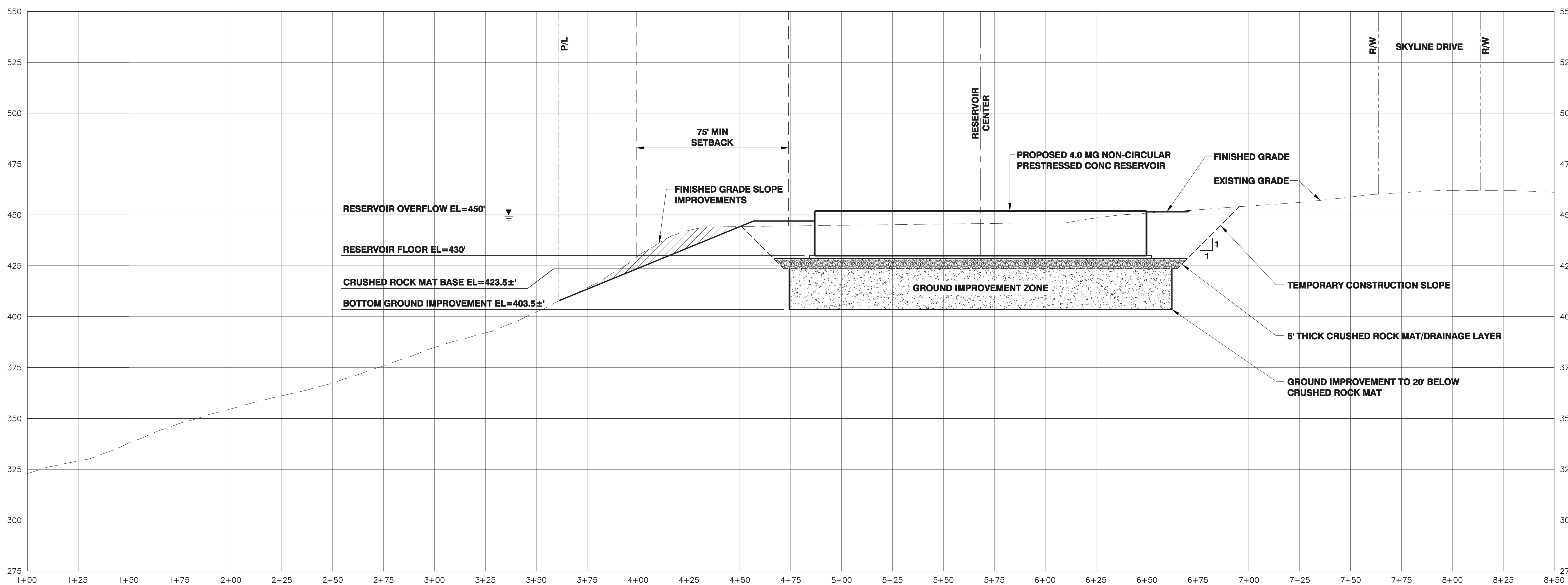
FIGURE 2.B

City of West Linn
4.0 MG Bolton Reservoir Siting
Conceptual Siting Analysis

Section A
Non-circular Reinforced Concrete Reservoir Alternative

August 2012

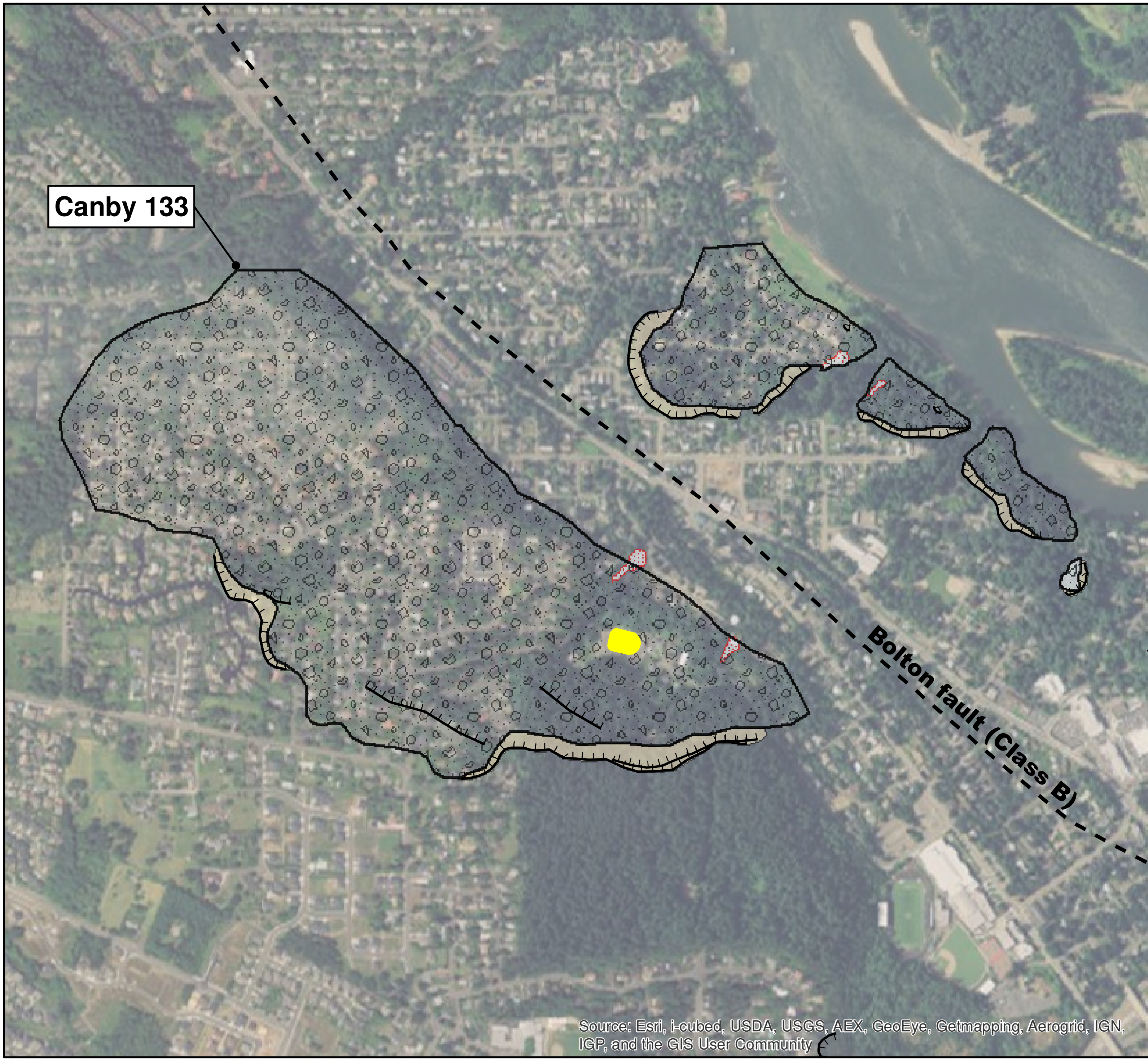
Murray, Smith & Associates, Inc.
Engineers/Planners
121 S.W. Salmon, Suite 900 Portland, Oregon 97204
PHONE 503-225-9010 FAX 503-225-9022



SECTION A
SCALE: 1"=50' HORIZ & VERT

ATTACHMENT 4

G:\PDX_Projects\12\1334\CAD\SHEETS\12-1334-401-OR-RECTANGULAR RES ALT.dwg FIG 2B - SECTION A 8/29/2012 1:12 PM DAK 18.1s (LMS Tech)



Attachment 5
Bolton Reservoir
West Linn

Translational Landslide
Moderate Confidence
Prehistoric >150 yrs
Deep Seated > 15 ft

Bolton Reservoir
Bolton Fault
Debris Flow
Landslide Deposits
Scarps
Scarp Flanks

Data Source:
-Interpertive Map Series 29 (IMS 29) Landslide Inventory Maps for the Canby Quadrangle, Clackamas, Marion, and Washington Counties, Oregon. Oregon Department of Geology and Mineral Industries.
-USGS Quaternary fault and fold database of the United States

NORTH
0 1,000 Ft

APPENDIX A

FIELD EXPLORATIONS

FIELD EXPLORATIONS

Subsurface materials and conditions for the planned reservoir were evaluated by GRI on June 15, 2012, with one boring, designated B-1. The location of the boring is shown on Figure 2. The exploration was observed by a geologist from GRI.

The boring was advanced to a depth of 76 ft with mud-rotary drilling methods using a CME 75 track-mounted drill rig provided and operated by Western States Soil Conservation, Inc., of Aurora, Oregon. Disturbed and undisturbed samples were obtained from the boring at about 2.5- to 5-ft intervals of depth. Disturbed samples were obtained using a standard split-spoon sampler. At the time of sampling, the Standard Penetration Test was conducted. This test consists of driving a standard split-spoon sampler into the soil a distance of 18 in. using a 140-lb hammer dropped 30 in. The number of blows required to drive the sampler the last 12 in. is known as the Standard Penetration Resistance, or N-value. The N-values provide a measure of the relative density of granular soils and the relative consistency of cohesive soils. The soil samples obtained in the split-spoon sampler were examined in the field, and representative portions were saved in airtight jars for further examination and physical testing in our laboratory.

The log of the boring is provided on Figure 1A. The log presents a descriptive summary of the various types of materials encountered in the boring and notes the depth where the materials and/or characteristics of the materials change. To the right of the descriptive summary, the numbers and types of samples taken during the drilling operation are indicated. Farther to the right, N-values are shown graphically, along with the natural moisture contents and Torvane shear strength values. The terms used to describe the soil encountered in the borings are defined in Tables 1A and 2A.

Table 1A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance (N-values) blows per foot
very loose	0 - 4
loose	4 - 10
medium dense	10 - 30
dense	30 - 50
very dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soil

Consistency	Standard Penetration Resistance (N-values) blows per foot	Torvane or Undrained Shear Strength, tsf
very soft	0 - 2	less than 0.125
soft	2 - 4	0.125 - 0.25
medium stiff	4 - 8	0.25 - 0.50
stiff	8 - 15	0.50 - 1.0
very stiff	15 - 30	1.0 - 2.0
hard	over 30	over 2.0

Sandy silt materials which exhibit general properties of granular soils are given relative density description.

Grain-Size Classification

Boulders
12 - 36 in.

Cobbles
3 - 12 in.

Gravel
1/4 - 3/4 in. (fine)
3/4 - 3 in. (coarse)

Sand
No. 200 - No. 40 sieve (fine)
No. 40 - No. 10 sieve (medium)
No. 10 - No. 4 sieve (coarse)

Silt/Clay - pass No. 200 sieve

Modifier for Subclassification

Adjective	Percentage of Other Material In Total Sample
clean	0 - 2
trace	2 - 10
some	10 - 30
sandy, silty, clayey, etc.	30 - 50

Table 2A

GUIDELINES FOR CLASSIFICATION OF ROCK

RELATIVE ROCK WEATHERING SCALE:

Term	Field Identification
Fresh	Crystals are bright. Discontinuities may show some minor surface staining. No discoloration in rock fabric.
Slightly Weathered	Rock mass is generally fresh. Discontinuities are stained and may contain clay. Some discoloration in rock fabric. Decomposition extends up to 1 in. into rock.
Moderately Weathered	Rock mass is decomposed 50% or less. Significant portions of rock show discoloration and weathering effects. Crystals are dull and show visible chemical alteration. Discontinuities are stained and may contain secondary mineral deposits.
Predominantly Decomposed	Rock mass is more than 50% decomposed. Rock can be excavated with geologist's pick. All discontinuities exhibit secondary mineralization. Complete discoloration of rock fabric. Surface of core is friable and usually pitted due to washing out of highly altered minerals by drilling water.
Decomposed	Rock mass is completely decomposed. Original rock "fabric" may be evident. May be reduced to soil with hand pressure.

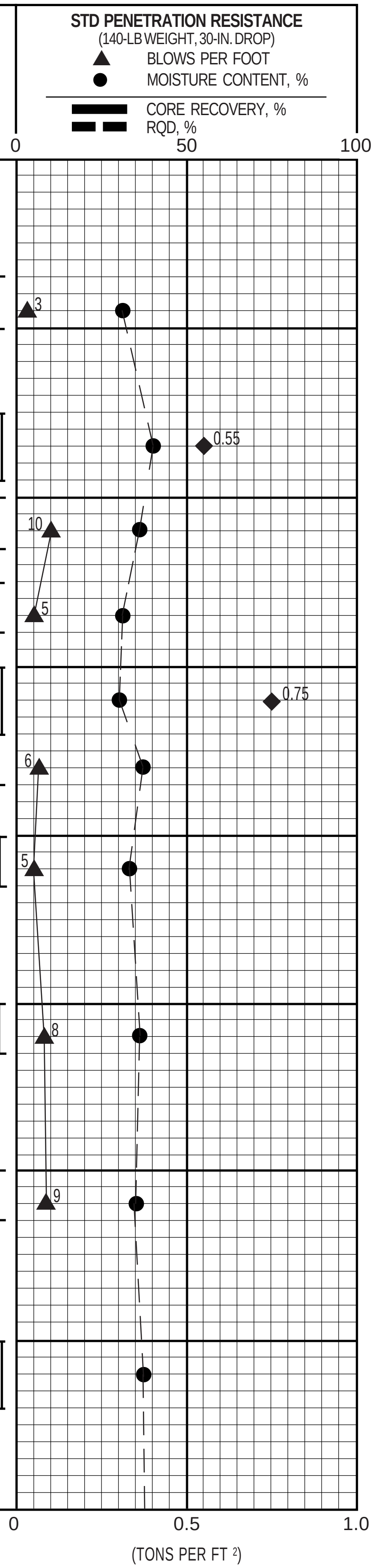
RELATIVE ROCK HARDNESS SCALE:

Term	Hardness Designation	Field Identification	Approximate Unconfined Compressive Strength
Extremely Soft	R0	Can be indented with difficulty by thumbnail. May be moldable or friable with finger pressure.	< 100 psi
Very Soft	R1	Crumbles under firm blows with point of a geology pick. Can be peeled by a pocket knife and scratched with fingernail.	100 - 1,000 psi
Soft	R2	Can be peeled by a pocket knife with difficulty. Cannot be scratched with fingernail. Shallow indentation made by firm blow of geology pick.	1,000 - 4,000 psi
Medium Hard	R3	Can be scratched by knife or pick. Specimen can be fractured with a single firm blow of hammer/geology pick.	4,000 - 8,000 psi
Hard	R4	Can be scratched with knife or pick only with difficulty. Several hard hammer blows required to fracture specimen.	8,000 - 16,000 psi
Very Hard	R5	Cannot be scratched by knife or sharp pick. Specimen requires many blows of hammer to fracture or chip. Hammer rebounds after impact.	> 16,000 psi

RQD AND ROCK QUALITY:

Relation of RQD and Rock Quality		Terminology for Planar Surface		
RQD (Rock Quality Designation), %	Description of Rock Quality	Bedding	Joints and Fractures	Spacing
0 - 25	Very Poor	Laminated	Very Close	< 2 in.
25 - 50	Poor	Thin	Close	2 in. – 12 in.
50 - 75	Fair	Medium	Moderately Close	12 in. – 36 in.
75 - 90	Good	Thick	Wide	36 in. – 10 ft
90 - 100	Excellent	Massive	Very Wide	> 10 ft

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
		SURFACE ELEVATION 430 ft (±)			
		FILL: Crushed Rock	0.9		
		FILL: Soft, brown SILT; trace fine-grained sand, medium plasticity			
5					S-1
		Stiff, brown to dark brown, clayey SILT; some fine- to medium-grained sand, medium plasticity, slight reddish-brown mottling (Possible Landslide Debris)	8.0		S-2
10		----------medium stiff below 11.5 ft			S-3, S-4
15					S-5, S-6
20					S-7
25					S-8
30		Stiff, brown and orange-brown, clayey SILT; medium to low plasticity, trace sand- to gravel-size fragments of extremely soft (R0), predominantly decomposed basalt (Possible Landslide Debris)	30.0		S-9
35					S-10
40					



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
NX CORE RUN
SLOTTED PVC PIPE
Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

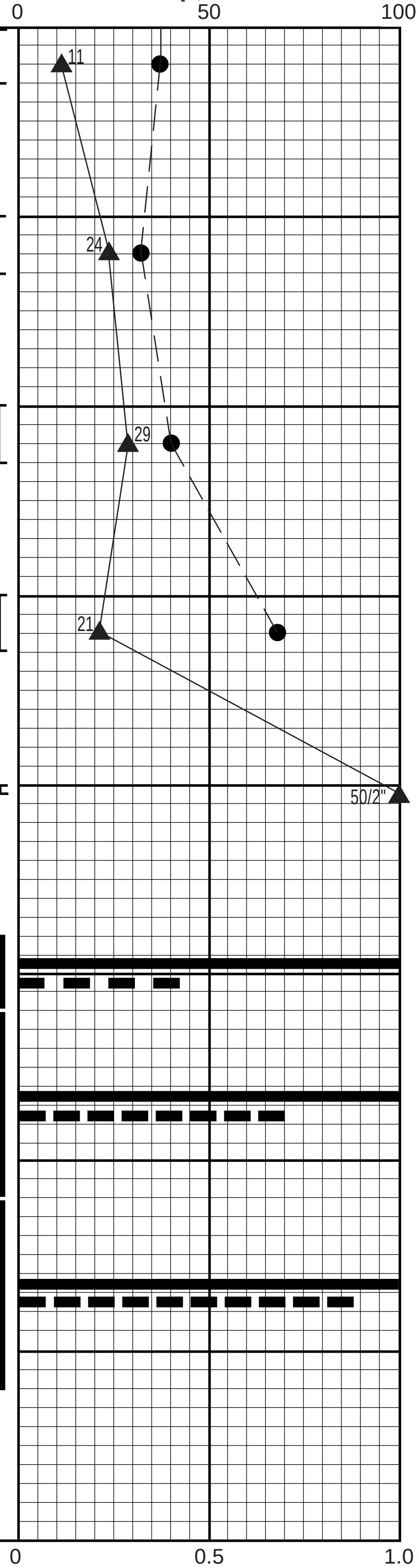
GRI

BORING B-1

AUG. 2012 JOB. NO. 5338 FIG. 1A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
40		SURFACE ELEVATION 430 ft (±)			
		Stiff, brown and orange-brown, clayey SILT; medium to low plasticity, trace sand- to gravel-size fragments of extremely soft (R0), predominantly decomposed basalt (Possible Landslide Debris)			S-11
45		Very stiff, red, clayey SILT; trace yellowish-white and black, sand-size fragments of predominantly decomposed basalt, relic rock structure (Possible Landslide Debris)	45.0		S-12
50		Very stiff, gray, clayey SILT; medium to high plasticity, some coarse-grained sand- to fine gravel-size fragments of extremely soft (R0), predominantly decomposed basalt (Possible Landslide Debris)	51.0		S-13
55					S-14
60		Soft to medium hard (R2 to R3), gray and reddish-brown BASALT; coarse-grained sand- to gravel-size fragments of slightly weathered rock (Columbia River Basalt)	60.0		S-15
65		Hard, gray BASALT; vesicular, close to moderately close fractures, fresh to slightly weathered (Columbia River Basalt)	64.0		RUN 1
70					RUN 2
75					RUN 3
		(6/15/2012)	76.0		
80					

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)
▲ BLOWS PER FOOT
● MOISTURE CONTENT, %
CORE RECOVERY, %
RQD, %



I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
▌ NX CORE RUN
SLOTTED PVC PIPE
▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY

Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-1 (cont.)

AUG. 2012 | JOB. NO. 5338 | FIG. 1A

121 S.W. Salmon, Suite 900 ▪ Portland, Oregon 97204-2919 ▪ www.msa-ep.com