SEISMIC RESILIENCY ANALYSIS REPORT

B&V PROJECT NO. 406828

PREPARED FOR

City of Salem

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City of Salem

Seismic Resiliency Analysis Report

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List of Abbreviations

2013 ORP	2013 Oregon Resilience Plan
2016 NIST CRPG	2016 Community Resilience Planning Guide for Buildings and Infrastructure Systems by the National Institute of Standards and Technology
AACE	Association for the Advancement of Cost Engineering
ALA	American Lifelines Alliance
ASCE	American Society of Civil Engineers
ASR	Aquifer Storage and Recovery
ADD	Average Daily Demand
CIP	Capital Improvement Plan
CIPP	Cured-in-Place Pipe
City	City of Salem
CMU	Concrete Masonry Unit
COF	Consequence of Failure
CSZ	Cascadia Subduction Zone
DIP	Ductile Iron Pipe
DOGAMI	Oregon Department of Geology and Mineral Industries
GIS	Geographic Information System
HDPE	High Density Polyethylene
LF	Linear Feet
LOF	Likelihood of Failure
LOS	Level of Service
MG	Million-Gallon
OAR	Oregon Administrative Rule
ODOT	Oregon Department of Transportation
OH&P	Overhead and Profit
OHA	Oregon Health Authority
O&M	Operations and Maintenance
OPCC	Opinion of Probable Construction Cost
PGD	Permanent Ground Deformation
PGV	Peak Ground Velocity
PVC	Polyvinyl Chloride
PVCO	Molecularly Oriented Polyvinyl Chloride
SCR	Structural Clay Research
SEFT	SEFT Consulting Group
USGS	U. S. Geological Survey
WTP	Water Treatment Plant

Executive Summary

The Oregon Health Authority (OHA), under Oregon Administrative Rule (OAR) 333-061-0060(5)(a)(J), requires community water systems with greater than 300 connections to develop a seismic resiliency assessment and mitigation plan. The plan needs to be a component of the Water System Master Plan which the City of Salem (City) is concurrently preparing. This Seismic Resiliency Analysis Report (Report) is intended to satisfy this requirement.

OHA recommendations are aimed at mitigating the impacts of a potential occurrence of a Magnitude 9.0 (M9.0) Cascadia Subduction Zone (CSZ) earthquake. Following a CSZ earthquake event, the City's water system could suffer significant damage, which can cause service disruptions, public safety hazards, and impact firefighting capabilities. The primary objectives of this Report are to:

1. Establish level of service (LOS) goals to assist the City in prioritizing restoration of functionality to support the community's most vital social and economic needs;
2. Identify infrastructure (both pipelines and facilities) needed to supply water to critical customers and locations after an earthquake emergency – also called the water system backbone;
3. Assess seismic hazards, such as shaking and ground displacement, liquefaction, and lateral spreading, and their likelihood to impact critical infrastructure;
4. Assess the expected seismic performance of the backbone pipelines and selected facilities; and
5. Identify preliminary recommendations for system improvements that should be implemented to restore water service more rapidly after a major earthquake to meet social and economic needs.

The City established LOS goals which define both customers and water system functions that will need to be operational within the short term (1 to 7 days), intermediate-term (within 4 weeks), and long-term (within months) following a CSZ earthquake. LOS goals are summarized in Table ES-1. The colors and corresponding letters below signify red for minimal, yellow for functional, and green for operational. These are explained in further detail in Table 2-4,

Table ES-1 Level of Service Goals following a CSZ Earthquake for City of Salem

Water Components	% "Operational" Scale/Scenario	Target Time Frame for Recovery							
		Phase 1: Short Term			Phase 2: Intermediate Term			Phase 3: Long Term	
		Days			Weeks			Months	
		0-1	1-3	3-7	1-2	2-4	4-12	3-6	6-12
Source									
Raw or source water and terminal reservoirs	% of winter average day demand (ADD)	R	Y		G				
Raw water conveyance (pump stations and piping to WTP)	% of winter ADD	R	Y		G				
Water production (flow rate)	% of winter ADD	R	Y		G				
Well and/or treatment operations functional (quality)	Minimum water quality objectives met	R	Y		G				
Transmission (including Booster Stations)									
Backbone transmission facilities (pipelines, pump stations, and tanks)	Supporting critical facilities and fire flow	G							
Water for fire suppression at key supply points (to promote redundancy)	% of fire flow x duration	G							
Control Systems & Instrumentation									
SCADA and other control systems (WTP and boosters)	% of components for normal operation	Y	G						
Distribution									
Critical Facilities									
Wholesale Customer - City of Turner	% of winter ADD	Y	G						
Critical City, community, and state facilities identified as having a short-term (no disruption) recovery goal in Table 2-2	% of winter ADD	G							
Critical City, community, county, and state facilities identified as having a short term (1-3 days) recovery goal in Table 2-2	% of winter ADD	Y	G						
Emergency Housing									
Emergency shelters	% of water for drinking & sanitation	Y	G						
Housing/Neighborhoods									
Potable water available at community distribution centers	% of water for drinking & sanitation		Y	G					
Water for fire suppression at fire hydrants	% of hydrants			R	Y	G			
Community Recovery Infrastructure									
All other customers	% of customers		R	Y	G				

The water system backbone was defined to more clearly lay out portions of the water system that are critical to provide short-term functionality and to define potential emergency shelters, community water distribution points, and vulnerable populations. To define the water system backbone, the City first established criticality levels for vertical facilities and distribution and transmissions system pipelines. A water system backbone map is provided in Section 3.0.

Both water system pipeline and vertical facilities were assessed for their vulnerability to earthquake damage, based on the characteristics of the facility or pipeline (such as bracing or joints) and the mapped geohazards from a 2021 Seismic Geohazard Evaluation Report completed by Shannon & Wilson. Table ES-2 summarizes the seismic hazard rankings for critical vertical facilities assessed by Shannon & Wilson.

Table ES-2 Seismic Hazard Rankings for Critical Vertical Facilities

Site ID	Locations	Site Class[1]	Liquefaction Settlement Hazard[2]	Landslide Hazard[2]	Fault Rupture Hazard[2]
1	Salem-Keizer Intertie/Cherry Ave Booster Pump Station	D	M	L	L
2	Grice Hill Reservoir and Repeater Tower	B	L	L	L
3	Hemlock Well[3]	B	L	L	L
4	Mountain View Reservoir and Pump Station	B	L	L	L
5	EOLA 1B Reservoir[3]	B	L	M	L
6	Limelight Pump Station[3]	B	L	L	L
7	Fairmount Reservoir[3]	B	L	L	L
8	Candalaria Reservoir	B	L	L	L
9	South Salem Repeater Tower	B	L	L	L
10	Croisan Lower Pump Station[3]	C/D	M	H	L
11	Edwards S1 Pump Station[4]	D	H	M	L
12	ASR Wells[3]	B	L	L	L
13	Skyline Repeater Tower[3]	B	L	L	L
14	Lone Oak Reservoir	B	L	L	L
15	Creekside Pump Station[3]	B	L	L	L
16	Champion Hill Reservoir	B	M	M	L
17	Boone Road Pump Station[3]	D	L	L	L
18	Deer Park Pump Station	B	L	L	L
19	Mill Creek Reservoir	B	L	L	L

Site ID	Locations	Site Class[1]	Liquefaction Settlement Hazard[2]	Landslide Hazard[2]	Fault Rupture Hazard[2]
20	Turner Control Facility	D	L	L	L
21	Franzen Reservoir and Repeater Tower[4]	B	L	H	M
22	Geren Island WTP	D	L	L	L

[1] Site classified as Site Class A, B, C, D, E or F based on the site soil properties in accordance with Chapter 20 of ASCE 7.
[2] L = Low, M = Moderate, H = High
[3] Sites did not have subsurface exploration data. Nearby well logs could not be found for these sites. Therefore, the risk assessments for these facilities are based on regional seismic hazard mapping only.
[4] Geologic maps may not adequately capture geohazards for locations indicated. Refer to the Shannon and Wilson 2021 Seismic Geohazard Evaluation Report for more discussion on this topic.

Table ES-3 summarizes the pipeline failures by permanent ground deformation (PGD) and peak ground velocity (PGV) .

Table ES-3 Pipeline Failures for PGD, PGV, Total

PGD-Related Failures			PGV-Related Failures			
Breaks	Leaks	Total Failures (Breaks + Leaks)	Breaks	Leaks	Total Failures (Breaks + Leaks)	Total Failures (Breaks + Leaks)
3360	840	4200	11	46	57	4257

Finally, recommended risk mitigation efforts and their associated costs were developed according to the City's LOS goals. In the **short term**, the City should focus on implementing mitigation that will help to preserve water in the system or to convey water to the backbone after an earthquake. As a priority, the City should implement the following strategies:

- Installation of seismic isolation valves installed at all reservoirs (the City already has seismic valves installed on a significant number of them) and seismic upgrades on the "very high" to "moderate" risk reservoirs and their control buildings.
- Seismic upgrades to pump stations which are appurtenant to reservoirs.

The City should also focus on conveyance of treated water to the backbone, by hardening the transmission lines from Geren Island Water Treatment Plant (WTP) to critical reservoirs, including to West Salem. The City should also implement providing alternative water supplies within this phase. Alternative local water supply development (such as drilling of new wells to access groundwater supplies) will provide additional supply reliability in the case of an emergency. The City should also complete studies to understand system vulnerability and risk at vertical facilities not assessed as part of this study, such as Franzen Reservoir. As part of the short-term phase, all "moderate" to "very high" risk facilities should be seismically improved, and all "moderate to high" to "very high" risk pipelines should be hardened.

In the **medium term**, the City should focus on hardening the rest of the backbone system so that the system will remain operational following a major earthquake. "Low to moderate" and "low" risk facilities should be seismically improved, and "moderate" and "low to moderate" risk pipelines (all remaining pipelines within the backbone system) should be hardened.

In the **long term**, the City should focus on hardening the rest of the distribution system to address the LOS goals discussed in Section 2.0. The City aims to serve a minimum of 80% of all customers within 1 to 2 weeks following a M9.0 CSZ earthquake. A limited number of breaks and leaks can be repaired by City crews in the days and weeks following an earthquake. To reduce the number of breaks and leaks down to an amount that can be quickly repaired by the City following an earthquake, and to meet the LOS goals, the City would need to replace most "low" risk pipelines.

A summary of the priorities and total costs for the short, medium, and long term are presented in Table ES-4. These costs were developed to the Class 5 (conceptual) level of accuracy, as defined by the Association for the Advancement of Cost Engineering (AACE), and expected to have an accuracy range from -30% to +50% of actual (2022) costs.

Table ES-4 Seismic Improvements Phasing and Cost Summary

Term	Priority	Risk Level of Facilities To Be Improved	Risk Level of Pipelines To Be Improved
Short (0 – 15 Years)	1. Preserve Water in the System	Very High	Very High
	2. Convey Treated Water	High	High
	3. Implement Alternative Supplies	Moderate to High	Moderate to High
	4. Complete Studies to Refine Understanding of Expected System Performance	Moderate	
Total Cost (Short Term)		**$8.61 – 12M**	**$1.82B**
Medium (10 – 25 Years)	5. Harden the Rest of the Backbone	Low to Moderate	Moderate
		Low	Low to Moderate
Total Cost (Medium Term)		**$0.41 – 0.90M**	**$0.56B**
Long (20 – 50 years)	6. Harden Distribution System to Reduce the Number of Repairs	-	Low
Total Cost (Long Term)		**$0**	**$1.27B**

1.0 Introduction

The Oregon Health Authority (OHA), under Oregon Administrative Rule (OAR) 333-061-0060(5)(a)(J), requires community water systems with greater than 300 connections to develop a seismic resiliency assessment and mitigation plan. The plan needs to be a component of the Water System Master Plan which the City of Salem (City) is concurrently preparing. This Seismic Resiliency Analysis Report (Report) is intended to satisfy this requirement.

1.1 Water System Description

The City's water system currently consists of the City's water transmission pipelines, the Geren Island WTP, water storage reservoirs, pump stations, and distribution system pipelines. The City relies on the North Santiam River to supply water for the City's approximately 200,000 customers. Water from North Santiam River flows to Detroit Lake, which eventually feeds the Geren Island WTP raw water intake, as shown on Figure 1-1.

Figure 1-1 Salem's Water System[1]

Large-diameter transmission pipelines deliver water from the Geren Island WTP to the 92-million-gallon (MG) Franzen Reservoir located in the City of Turner and, subsequently, the City's transmission and distribution system. The City's transmission and distribution system is supported by numerous pump stations and storage reservoirs within and adjacent to the City's service area. The City also operates four aquifer storage and recovery (ASR) wells in Woodmansee Park that supplement the water supply.

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[1] Source: https://online-voice.net/salemgerenisland/, December 2022.

1.2 Project Overview

Following a Cascadia Subduction Zone (CSZ) earthquake, the City’s water system could potentially suffer significant damage, which can cause service disruptions, public safety hazards, and impact firefighting capabilities. This Report serves the following primary objectives:

1. Establish LOS goals to assist the City in prioritizing restoration of functionality to support the community's most vital social and economic needs;
2. Identify infrastructure (both pipelines and facilities) needed to supply water to critical customers and locations after an earthquake emergency – also called the water system backbone;
3. Assess seismic hazards, such as shaking and ground displacement, liquefaction, and lateral spreading, and their likelihood to impact critical infrastructure;
4. Assess the expected seismic performance of the backbone pipelines and selected facilities; and
5. Identify preliminary recommendations for system improvements that should be implemented to restore water service more rapidly after a major earthquake to meet social and economic needs.

This Report analyses a subset of the following assets:

- Storage reservoirs.
- Pump stations.
- ASR wells.
- Pipelines (including pressure relief valves).
- Major control features (Turner Control Facility).

1.3 Study Limitations

The recommendations presented in this Report were developed with the standard of care commonly used for the profession. No other warranties are included, either expressed or implied, as to the professional advice included in this Report. This Report has been prepared for the City, to be used solely in its evaluation of the seismic safety of the water system components referenced. This Report has not been prepared for use by other parties and may not contain sufficient information for purposes of other parties or uses.

1.4 Background Information

The following available information was used as a part of this Report:

- 2021 Seismic Geohazard Evaluation Report completed by Shannon & Wilson.
- Geographic Information System (GIS) data including land use, tax lots, water system, etc., dated September 2020 and May 2021.
- Relevant reports pertaining to the City's water system and emergency management measures, including the 2003 Emergency Operations Plan, 1999 and 2014 Salem Emergency Management Plan, the 2020 American Water Infrastructure Act Risk and Resilience Assessment, 2004 Salem Water System Master Plan, 2019 Water Management and Conservation Plan, 2017 Natural Hazards Mitigation Plan.

- Selected record drawings of critical City water system facilities.
- Available seismic evaluations and seismic studies of the City's water system facilities.
- Maps of the City's reservoirs, pump stations, treatment, distribution systems, and upper and lower transmission maps.
- Field reconnaissance performed in 2021 by Black & Veatch and its subconsultant, SEFT Consulting Group (SEFT), at critical facilities and the Center Street and Marion Street Bridges.
- Meetings and workshops with City staff, conducted in 2021 and 2022, to discuss critical facilities, LOS, the water system backbone, geohazards, system vulnerability, and system mitigation and improvements.
- Meetings with key stakeholders, including Marion County, Polk County, the State of Oregon, and the City Fire Department.
- 2001 American Lifelines Alliance (ALA), Seismic Fragility Formulations for Water Systems (ALA, 2001), which is used widely for pipeline vulnerability assessments.
- Geohazards datasets, including Earthquake Hazard Maps of the Salem East and Salem West Quadrangles, Marion and Polk Counties, Oregon (GMS-105; Wang and Leonard, 1996); the Oregon Geologic Data Compilation Release 5; Statewide Landslide Information Database for Oregon Release 2 (Burns and others, 2011); the bedrock ground motions included in the publication provided to the Oregon Department of Geology and Mineral Industries (DOGAMI) by the U.S. Geological Survey (USGS) based on the USGS Cascadia M 9.0 scenario ShakeMap®; Seismic Hazard Maps based on the Magnitude 9.0 CSZ scenario defined in the Oregon Resilience Plan; and local geological information compiled by Shannon & Wilson.

2.0 Level of Service Goals

LOS goals establish target post-earthquake restoration timeline expectations for buildings, water system components, and customer groups based on supporting the community's social and economic needs after an earthquake. This section presents a definition of LOS goals and highlights special considerations based on City-specific circumstances.

2.1 Purpose

LOS goals, paired with a detailed understanding of the water system backbone, will be used to help identify the gaps between the system's anticipated performance and the City's desired performance during disaster recovery (NIST, 2016). Therefore, in addition to helping to establish a "triage" response to disaster recovery by assigning degrees of urgency to key system components, these LOS goals will also be used to prioritize improvements that address performance deficiencies (defined in the Risk Mitigation Plan in Section 6.0).

LOS goals establish a phased approach to restoring water system operation (in terms of both water quantity and quality) in the days, weeks, and months after a major earthquake and help the City prioritize restoration of functionality. Fifty to 60% of businesses in Oregon are small businesses that can only tolerate 2 to 4 weeks of disruption of essential services.

A system with low resilience requires a longer recovery time, resulting in more interruption in lifeline services, as shown on Figure 2-1. Pre-disaster mitigation; disaster preparedness; and a phased, prioritized approach to recovery can help to shorten recovery time and build resilience for essential services.

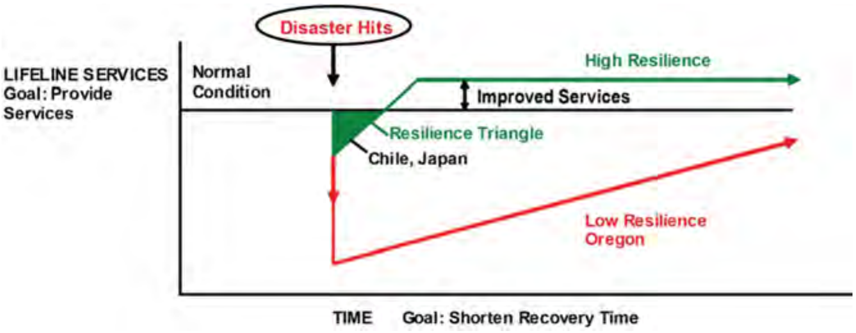

Figure 2-1 Resilience Triangle[2]

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[2] Source: Wang, et al., 2012

2.2 Standards and References

Two key references were considered when developing City-specific LOS goals:

- The 2013 Oregon Resilience Plan (2013 ORP) developed by the Oregon Seismic Safety Policy Advisory Commission, which provides a roadmap for reducing risk and improving recovery after a CSZ earthquake. The 2013 ORP suggests performance goals for the time required to restore water services to affected communities in the aftermath of a CSZ earthquake.
- The 2016 Community Resilience Planning Guide for Buildings and Infrastructure Systems by the National Institute of Standards and Technology (2016 NIST CRPG). This document establishes a resilience planning process which involves determining a community's resilience goals and objectives. It also includes refinement for LOS categories (versus the categories included in the 2013 ORP) to more transparently cluster assets into groups based upon their functions and the degree of urgency for restoring their functions.

A phased approach to disaster recovery (shown in Table 2-1) considers those primary functions that are necessary in three key phases following the disaster: short term (days), intermediate term (weeks), and long term (months). LOS goals are defined both in terms of the estimated time for recovery as well as the target functionality of the system. Different levels of functionality are necessary at different phases of recovery to meet the customers' life-safety needs in the short term, social needs in the intermediate term, and economic recovery needs in the long term.

Table 2-1 Phased Recovery of the Built Environment (2016 NIST CRPG)

Phase	Primary Functions	Associated Infrastructure Clusters
Short Term (Days)	Secure, rescue, stabilize, clear routes	Critical facilities, emergency housing, related infrastructure systems
Intermediate Term (Weeks)	Restore neighborhoods, meet social needs	Housing, medical, main street, schools, churches, related infrastructure systems
Long Term (Months)	Community social and economic recovery	Commercial businesses, industrial businesses, related infrastructure systems

2.3 Level of Service Workshop

The project team conducted a LOS workshop with the City's Public Works staff on October 13, 2020 and continued on October 29, 2020. At this workshop, the team provided an overview of resilience planning, discussed several examples of other resilience plans, and discussed LOS goals. The objective of the workshop was to establish a mutual understanding of seismic resilience and resilience planning for water infrastructure and to set LOS goals for water system components and customer groups based on supporting the community's social and economic needs after an earthquake.

At the workshop, the following topics were discussed and defined to establish LOS goals for each asset category:

1. Categories of critical facilities that need water after an earthquake emergency;
2. Measurement of operational service performance; and
3. Emergency response coordination efforts with state, county, City fire department or other emergency services, and retail water agencies.

2.4 Community Needs Following a Major Earthquake

Table 2-2 provides a breakdown of restoration priorities for City customers that was developed jointly in collaboration with City staff and other state and county stakeholders. Table 2-2 links social/economic needs to service recovery goals.

Table 2-2 Social/Economic Needs of the Community

Recovery Phase	Social/Economic Needs	
Short Term (no disruption)	City/Community Services	• Water for fire suppression at key supply points • Salem Health Hospital • Dialysis centers
	State of Oregon Services	• Anderson Readiness Center • Department of Public Safety Standards and Training Campus • State Data Center • Oregon State Hospital
	Wholesale Customers	• City of Turner
Short Term (1-7 days)	City/Community Services	• City Police Department • Willamette Valley Communications Center • City Fire Stations[1] • City Hall • City Shops Complex • Salem Municipal Airport • City Main Library • Community water distribution points[1] • Emergency shelters[1] o High schools o Middle schools o Colleges • Vulnerable populations[1] o Special needs facilities o Rehabilitation facilities o Senior care facilities • Urgent care centers[1] o Salem Clinic o Salem Health Urgent Care o SwiftCare LLC o Urgent Care Clinic South o Urgent Care Kaiser Permanente North Lancaster o ZOOM+Care
	Marion County Services (exact locations should be coordinated between county and City staff)	• Marion County Sheriff's Office • Marion County Correctional Facility • Marion County Office Building • Marion County Health & Human Services Building

Recovery Phase	Social/Economic Needs	
Short Term (1-7 days)	State of Oregon Services	• Oregon National Guard Army Aviation Support Facility • Oregon Department of Aviation • Oregon State Police/Oregon State Fire Marshall • Oregon Department of Transportation (ODOT) Campus • State Motor Pool • Department of Forestry Campus • Department of Corrections o Mill Creek Correctional Facility o Oregon State Correctional Institute o Oregon State Penitentiary o Santiam Correctional Institute • State Buildings around Capitol Mall o State Capitol o State Library o State Supreme Court Building o Department of Administrative Services o Transportation Building (ODOT) o Department of Energy Building o Public Services Building o Barbara Roberts Human Service Building o Public Utilities Commission Building • State Fair Grounds • Treasury Building • Lottery
Intermediate Term (within 4 weeks)	City/County/State Services	• Remaining City/County/State service facilities • School district facilities
	Wholesale Customers	• Suburban East Salem Water District • Orchard Heights Water Association
	Retail Customers	• Medical office buildings • 90% of businesses, residential customers, fire hydrants
Long Term (months)	Retail Customers	• Remaining 10% of customer connections and fire hydrants

[1] Critical facilities were determined by a desktop assessment performed in collaboration with City staff. Further vetting and assessment of these locations will occur following this report, to finalize the list of critical fire stations, community water distribution points, emergency shelters, vulnerable populations, and urgent care centers.

The recovery phase goals in Table 2-2 have been established based on our current understanding of the community's social and economic needs, without consideration or knowledge of the current expected seismic performance of these existing community facilities. To support community social and economic needs on a similar timeline to that proposed for the water system, many of these community facilities may need to be relocated, seismically retrofitted, or replaced with new facilities that are designed with a higher structural and non-structural performance objective.

2.5 Level of Service Components

This section describes the three components of LOS goals: 1) water system functional categories, 2) target time frames for recovery, and 3) restoration levels.

2.5.1 Functional Categories

The City's water system is grouped into four functional categories, as shown in Table 2-3. The four categories are based upon the 2016 NIST CRPG: Source, Transmission, Control Systems & Instrumentation, and Distribution. Distribution is further broken down into four subcategories: Critical Facilities, Emergency Housing, Housing/Neighborhoods, and Community Recovery Infrastructure. Water system categorization helps to facilitate assigning target time frames for recovery by asset class and function, not by individual asset.

Table 2-3 Water System Functional Categories

Functional Category	System Components	Description
Source	Raw or source water and terminal reservoirs	Water source itself before intake facilities
	Raw water conveyance	Pump stations and piping to WTP
	Water production	Production flow rate
	Well and/or treatment operations	Water quality
Transmission (including Booster Stations)	Backbone transmission facilities	Pipelines, pump stations, and tanks
	Water for fire suppression at key supply points	Reservoirs, hydrants on the backbone, temporary water sources to promote redundancy
Control Systems & Instrumentation	SCADA and other control systems	Server and communication facilities (WTP vs. booster stations)
Distribution	Critical facilities	Wholesale customers, hospitals, emergency operations centers, vulnerable populations
	Emergency housing	Emergency shelters
	Housing/Neighborhoods	Potable water available at community distribution centers; Water for fire suppression at fire hydrants
	Community Recovery Infrastructure	All other customers

2.5.2 Target Time Frames for Recovery

There are three recovery phases that have target time frames for water system recovery:

- Recovery Phase 1 – Short Term (0-7 days)
- Recovery Phase 2 – Intermediate Term (1-12 weeks)
- Recovery Phase 3 – Long Term (3-12 months)

2.5.3 Restoration Levels

Descriptions of suggested LOS restoration levels (adapted from the 2013 ORP) are shown in Table 2-4.

Table 2-4 Level of Service Restoration Levels

Restoration Stage and Description	Operational Level	Symbology
Minimal: A minimum LOS is restored, primarily for use of emergency responders, repair crews, and in support of critical health and human services.	20-30% Operational	Red
Functional: Although service is not yet restored to full pre-event capacity, it is sufficient to get the economy moving again. Limits may be placed on uses that take up a lot of capacity.	50-60% Operational	Yellow
Operational: A full LOS has been restored and is sufficient to allow people to use the system for non-essential activities	80-90% Operational	Green

2.6 Level of Service Goals

The LOS categories and their respective target time frames for recovery agreed upon in the LOS workshop are presented in Table 2-5. The City also determined the units by which the percentage (%) of operational level could be measured. These units vary by asset, or group of assets, and are summarized in Table 2-5.

Table 2-5 Level of Service Goals

		Target Time Frame for Recovery							
		Phase 1: Short Term			Phase 2: Intermediate Term			Phase 3: Long Term	
		Days			Weeks			Months	
Water Components	**% "Operational" Scale/Scenario**	**0-1**	**1-3**	**3-7**	**1-2**	**2-4**	**4-12**	**3-6**	**6-12**
Source									
Raw or source water and terminal reservoirs	% of winter average day demand (ADD)	R	Y		G				
Raw water conveyance (pump stations and piping to WTP)	% of winter ADD	R	Y		G				
Water production (flow rate)	% of winter ADD	R	Y		G				
Well and/or treatment operations functional (quality)	Minimum water quality objectives met	R	Y		G				
Transmission (including Booster Stations)									
Backbone transmission facilities (pipelines, pump stations, and tanks)	Supporting critical facilities and fire flow	G							
Water for fire suppression at key supply points (to promote redundancy)	% of fire flow x duration	G							
Control Systems & Instrumentation									
SCADA and other control systems (WTP and boosters)	% of components for normal operation	Y	G						
Distribution									
Critical Facilities									
Wholesale customer – City of Turner	% of winter ADD	Y	G						
Critical City, community, and state facilities identified as having a short-term (no disruption) recovery goal in Table 2-2	% of winter ADD	G							
Critical City, community, county, and state facilities identified as having a short-term (1-3 days) recovery goal in Table 2-2	% of winter ADD	Y	G						
Emergency Housing									
Emergency shelters	% of water for drinking & sanitation	Y	G						
Housing/Neighborhoods									
Potable water available at community distribution centers	% of water for drinking & sanitation		Y	G					
Water for fire suppression at fire hydrants	% of hydrants			R	Y	G			
Community Recovery Infrastructure									
All other customers	% of customers		R	Y	G				

3.0 Water System Backbone Definition

This section describes the water system backbone consisting of transmission pipelines, pump stations, and storage and treatment facilities, which are needed to support fire flow and the critical social/economic needs of the community. The backbone system will support the Short-Term Recovery Phase outlined in Table 2-2 in the initial days following a CSZ earthquake.

The long-term goal for the water system backbone is that it remains operational or experiences only minor damage after a major earthquake. Because it will be challenging to implement any significant repairs to the water system backbone in the initial days after an earthquake, backbone components should be capable of remaining operational without sustaining significant damage during a CSZ earthquake event.

3.1 Water System Backbone Workshops

The project team conducted a workshop with City staff on November 5, 2020, to establish the needs for the backbone system. At this workshop, the project team provided an overview for identification and prioritization of a water system backbone. This backbone was developed following this workshop with the City through a collaborative and iterative process. The City engaged the fire department, Marion and Polk Counties, and the state in conversations about their critical facilities that need to remain operational and be staffed following a CSZ earthquake event.

3.2 Water Supply Points for Fire Suppression

A key long-term goal for the water system backbone is that it provides a reliable source for tanker trucks to obtain water for fire suppression following an earthquake. To enable this goal, the backbone must consist of a seismically-hardened system of pipelines with hydrants and key reservoir sites distributed throughout the City. The majority of the City's reservoirs have seismic shutoff valves to preserve water storage. Additionally, City fire trucks are able to draft water directly from the Willamette River.

3.3 Critical Social/Economic Needs

The process of identifying the water system backbone begins by locating critical water system customers in the Short Term Recovery Phase, which include the following:

- Hospitals
- Urgent Care Centers
- Dialysis Centers
- City of Salem Critical Services
- State of Oregon Critical Services
- Marion County Critical Services
- Correctional Facilities
- Emergency Shelters
- Community Water Distribution Points
- Vulnerable Populations

Appendix A includes a detailed list of these facilities within the City limits that are outlined in Table 2-2.

A special consideration in the above list are dialysis facilities. There are several dialysis facilities in Salem which provide specialty care. Approximately 100 gallons of water is required every 3 days per dialysis patient. The City is working with the dialysis centers in the region to identify more permanent facility locations that may potentially be connected to the backbone, along with the hospital and urgent care facilities. These facilities were not identified at the conclusion of this Report.

3.3.1 Emergency Shelters

Emergency shelters are typically located in existing dormitories or large, open buildings where temporary shelters can quickly be established. In addition to university dormitories, convention/exposition centers and school gymnasiums have the potential to serve as emergency shelters following an earthquake, provided that these buildings are constructed or retrofitted to perform well during a CSZ earthquake event.

Table 3-1 lists 23 potential emergency shelter locations identified by City staff, though there is no agreement currently in place between the City and Salem Keizer School District to operate any schools as an emergency shelter. To date, there have been concerns about the seismic performance of some of these buildings. Since resilience planning considers implementing improvements over a long time frame (approximately 50 years), it may be reasonable to assume that the seismic performance of these facilities may be improved, making them viable shelter locations in the future.

Table 3-1 Potential Emergency Shelter Locations

Location	Address	Building Type
Auburn Elementary School	4612 Auburn Rd NE	Public Elementary School
Battle Creek Elementary School	1640 Waln Dr SE	Public Elementary School
Brush College Elementary School	2623 Doaks Ferry Rd NW	Public Elementary School
Chemeketa Community College	4000 Lancaster Dr NE	Community College
Corban University	5000 Deer Park Dr SE	College / University Building
Crossler Middle School	1155 Davis Rd S	Public Middle School
Houck Middle School	1155 Connecticut St SE	Public Middle School
Judson Middle School	4512 Jones Rd SE	Public Middle School
Leslie Middle School	3850 Pringle Rd SE	Public Middle School
McKay High School	2440 Lancaster Dr NE	Public High School
North Salem High School	765 14th St NE	Public High School
Parrish Middle School	802 Capitol St NE	Public Middle School
Putnam University Center	935 Mill St SE	College / University Building
Robert W Straub Middle School	1920 Wilmington Av NW	Public Middle School
Roberts High School-State Street Campus	3620 State St	Public Alternative High School
Salem Convention Center	200 Commercial St SE	Assembly / Exhibition Hall
South Salem High School	1910 Church St SE	Public High School
Sprague High School	2373 Kuebler Rd S	Public High School
Stephens Middle School	4962 Hayesville Dr NE	Public Middle School
Tokyo International University of America	1300 Mill St SE	College / University Building
Waldo Middle School	2805 Lansing Av NE	Public Middle School
Walker Middle School	1075 8th St NW	Public Middle School
West Salem High School	1655 Doaks Ferry Rd NW	Public High School

3.3.2 Community Water Distribution Points

Community water distribution points are locations throughout the service area where customers can fill their own containers during a water outage. Distribution points can be permanent locations along the water system backbone, or they can be temporary sites, as shown on Figure 3-1, where portable systems are deployed (e.g., water trucks, portable tanks, blivets, etc.).

In May 2018, the following seven temporary community water distribution points were established in the Salem water service area during a cyanotoxin water advisory:

1. Chemeketa Community College, 4000 Lancaster Drive NE, Salem
2. Oregon State Fair Grounds, 2330 17th Street NE, Salem (refer to Figure 3-1)
3. Wallace Marine Park Softball Complex, 200 Glen Creek Road NW, Salem
4. AMF Firebird Lanes, 4303 Center Street NE, Salem
5. Bush's Pasture Park, 600 Mission Street SE, Salem
6. Woodmansee Park, 4629 Sunnyside Road SE, Salem
7. Former Chevrolet Dealership, 5325 Denver Street, Turner

Photo credit: Kelly Jordan, Statesman Journal

Figure 3-1 Temporary Water Distribution Point at the Oregon State Fairgrounds, June 1, 2018

After a large regional earthquake, it will be difficult to deploy and staff temporary distribution points on a large scale, due to increased demands on City staff. Therefore, the City is planning to establish permanent community water distribution points along the water system backbone at the key sites listed in Table 3-2, including the following:

- All the emergency shelters listed in Table 3-1, which are expected to be operated by the Red Cross or other emergency relief organizations;
- All 11 of the City's fire stations, which are expected to be operated by the Salem Fire Department; and
- Eight other City water facilities and parks, which are expected to be operated by the City Public Works Department.

Table 3-2 Community Water Distribution Points

Location	Address	Building Type
Emergency Shelter Locations – refer to Table 3-1		
Salem Fire Station 1	370 Trade St SE	Salem City Fire Station
Salem Fire Station 2	875 Madison St NE	Salem City Fire Station
Salem Fire Station 3	1884 Lansing Av NE	Salem City Fire Station
Salem Fire Station 4	200 Alice Av S	Salem City Fire Station
Salem Fire Station 5	1520 Glen Creek Rd NW	Salem City Fire Station
Salem Fire Station 6	2740 25th St SE	Salem City Fire Station
Salem Fire Station 7	1970 Orchard Heights Rd NW	Salem City Fire Station
Salem Fire Station 8	4000 Lancaster Dr NE	Salem City Fire Station
Salem Fire Station 9	5080 Battle Creek Rd SE	Salem City Fire Station
Salem Fire Station 10	3611 State St	Salem City Fire Station
Salem Fire Station 11	5021 Liberty Rd S	Salem City Fire Station
Cascades Gateway Park	2100 Turner Rd SE	Developed City Park
Limelight Water Pump Station	880 Van Buren Dr NW	Public Water Pump Station
River Road Park	3045 River Rd N	Developed City Park
Salem City Shops Building 16 Water	1440 20th St SE	Salem City Facility
Salem/Keiser Intertie #1 Pump Station	4000 Block Cherry Ave NE	Public Water Pump Station
South River Road Water Pump Station	3285 River Rd S	Public Water Pump Station
Turner Control Water Facility	7100 3rd St SE	Public Water Facility
Weathers Street Park	4188 Weathers St NE	Developed City Park

The community water distribution points listed in the table above are also included in Appendix A.

3.3.3 Vulnerable Populations

The City's emergency planning efforts have also taken into consideration ways to serve vulnerable customers. Appendix A includes a detailed list of sizable care facilities; retirement centers where seniors receive assisted living, memory, or nursing care; and the Oregon School for the Deaf. All these facilities serve vulnerable populations that need to be supported by the water system backbone.

3.4 Water Facility Criticality Levels

In the development of the water system backbone, City staff prioritized the pumping, storage, piping, and valve facilities within the water transmission system by how important the facility is to the overall operation of the water system (and, therefore, how high the consequence of its failure is) using the priority system described in Table 3-3. Table 3-4 lists the City's water facilities in order of criticality, with the facilities listed alphabetically within each level.

Table 3-3 Water Facility Criticality/Consequence of Failure Level Definitions

Criticality Level	Definition
5 – Highly Critical	Paramount to the operation of the system.
4 – Critical	Necessary to supply water to a significant area.
3 – Semi Critical	The system could operate at reduced capacity without these facilities.
2 – Local Critical	Necessary to supply water to an isolated local area.
1 – Not Critical/Redundant	The system can operate without these facilities. These facilities are not considered part of the system backbone.

Table 3-4 Storage and Pumping Facility Criticality Levels

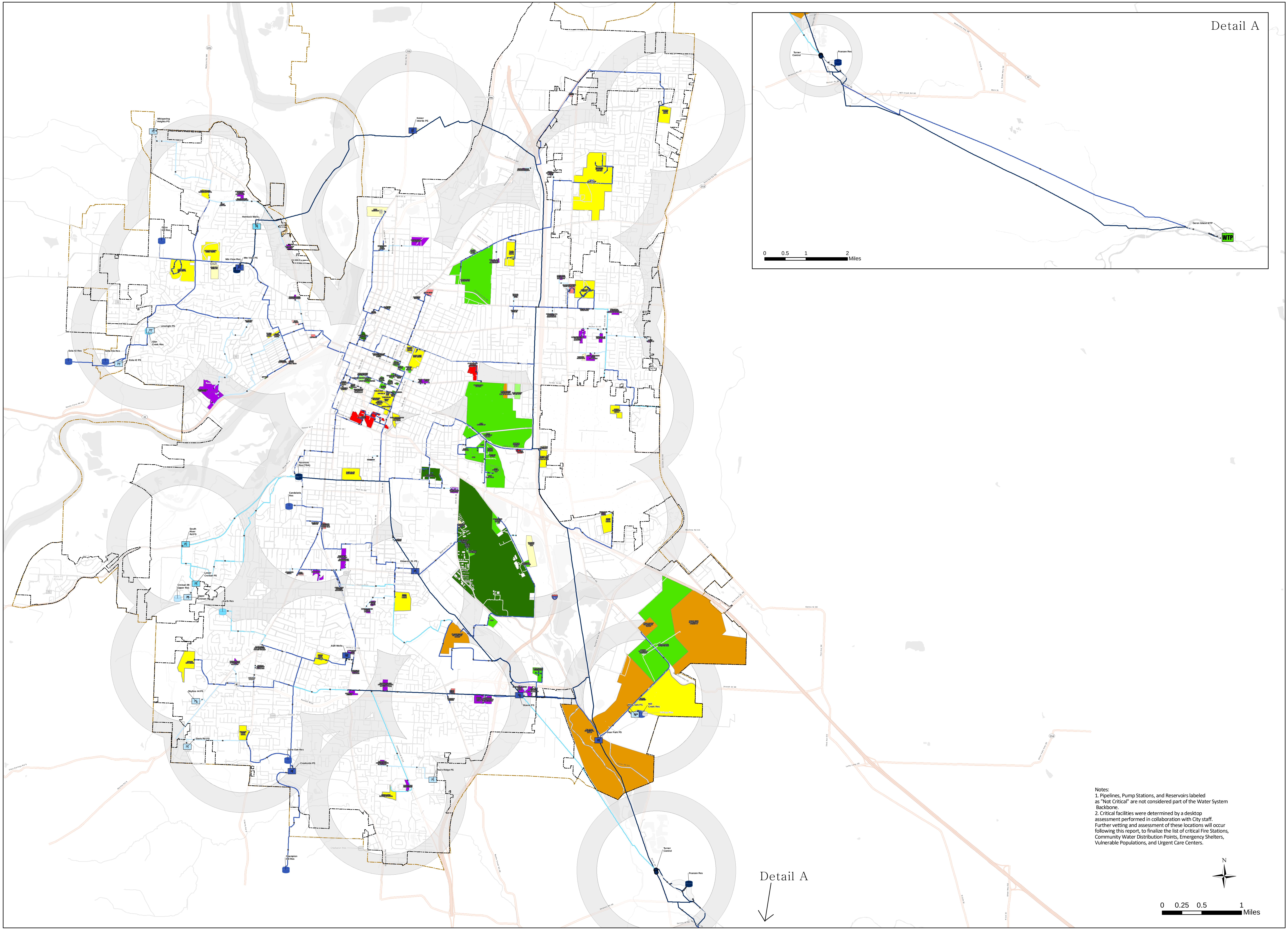
Criticality Level	Name	Service Level	Elevation (ft)
Supply/Valves			
5 – Highly Critical	Geren Island WTP	G-0	470
5 – Highly Critical	Turner Control Facility	G-0	266
4 – Critical	ASR Wells	S-2	~382
2 – Local Critical	Hemlock Well	G-0	188
Reservoirs			
5 – Highly Critical	Fairmont Reservoir	G-0	314
	Franzen Reservoir	Franzen	386
	Mountain View Reservoir	G-0	313
4 – Critical	Candalaria Reservoir	S-1	429
	Champion Hill Reservoir	S-3	709
	Eola #1b Reservoir	W-2	636
	Eola #2 Reservoir	W-3	763
	Grice Hill Reservoir	W-1	483
	Lone Oak Reservoir	S-2	574
	Mill Creek Reservoir	MCCC S-1	424
3 – Semi Critical	Glen Creek Reservoir	W-1	483
	Kurth Reservoir	S-2	553
2 – Local Critical	Croisan Mt Upper Reservoir	S-2	579
1 – Not Critical/ Redundant	Chakarun Reservoir	S-2	580
	College Reservoir	T	438
	Mader Reservoir	S-1	385
	Seeger Reservoir	S-2	553
	Skyline Reservoir	S-3	708

Criticality Level	Name	Service Level	Elevation (ft)
Pump Stations			
4 – Critical	Boone Pump Station	S-2	235
	Creekside Pump Station	S-3	491
	Deer Park Pump Station	S-1	306
	Edwards S1 Pump Station	S-1	206
	Keizer Intertie Pump Station	G-0	130
	Mountain View Pump Station	W-1	308
3 – Semi Critical	Lower Croisan Pump Station	S-2	418
	South River Rd Pump Station	S-1	153
2 – Local Critical	Davis Road Pump Station	S-4	697
	Eola #2 Pump Station	W-3	530
	Limelight Pump Station	W-2	477
	Mill Creek Pump Station	T	349
	Rock Ridge Pump Station	S-3	464
	Skyline #4 Pump Station	S-4	620
	Upper Croisan Pump Station	S-3	510
	Whispering Heights Pump Station	W-2	426
1 – Not Critical/ Redundant	Chatnicka Pump Station	W-3	546
	Edwards S2 Pump Station	S-2	206
	Fairmont Pump Station	S-2	312
	Illahe Pump Station (Private)	S-1	240
	Jefferson Pump Station	W-1	240
	Skyline Pump Station	S-3	502

3.5 Water System Backbone

The resulting water system backbone that is needed to connect to each of the critical water system components (supply, reservoirs, and pump stations) and the social/economic needs of the community (such as critical public agency buildings, emergency shelters, community water distribution points, and vulnerable populations) is shown on Figure 3-2. The water system backbone piping shown on Figure 3-2 connects the tax lots where critical facilities are located. Critical facilities include medical facilities (hospitals and urgent care centers), government facilities, correctional facilities, emergency shelters, community water distribution points, and vulnerable populations. Also shown on the figure are grey 0.75- and 1.0-mile radii around each community water distribution point. These radii represent reasonable walking distances, in case transportation becomes limited after a CSZ earthquake. As shown on Figure 3-2, a significant amount of the City is within 0.75 mile of a community water distribution point, and nearly all of the City is within 1 mile of a community water distribution point.

The water system backbone serves as the foundation for prioritizing seismic upgrades recommended in further sections of this Report.



Notes:
1. Pipelines, Pump Stations, and Reservoirs labeled as "Not Critical" are not considered part of the Water System Backbone.
2. Critical facilities were determined by a desktop assessment performed in collaboration with City staff. Further vetting and assessment of these locations will occur following this report, to finalize the list of critical Fire Stations, Community Water Distribution Points, Emergency Shelters, Vulnerable Populations, and Urgent Care Centers.

Legend

Pipeline Consequence of Failure
- Highly Critical
- Critical
- Semi Critical
- Local Critical
- Labeled w/ Diameter

Pump Station (PS) Consequence of Failure
- Highly Critical
- Critical
- Semi Critical
- Local Critical

Reservoir (Res) Consequence of Failure
- Highly Critical
- Critical
- Semi Critical
- Local Critical

Valve Risk Level
- ● Highly Critical

Social/ Economic Needs
- Urgent Care Center
- Dialysis Center
- Hospital
- City of Salem
- State of Oregon
- Marion County
- Correctional Facility
- Emergency Shelters/ Community Water Distribution Points
- Community Water Distribution Points
- Vulnerable Population
- 0.75-mile Radius from Community Water Distribution Points
- 1.0-mile Radius from Community Water Distribution Points
- Pipeline
- Pump Station
- Reservoir
- City Limits
- Urban Growth Boundary

Figure 3-2:
Water System Backbone

CITY OF Salem

Seismic Resiliency Analysis

BLACK & VEATCH

April 2023

3.6 Considerations and Future Coordination Efforts

Sites identified as community water distribution points require further coordination between the City's Water & Utilities and Public Works Departments and emergency services of the City and Marion and Polk Counties. All fire stations are currently designated as community water distribution points, which means that following a CSZ earthquake, the fire stations will have increased public traffic. This has a potential to interfere with fire apparatus responding to an emergency, depending upon how the public will access the fire station and water. The City Fire Department will need to consider any potential impacts to both staffing and traffic. Similarly, City staff will need to consider the implications of increased traffic and staffing at the City Shops Complex, which is also designated as a community water distribution point. Proactive coordination between the City's Water & Utilities and Public Works Departments and emergency services of the City and Marion and Polk Counties ahead of an emergency can help to effectively support the community following a crisis.

The City is also working with the dialysis centers in the region to identify more permanent facility locations that may potentially be connected to the backbone, along with the hospital and urgent care facilities. These facilities were not identified at the conclusion of this Report.

The City should also coordinate with the Salem Keizer School District with regards to operation of schools as an emergency shelter.

4.0 Water System Seismic Vulnerability Assessment

Both the pipeline system and priority facilities were evaluated to ascertain the likelihood and potential extent of damage to structures and other system infrastructure during an earthquake. Hazards associated with seismic activity that have the potential to adversely affect pipelines or water system facilities include ground rupture, liquefaction, lateral spreading, strong ground shaking, and earthquake-induced landslides. The degree to which these hazards could impact the water system is dependent upon the earthquake magnitude and distance from each pipeline or facility, the proximity to faults, the amount and type of soil displacement, and the joint systems and construction characteristics of the pipeline or facility.

4.1 Geohazards

A Seismic Geohazard Evaluation Report was completed by Shannon & Wilson in May 2021 (refer to Appendix B) to assess the potential for earthquake-induced geologic hazards and formed the basis for developing the seismic vulnerability assessment. The Seismic Geohazard Evaluation Report maps various seismic parameters within the study area based on geological information for the general area. The study area encompassed the City's major water transmission mains and facilities. Seismic hazard maps include peak ground velocity, peak ground acceleration, 0.3-and 1.0-second spectral accelerations, probability of liquefaction, liquefaction induced settlement, and landslide induced permanent ground deformation (PGD) based on the methodology developed by HAZUS. The DOGAMI publishes detailed maps showing bedrock, surficial, or engineering geology for specific regions. GMS-105, one of the DOGAMI maps which focused on the relative earthquake hazard of Marion and Polk Counties, was used as the primary source of liquefaction susceptibility within the Salem area. Permanent ground deformations from liquefaction-induced lateral spreading were calculated.

4.1.1 Pipeline Geohazards

Table 4-1 summarizes the portions of the City's water transmission and distribution system subject to liquefaction-induced lateral spreading, liquefaction-induced settlement, and peak ground velocity (PGV) based on the hazard mapping provided by Shannon & Wilson in Appendix B.

Table 4-1 Potable Water Pipelines Subject to Seismically-Induced Ground Movement

Severity Level	Liquefaction-Induced Lateral Spreading (in.)	% Water System	Settlement (in.)	% Water System	PGV (in./s)	% Water System
Low to High	0 – 0.1	68.6%	0	43.0%	0.00 – 2.90	0.0%
	0.11 – 2	1.3%	1	27.0%	2.91 – 5.90	36.1%
	2.1 – 6	4.2%	2	18.4%	5.91 – 11.90	61.2%
	6.1 – 12	20.7%	6	10.3%	11.91 – 23.90	2.7%
	12.1 – 16	5.1%	Other	1.4%	> 23.91	0.0%

A small portion of the City's northernmost distribution system was outside of the limits of the area assessed for earthquake-induced geologic hazards. The data set for lateral spreading was larger than the data set for settlement, and it was observed that in the northernmost portion of the City, anticipated lateral spreading was directly proportional to anticipated settlement. Therefore, when lateral spreading was known, but the settlement was unknown, settlement was assumed to follow a similar distribution as lateral spreading. In the few areas where lateral spreading and settlement were not known, it was deemed appropriate to assign a value of 6 inches for lateral spread and 2 inches for settlement, based on settlement levels in the adjacent area.

For the pipelines, the main hazards were determined to be localized liquefaction and lateral spreading at the Sunset Park Willamette River crossing and fault rupture where the pipelines cross the Turner and Mill Creek Faults and Waldo Hills Fault. The potential for localized liquefaction is highest at the Willamette River crossings, near the City of Turner and the Geren Island WTP. Note that recent site-specific geotechnical engineering reports for Geren Island WTP indicate that the map-based liquefaction hazard shown in Appendix B may be somewhat overestimated due to the relatively high percentage of gravels underlying that site.

4.1.2 Priority Vertical Facility Geohazards

Facility geohazards were assessed using seismic hazard parameters mapped by Shannon & Wilson in the Seismic Geohazard Evaluation Report in Appendix B. These parameters included ground shaking, liquefaction-induced settlement, liquefaction-induced lateral spreading, and earthquake-induced landslide PGD.

The facilities were selected in close consultation with the City's Engineering Division based on whether the facilities are key to maintaining the integrity of the water system backbone. A total of 24 priority facilities were evaluated as part of this study, as listed in Table 4-2. A detailed structural condition assessment of the structures was not included in the scope of this project.

Table 4-2 Facilities Assessed as Part of this Study

Facilities Assessed

- ASR #1 and #2 Pump Station
- ASR #4 Pump Station
- ASR #5 Pump Station
- Boone Road Pump Station
- Candalaria Reservoir
- Champion Hill Reservoir
- Champion Hill Reservoir Control Building
- Creekside Pump Station
- Deer Park Pump Station
- Edwards S1 Pump Station
- Eola #1B Reservoir
- Fairmount Reservoir
- Fairmount Reservoir Control Building
- Grice Hill Reservoir Control Building
- Limelight Pump Station
- Lone Oak Reservoir
- Lone Oak Reservoir Control Building
- Mill Creek #1 Reservoir
- Mill Creek #1 Reservoir Control Building
- Mountain View Pump Station
- Mountain View Reservoir
- Salem/Keizer Intertie #1 Pump Station
- Turner Control Facility

It is recommended that the City conduct seismic evaluations of the remaining inventory of water system structures (pump stations, reservoirs, communications towers, etc.) as part of a future project. Several facilities were considered critical facilities but were excluded from this evaluation for the following reasons:

- The Geren Island WTP is a key part of the backbone, but the City requested that this facility not be included in the assessment because seismic resiliency upgrades to this facility were being implemented at the time of this study.

- Franzen Reservoir was excluded from assessment because it is an earthen facility which requires a specialized evaluation to review seismic deficiencies and potential improvements. A seismic evaluation of the Franzen Reservior is underway under a separate scope.
- Lower Croisan Pump Station was excluded from further structural and nonstructural assessment because the Shannon & Wilson Geohazard Study recommended a full replacement of this facility.
- The Upper Transmission System (Lines 1 and 2) was evaluated by Carollo Engineers in 2016 under a separate scope. The findings indicated peak ground velocity between Geren Island and Turner is consistent and landslides and liquefaction is unlikely. The majority of anticipated damage to the Upper Transmission System will be near the Turner Control Facility.

4.1.3 Vertical Facility Hazard Rankings

The geotechnical evaluation resulted in a set of hazard rankings being assigned to the critical vertical facilities based on regional seismic mapping and review of existing information on the facilities, as shown in Table 4-3. The geotechnical evaluation found that numerous facilities were in areas where rock is mapped as the geological surface unit. The risk of PGDs at these sites were considered low.

Table 4-3 Seismic Hazard Rankings for Critical Vertical Facilities

Site ID	Locations	Site Class[1]	Liquefaction Settlement Hazard[2]	Landslide Hazard[2]	Fault Rupture Hazard[2]
1	Salem-Keizer Intertie/Cherry Ave Booster Pump Station	D	M	L	L
2	Grice Hill Reservoir and Repeater Tower	B	L	L	L
3	Hemlock Well[3]	B	L	L	L
4	Mountain View Reservoir and Pump Station	B	L	L	L
5	EOLA 1B Reservoir[3]	B	L	M	L
6	Limelight Pump Station[3]	B	L	L	L
7	Fairmount Reservoir[3]	B	L	L	L
8	Candalaria Reservoir	B	L	L	L
9	South Salem Repeater Tower	B	L	L	L
10	Croisan Lower Pump Station[3]	C/D	M	H	L
11	Edwards S1 Pump Station[4]	D	H	M	L
12	ASR Wells[3]	B	L	L	L
13	Skyline Repeater Tower[3]	B	L	L	L
14	Lone Oak Reservoir	B	L	L	L
15	Creekside Pump Station[3]	B	L	L	L
16	Champion Hill Reservoir	B	M	M	L
17	Boone Road Pump Station[3]	D	L	L	L
18	Deer Park Pump Station	B	L	L	L

Site ID	Locations	Site Class[1]	Liquefaction Settlement Hazard[2]	Landslide Hazard[2]	Fault Rupture Hazard[2]
19	Mill Creek Reservoir	B	L	L	L
20	Turner Control Facility	D	L	L	L
21	Franzen Reservoir and Repeater Tower[4]	B	L	H	M
22	Geren Island WTP	D	L	L	L

[1] Site classified as Site Class A, B, C, D, E, or F based on the site soil properties in accordance with Chapter 20 of ASCE 7.
[2] L = Low, M = Moderate, H = High
[3] Sites did not have subsurface exploration data. Nearby well logs could not be found for these sites. Therefore, the risk assessments for these facilities are based on regional seismic hazard mapping only.
[4] Geologic maps may not adequately capture geohazards for locations indicated. Refer to the Shannon and Wilson 2021 Seismic Geohazard Evaluation Report for more discussion on this topic.

The following facilities were rated as having a moderate geologic seismic hazard:

- **EOLA 1B Reservoir:** There is a moderate landslide hazard, as the reservoir is near an existing landslide and there was lack of available site-specific subsurface information. If additional subsurface information is obtained in the future, the hazard potential for this site may be reassessed for landslide hazard.
- **Champion Hill Reservoir:** This facility was assigned a moderate to high hazard for potential liquefaction and landslides. Nearby well logs indicate that the soil is mantled by fine grained flood deposits which are more likely to experience PGD during a seismic event. The geohazard rankings may be reassessed if additional subsurface data is available in the future.

The following facilities were assigned a moderate to high geologic seismic hazard:

- **Croisan Lower Pump Station:** This facility was assigned a moderate hazard for potential liquefaction and high hazard ranking for landslides. The site is near the contact between a large existing landslide and volcanic rock, and there is a lack of available site-specific information. The geohazard rankings may be reassessed if additional subsurface data is available in the future.
- **Edwards S1 Pump Station:** Flood maps and well logs indicate presence of poor soils at the site, and the pump station may be underlain by these soils. Uncontrolled releases of water have resulted in surface settlement around the building foundations. Due to uncertainties associated with liquefaction potential and subgrade, the potential for PGD was considered moderate to high during a seismic event.
- **Franzen Reservoir and Repeater Tower:** These facilities received a moderate hazard ranking for landslide risk and high hazard for fault rupture. These ratings were based on information gathered from existing basis of design reports and understanding of past instability along the earthen embankments.

4.2 Pipeline Vulnerability Assessment

The City's water system was evaluated using the ALA (American Lifelines Analysis) Seismic Fragility Formulations for Water Systems (ALA, 2001), which is used widely for pipeline loss assessments. This method uses fragility curves that can be applied to water system components to evaluate the probability of damage from earthquake hazards. Damage estimates are expressed as pipeline repair rates for breaks and leaks. The general approach is to quantify earthquake shaking (wave propagation) intensity using PGV, quantify the amount of ground movement using PGD, and to use both PGV and PGD to estimate the damage of the system pipelines. The ALA methodology includes pipeline vulnerability functions for both PGV and PGD inputs, which vary based on pipe material.

4.2.1 Pipeline Joint Assumptions

The system includes 934.5 miles of pipe with diameters ranging from 0.75 inch to 69 inches. The pipe material, length, assumed joint type, and assumed K1 and K2 values for each are shown in Table 4-4. K1 and K2 values are constants used in the equation to represent the expected performance of the various pipe materials. K1 and K2 can have a maximum value of 1.0 each, representing the highest degree of vulnerability, which is the value used for cast iron pipe.

Table 4-4 Pipe Material, Length, Joint Type, and K1 and K2 Values

Material	Acronym in City's Database	Length (miles)	Percent of System	Assumed Joint Type	K1	K2
Ductile Iron	DI	453.6	48.5%	Rubber Gasket	0.5	0.5
Cast Iron (pre-1950)	CI	110.9	11.9%	Cement	1.0	1.0
Cast Iron (post-1950)	CI	170.0	18.2%	Rubber Gasket	0.8	0.8
Steel	STEEL	83.8	9.0%	Rubber Gasket	0.7	0.7
Asbestos Cement	AC	36.3	3.9%	Rubber Gasket	0.5	0.8
Concrete Cylinder Pipe	CCP	35.5	3.8%	Rubber Gasket	0.8	0.7
Unknown	UNK	14.1	1.5%	Assume to Be Cast Iron Pipe	1.0	1.0
Polyvinyl Chloride	PVC	10.0	1.1%	Rubber Gasket	0.5	0.8
Blank	Blank	8.4	0.9%	Assume to Be Cast Iron Pipe, pre-1950	1.0	1.0
Iron Pipe	IP	5.5	0.6%	Threaded, no gasket	0.5	0.5
High Density Polyethylene	HDPE	3.9	0.4%	Fused	0.3	0.3
Needs to Be Fixed	FIX	1.5	0.2%	Assume to Be Cast Iron Pipe, pre-1950	1.0	1.0
Concrete	C	0.4	0.04%	Rubber Gasket	0.5	0.8
Unknown	OD	0.2	0.02%	Assume to Be Cast Iron Pipe, pre-1950	1.0	1.0
Blue Brut Polyvinyl Chloride	BB	0.1	0.01%	Rubber Gasket	0.5	0.8
Plastic	P	0.1	0.01%	Assume to Be PVC Pipe	0.5	0.8
Cross-Linked Polyethylene (Pex Pipe)	PEX	0.0	<0.01%	Fused	0.3	0.3
Steel	S	0.0	<0.01%	Rubber Gasket	0.7	0.7
Totals		**934.5**	**100.0%**			

Black & Veatch met with City staff to discuss the assumed joint type for each pipe material based on what was typically installed in the City. In the case of cast iron pipe, the joint type was dependent upon the age of the pipe; prior to 1950, cast iron had cemented joints rather than rubber gasketed joints. If the incorrect joint types are assumed, it could result in different K1 and K2 values, increasing or decreasing the estimated number of failures. The K1 and K2 values for specific pipe materials are taken directly from the ALA document. When there are no values for some types of pipes represented in the City in the ALA document, K1 and K2 values are estimated based on similar types of pipe and pipe joints. The ALA fragility relationships assign variables to each pipe material depending on its relative performance.

There is not enough evidence to prove a diameter effect exists for all pipe materials in any given water system. However, the empirical evidence strongly indicates that some relationship does exist and that the largest pipes, those over 12 inches in diameter, have lower damage rates than common diameter distribution pipes of 4 inches to 12 inches in diameter. Therefore, it is more conservative to assume that pipe diameters are small when assigning K1 and K2 values.

In Table 4-4, the Assumed Joint Type column shows the basis for assuming the K1 and K2 values. Unknown pipe materials (FIX, OD, UNK, and materials left blank) are assumed to have the same performance attributes as cast iron pipe. Plastic pipe is assumed to have the same performance as polyvinyl chloride (PVC) pipe. These pipe materials (FIX, OD, UNK, materials left blank, and P) make up a small percentage of the system (totaling less than 2.6%) and will have a small influence on overall system performance.

4.2.2 Pipeline Failure Assessment

The number of pipe failures is calculated by multiplying the pipe repair rate (RR, repairs/1,000 feet of pipe) times the pipe length (in 1,000s of feet). The ALA fragility relationships used to calculate the RR are as follows:

- **RR = K1 x 0.00187 x PGV**, where PGV = Peak ground velocity in inches/sec
- **RR = K2 x 1.06 x PGD$^{0.319}$**, where PGD = Peak ground displacement in inches

RRs are calculated separately for PGD and PGV and are much lower for PGV than PGD. It was conservatively assumed that the PGD for the purposes of the ALA fragility relationships was the sum of the PGD from both liquefaction and earthquake-induced landslides.

In accordance with the 2001 ALA Guidelines, the vector sum of the liquefaction-induced lateral spreading PGD (horizontal deformation) and liquefaction-induced settlement PGD (vertical deformation) was used to calculate the total PGD associated with liquefaction, which is the distance a block of soil is expected to move during an earthquake (typically downhill or towards a free face) before remaining in that position within a few minutes after the earthquake shaking has stopped.

The breakdown of the number of leaks and breaks is dependent on the hazard environment where the pipe is located. Repairs include both leaks and breaks. The following methodology was used to segregate pipe failures:

- PGD-related failures – 80% breaks and 20% leaks
- PGV-related failures – 20% breaks and 80% leaks

Breaks are described as loss of hydraulic continuity, e.g., the loss of the ability to transmit water from Point A to Point B. "Breaks" include separation of a pipe joint by more than approximately 1 inch, or the blowout of the pipe wall. A break results in significant loss of water; a pipe break results in the pipe being nonfunctional and must be repaired before the immediate service area can be put back into service. A leak is simply a failure resulting in loss of water. A leak does not necessarily need to be restored immediately for the immediate service area to be put back into service. A leak versus a break is based on the ground deformation associated with each hazard parameter. PGD can range from inches to many feet, but PGV is typically fractions of an inch. Pipe with rigid joints such as cast iron pipe with leaded joints is particularly vulnerable to PGV, but pipe with elastomeric joints can absorb all but the very strongest PGV movements.

The results of the failure analysis are shown in Table 4-5 grouped by PGD- and PGV-related failures and leaks versus breaks.

Table 4-5 Pipeline Failures for PGD, PGV, and Total

PGD-Related Failures			PGV-Related Failures			
Breaks	Leaks	Total Failures (Breaks + Leaks)	Breaks	Leaks	Total Failures (Breaks + Leaks)	Total Failures (Breaks + Leaks)
3360	840	4200	11	46	57	4257

There is no firm threshold above which pipelines need to be replaced. The highest failure rates are typically a function of vulnerable pipe materials (e.g., cast iron) and soils subject to PGD (liquefiable soils). The number of estimated failures is an approximation based on empirical data and is intended to be used for planning purposes. The number of actual failures encountered may range from twice as many as those listed to half as many as those listed in the table. Geohazards identified for pipelines are based on large-scale mapping for seismic hazards. Site-specific surveys and aerial photographs should be used to estimate the potential for loss associated with landslides or liquefaction for specific pipeline alignments prior to undertaking a capital improvement plan (CIP) project.

4.2.3 Willamette River Crossing Vulnerabilities

Pipeline crossings of the Willamette River suspended from the Center Street and Marion Street Bridges are vital water supplies to West Salem and were observed as part of this project.

4.2.3.1 Center Street Bridge

Findings of Observation: It was found that the water main under the bridge is all flanged piping with rigid Victaulic couplings in some areas. The 24-inch inner diameter DI pipe is sliplined with 22-inch HDPE. The piping is suspended under the bridge with minimal bracing. On the east side of the bridge, possible flexible joints are present, but the piping was inaccessible for assessment. It was assumed that there are no flexible joints present on any aboveground piping. The piping suspended under the bridge was determined to be vulnerable, particularly because the bridge columns are supported on piles, while the pipe is supported by soil on either side, which results in differential settlement and separation during an earthquake.

Recommendations for Improvement: The pipeline should have flexible joints at either end (where the pipe exits or enters the soil) and at each bridge expansion joint to allow for differential settlement. In addition, between flexible joints, the pipe should be properly braced to the bridge deck. The City is currently scoping the replacement of this line as part of ODOT's seismic retrofit of the Center Street Bridge.

4.2.3.2 Marion Street Bridge

Findings of Observation: A similar on-site assessment was conducted for the Marion Street Bridge. Under the bridge, there were two large diameter pipes, one assumed to be the water main. One of the pipes was observed to have welded joints and the other pipe was flanged. No flexible joints were observed. The bridge piping was determined to be vulnerable to differential settlement and separation during an earthquake since the bridge columns are supported on piles, while the pipe is supported by soil on either side.

Recommendations for Improvement: Similar to the Center Street Bridge, this pipeline should have flexible joints at either end (where the pipe exits or enters the soil) and at each bridge expansion joint to allow for differential settlement. In addition, between flexible joints, the pipe should be properly braced to the bridge deck. According to ODOT's evaluation of this bridge, the structure is not expected to survive a CSZ level event, so further investment in the waterline may not be warranted unless the bridge is first seismically retrofitted or replaced.

4.3 Vertical Facilities Vulnerability Assessment

SEFT conducted a preliminary seismic assessment based on review of design documents and site visits for a selected group of vertical facilities, which include key pump stations, reservoirs, and control buildings. The findings of this assessment are included in the Pump Station and Reservoir Seismic Vulnerability Assessment Report by SEFT, which is located in Appendix C. The main objective of the vulnerability assessment for the facilities sites was two-fold:

- To identify deficiencies in each of the facilities that affect ability to maintain service in the event of a major earthquake (M9.0 CSZ scenario); and
- To develop preliminary recommendations for mitigation measures to address the identified deficiencies.

This planning-level Report is the first step in identifying and addressing seismic resiliency needs, and the findings of this study are intended to support City planning efforts when budgeting for and prioritizing facility seismic improvements.

4.3.1 Facility Assessment Summary

The seismic structural evaluations of pump stations, control facilities, and reservoir control buildings were completed using the Tier 1 screening procedure of American Society of Civil Engineers (ASCE) 41-17, Seismic Evaluation and Retrofit of Existing Buildings. This Tier 1 procedure uses a checklist-based approach to identify potential seismic structural deficiencies that have been commonly observed in past earthquakes. It also uses quick-check calculations to identify potential deficiencies in the primary components of the seismic lateral-force resisting system.

It is important to note that the Tier 1 assessment identified structural deficiencies that were confirmed, as well as structural deficiencies that were unconfirmed and to be evaluated in future Tier 2 assessments recommended for various facilities. It was not possible to confirm certain structural deficiencies that were identified in this Tier 1 assessment, because of the following reasons:

- Engineering drawings for several of the facilities were not available for review; therefore, preliminary conclusions were drawn based on observations of readily accessible portions of the facilities.

- The visual assessment was further limited as it was not possible to observe various structural elements (such as roof to wall connections which were concealed by the ceiling and/or insulation). These structural elements need to be inspected as part of a detailed investigation. The SEFT report in Appendix C identifies specific measures needed to perform the detailed evaluation.
- Detailed structural analyses need to be performed to determine the adequacy of certain elements such as reservoir column reinforcing lap splices. These analyses are beyond the scope of this study and need to be performed as part of a Tier 2 assessments.

Seismic nonstructural evaluations were completed using the nonstructural seismic evaluation checklists presented in ASCE 41-17 supplemented by the Technical Council on Lifeline Earthquake Engineering Monograph No. 22, Seismic Screening Checklists for Water and Wastewater Facilities*.* Like the ASCE 41 Tier 1 structural evaluation procedure, this checklist-based evaluation approach is used to identify potential seismic nonstructural deficiencies that have been commonly observed in past earthquakes.

4.3.2 Facility Seismic Deficiencies

Table 4-6 broadly summarizes the structural and nonstructural deficiencies identified at pump stations and control buildings. Typical pump station and control building deficiencies included inadequate roof to wall in-plane connections; inadequate roof to wall out-of-place bracing; inadequate piping, valve, or pump bracing; and unanchored control cabinets or unanchored electrical transformers. Table 4-7 summarizes the structural and nonstructural deficiencies at reservoirs. Typical reservoir deficiencies included insufficient reinforcing splice length on concrete columns, overstressed walls, lack of positive connections between roofs and walls, overstressed columns, lack of dowels or seismic cables at wall connections, and lack of positive connections between pipe pedestals and reservoir floors.

Table 4-6 Pump Station and Control Facility Deficiency Summary

	Structural Deficiencies								Nonstructural Deficiencies								
Vertical Facility	Roof Anchorage	Roof Design	Wall Bracing	Ceiling Design	Cracks in Walls	Masonry Wall Design	Liquefaction Hazard	Corrosion Damage	Pump Bracing	Pipe Bracing	Pipe Flexibility	Light Fixture Covers	Conduit Mobility	Electrical Cabinet Bracing	Misc. Element Bracing	Transformer Anchorage	SEFT Report Table Reference (Appendix C)
ASR # 1 and #2 Pump Station	■	■	■						■	■	■	■	■	■	■	■	3.1
ASR #4 Pump Station	■	■	■						■	■	■	■			■	■	3.2
ASR #5 Pump Station			■	■				■	■	■		■	■	■		■	3.3
Boone Road Pump Station	■	■	■						■	■	■	■			■	■	3.4
Creekside Pump Station	■	■	■						■	■	■			■	■		3.5
Deer Park Pump Station	■	■	■						■	■	■			■	■	■	3.6
Edwards Pump Station	■	■	■		■	■			■	■	■	■		■	■		3.7
Limelight Pump Station	■	■	■		■				■	■	■			■	■	■	3.8
Mountain View Pump Station	■	■	■						■	■	■	■		■	■	■	3.9
Salem/Keizer Intertie #1 Pump Station	■	■	■				■		■	■	■			■	■	■	3.10
Turner Control Facility	■	■	■				■		■	■	■			■	■	■	3.11
Champion Hill Control Building	■	■	■						■	■				■	■	■	3.14
Fairmount Control Building			■						■	■	■		■	■	■	■	3.17
Grice Hill Control Building		■	■							■				■	■	■	3.19
Lone Oak Control Building		■				■			■	■				■	■	■	3.21
Mill Creek #1 Control Building	■		■						■	■	■			■	■	■	3.23

Table 4-7 Reservoirs Deficiency Summary

Vertical Facility	Structural Deficiencies					Nonstructural Deficiencies				SEFT Report Table Reference (Appendix C)
	Reinforcement Lap Splice Length	Overstressed Walls	Roof to Wall or Wall to Foundation Connection	Concrete Cracking / Deterioration	Potential Liquefaction	Precast Vault	Cast Iron Pipe	Pipe Bracing	Miscellaneous Elements Bracing	
Candalaria Reservoir	■					■	■	■	■	3.12
Champion Hill Reservoir	■				■			■		3.13
Eola #1B Reservoir	■			■		■		■		3.15
Fairmount Reservoir	■	■	■				■			3.16
Grice Hill Reservoir	■							■		3.18
Lone Oak Reservoir										3.20
Mill Creek #1 Reservoir	■							■	■	3.22
Mountain View Reservoir	■	■	■				■			3.24

Table 4-8 summarizes the readiness of various facilities to meet immediate occupancy, operational, or life safety performance under a CSZ M9.0 earthquake. These performance objectives are defined as follows:

- Immediate Occupancy: "Immediate Occupancy" refers to the post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical- and lateral-force-resisting systems of the building retain almost all their pre-earthquake strength and stiffness. The risk of life-threatening injury from structural damage is very low, and although some minor structural repairs might be appropriate, these repairs would generally not be required before re-occupancy. Continued use of the building is not limited by its structural condition but might be limited by damage or disruption to nonstructural elements of the building, furnishings, or equipment and availability of external utility services.
- Operational: "Operational" refers to the performance level where most nonstructural systems required for normal use of the building are functional, although minor cleanup and repair of some items might be required. Achieving the Operational nonstructural performance level requires considerations of many elements beyond those that are normally within the sole province of the structural engineer's responsibilities. For Operational nonstructural performance, in addition to ensuring that nonstructural components are properly mounted and braced within the structure, it is often necessary to provide emergency standby equipment to

provide utility services from external sources that might be disrupted. It might also be necessary to perform qualification testing to ensure that all necessary equipment will function during or after strong shaking.

- Life Safety: "Life Safety" refers to the post-earthquake damage state in which significant damage to the structure has occurred but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this damage has not resulted in large falling debris hazards, either inside or outside the building. Injuries might occur during the earthquake; however, the overall risk of life-threatening injury as a result of structural damage is expected to be low. It should be possible to repair the structure; however, for economic reasons, this repair might not be practical. Although the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing before re-occupancy.

Most of the facilities do not meet the criteria for immediate occupancy, operational nonstructural performance, or life safety. Completion of the structural and nonstructural mitigation measures identified in the SEFT report will enable these facilities to meet these occupancy and safety criteria. The degree to which these facilities require mitigation (and the associated cost) vary significantly from one facility to the other as discussed later in this Report.

Table 4-8 Facility Assessment Summary

	Readiness to Meet M9.0 CSZ Earthquake		
Facility	**Immediate Occupancy Structural Performance**	**Operational Nonstructural Performance**	**Life Safety Structural Performance**
ASR # 1 and #2 Pump Station	No	No	No
ASR #4 Pump Station	No	No	No
ASR #5 Pump Station	No	No	No
Boone Road Pump Station	No	No	No
Creekside Pump Station	No	No	No
Deer Park Pump Station	No	No	No
Edwards Pump Station	No	No	No
Limelight Pump Station	No	No	No
Mountain View Pump Station	No	No	No
Salem/Keizer Intertie #1 Pump Station	No	No	No
Turner Control Facility	No	No	No
Candalaria Reservoir	No	No	N/A
Champion Hill Reservoir	No	No	N/A
Champion Hill Control Building	No	No	No
Eola #1B Reservoir	No	No	N/A
Fairmount Reservoir	No	No	N/A

Readiness to Meet M9.0 CSZ Earthquake			
Facility	Immediate Occupancy Structural Performance	Operational Nonstructural Performance	Life Safety Structural Performance
Fairmount Control Building	No	No	No
Grice Hill Reservoir	No	No	N/A
Grice Hill Control Building	No	No	No
Lone Oak Reservoir	Yes	Yes	N/A
Lone Oak Control Building	No	No	No
Mill Creek #1 Reservoir	No	No	N/A
Mill Creek #1 Control Building	No	No	No
Mountain View Reservoir	No	No	N/A

5.0 Water System Risk Assessment

5.1 Risk Assessment of Pipelines and Vertical Facilities

A risk assessment approach can support development and execution of a seismic rehabilitation and replacement capital improvement strategy. The risk assessment considers both the Consequence of Failure (COF) and Likelihood of Failure (LOF) of an asset to rank and prioritize that asset's overall risk. In the case of this Report, COF is a measure of the asset's criticality and LOF is a measure of the asset's vulnerability to seismic geohazards. Total risk for an asset is the LOF multiplied by the COF.

Together, the threat and vulnerability of an asset make up that asset's LOF. Assets that have a high LOF are those that have both a) physical vulnerabilities to seismic hazards and b) a high likelihood of seismic hazards. Assets that have a high COF are those that are part of the water system backbone and are critical to supporting fire flow and the critical social/economic needs of the community during the Short-Term Recovery Phase in the initial days following a CSZ earthquake. It is recommended that high risk assets are given higher priority for replacement/retrofit over lower risk assets.

An asset's risk score is calculated based by multiplying its LOF by its COF. The risk score may range from 1 to 25, as shown on Figure 5-1.

RISK		LIKELIHOOD				
		1	2	3	4	5
CONSEQUENCE	5	5	10	15	20	25
	4	4	8	12	16	20
	3	3	6	9	12	15
	2	2	4	6	8	10
	1	1	2	3	4	5

Risk
Very High (25)
High (20)
Moderate to High (15 – 16)
Moderate (10 – 12)
Low to Moderate (8 – 9)
Low (1 – 5)

Figure 5-1 Range of Potential Risk Scores

5.1.1 Consequence of Failure

The COF score for each asset (facility or pipeline segment) is equal to its criticality level. Criticality levels were assigned when establishing the system backbone (refer to Section 3.0, Water System Backbone Definition). COF values range from 1 to 5, as listed below:

- 5 – Highly Critical
- 4 – Critical
- 3 – Semi Critical
- 2 – Local Critical
- 1 – Not Critical/Redundant (not part of the system backbone)

A summary of COF scores for the City's backbone facilities is presented in Table 3-4 (as characterized by the numerical Criticality Level). COF values for pipelines are assigned in a GIS database. Pipeline segment COF scores were coordinated with the COF scores of vertical facilities they connect to, because these pipelines and vertical facilities are interdependent.

5.1.2 Likelihood of Failure for Pipelines

A LOF score was assigned to each pipeline segment, based upon the number of breaks per 1,000 feet within that pipe segment, as shown in Table 5-1.

Table 5-1 Likelihood of Failure Scores for Pipelines

Likelihood of Failure	Breaks per 1,000 Feet	Percentage of Pipelines	Miles of Pipeline
Low (1)	<0.1	29.1%	**230**
Low to Moderate (2)	0.10 – 0.69	23.4%	**185**
Moderate (3)	0.7 – 0.89	19.2%	**161**
Moderate to High (4)	0.90 – 1.39	16.6%	**115**
High (5)	1.40 – 3.01	13.2%	**99**

5.1.3 Risk Assessment for Vertical Facilities

The LOF, COF, and risk scores for the 22 vertical facilities that were assessed in this project are summarized in Table 5-2.

Table 5-2 Risk Assessment for Vertical Facilities

Facility	Service Level	Potential Liquefaction	Potential Landslide	LOF	COF	Risk	
Fairmount Reservoir	G-0			5	5	25	Very High
Mountain View Reservoir	G-0			4	5	20	High
Deer Park Pump Station	S-1			5	4	20	High
Edwards Pump Station	S-1, S-2	■		5	4	20	High
Turner Control Facility	G-0	■		4	5	20	High
Lower Croisan Pump Station	S-2	■	■	5	4	20	High
Mountain View Pump Station	W-1			4	4	16	Moderate to High
ASR #1 and #2 Wells	S-2			4	4	16	Moderate to High
ASR #5 Well	S-2			4	4	16	Moderate to High
Salem/Keizer Intertie #1	G-0	■		4	4	16	Moderate to High

Facility	Service Level	Potential Liquefaction	Potential Landslide	LOF	COF	Risk	
Boone Road Pump Station	S-2			4	4	16	Moderate to High
Champion Hill Reservoir Control Building	S-3	■		4	4	16	Moderate to High
Grice Hill Reservoir Control Building	W-1			3	4	12	Moderate
ASR #4 Well	S-2			3	4	12	Moderate
Candalaria Reservoir	S-1			3	4	12	Moderate
Champion Hill Reservoir	S-3	■		3	4	12	Moderate
Lone Oak Reservoir Control Building	S-2			3	4	12	Moderate
Mill Creek #1 Reservoir Control Building	MCCC S-1			3	4	12	Moderate
Creekside Pump Station	S-3			3	4	12	Moderate
Fairmount Pump Station	S-2			5	1	5	Low

6.0 Water System Risk Mitigation Plan

This section describes the phasing of recommended improvements to address higher risk assets and rapidly restore water service after a major earthquake to meet social and economic needs. Improvements include replacement and hardening of pipelines and correction of deficiences for vertical facilities which were identified through the vulnerability assessment. This risk mitigation plan leverages knowledge of pipeline and facility seismic vulnerabilities to develop a long-term plan for implementing water system seismic resilience improvements. Recommendations are provided in 15 to 30 year phases to allow the flexibly to incorporate these recommendations into the City's capital improvement plan.

6.1 Capital Program Prioritization Methodology

The project team developed priorities for the short, medium, and long-term CIP for seismic improvements in close consultation with City staff. The recommended risk mitigation efforts are informed by the City's LOS goals. This prioritization is summarized in Table 6-1.

Table 6-1 Capital Program Terms and Priorities

Term	Priority	Risk Level of Facilities to Be Improved	Risk Level of Pipelines to Be Improved
Short (0 – 15 Years)	1. Preserve Water in the System 2. Convey Treated Water 3. Implement Alternative Supplies 4. Complete Studies to Refine Understanding of Expected System Performance	Very High	Very High
		High	High
		Moderate to High	Moderate to High
		Moderate	
Medium (10 – 25 Years)	5. Harden the Rest of the Backbone	Low to Moderate	Moderate
		Low	Low to Moderate
Long (20 – 50 years)	6. Harden Distribution System to Reduce the Number of Repairs	-	Low

In the **short term**, the City should focus on implementing mitigation that will help to preserve water in the system after an earthquake or to convey water to the backbone after an earthquake. As a priority, the City should implement the following strategies:

- Installation of seismic isolation valves installed at all reservoirs (the City already has seismic valves installed on a significant number of them) and seismic upgrades on the "very high" to "moderate" risk reservoirs and their control buildings.
- Seismic upgrades to pump stations which are appurtenant to reservoirs.

The City should also focus on conveyance of treated water to the backbone by hardening the transmission lines from Geren Island WTP to critical reservoirs, including to West Salem. The City should also implement providing alternative water supplies within this phase. Alternative local water supply development (such as drilling of new wells to access groundwater supplies) will provide additional supply reliability in the case of an emergency. The City should also complete studies to understand system hazards at vertical facilities not assessed as part of this study, such as Franzen Reservoir. As part of the short-term phase, all "moderate" to "very high" risk facilities should be seismically improved and all "moderate to high" to "very high" risk pipelines should be hardened.

In the **medium term**, the City should focus on hardening the rest of the backbone system so that the system will remain operational following a major earthquake. "Low to moderate" and "low" risk facilities should be seismically improved and "moderate" and "low to moderate" risk pipelines (all remaining pipelines within the backbone system) should be hardened.

In the **long term**, the City should focus on hardening the rest of the distribution system to address the LOS goals discussed in Section 2.0. The City aims to serve a minimum of 80% of all customers within 1 to 2 weeks following a M9.0 CSZ earthquake. A limited number of breaks and leaks can be repaired by City crews in the days and weeks following an earthquake. To reduce the number of breaks and leaks down to an amount that can be quickly repaired by the City following an earthquake, and to meet the LOS goals, the City should need to replace most "low" risk pipelines.

6.2 Basis for Establishing Opinion of Probable Construction Costs

An Opinion of Probable Construction Cost (OPCC) was developed for each of the major vertical facilities and buried infrastructure identified in this Report. The OPCC was developed to the Class 5 (conceptual) level of accuracy, as defined by the Association for the Advancement of Cost Engineering (AACE), and expected to have an accuracy range from -30% to +50% of actual (2022) costs.

6.2.1 OPCC Assumptions for Pipelines

To develop the OPCC for pipelines, unit costs were developed using 1,000 linear feet (LF) of waterline. Three different pipe depths and sizes were used, and the costs were averaged to develop representative waterline replacement costs. The following items were included in the OPCC for pipelines:

- Mobilization.
- Insurance and bonds.
- System ties.
- Shoring for jacking pits.
- Corrosion protection.
- Cathodic protection.
- Fittings allowance.
- Pavement demolition and replacement over the top of the waterline.

Markups associated with the OPCC for pipelines varied depending on whether the pipelines were at rail, highway, waterway crossings, or not at any of these crossings, as shown in Table 6-2.

Table 6-2 Markups Associated with OPCC for Pipelines

Cost Component	Pipelines	Rail & Highway Crossings	Waterway Crossings
Contingencies[1]	40%	40%	40%
Professional Services[1]			
Engineering	10%	15%	20%
Construction Management and Inspection	10%	10%	10%
Permitting	5%	8%	10%
City Administration, Public Outreach, and Legal	8%	8%	8%

[1] Excludes right-of-way acquisition.

The following items were not included in the water pipeline OPCC:

- Fire hydrant with a gate valve and 6-inch fire service replacement, tracer wires, and butterfly valves. These are anticipated to be a minor additional cost (<$5,000 on a 1,000 LF waterline replacement contract) to the project and generally covered by the "fittings allowance" or cost contingencies.
- Program costs (such as City staffing).
- Service line replacements – City staff noted that concurrent with replacement of the water mains, all service lines to the meter connection are also replaced. The waterline database used to develop the water pipeline OPCC had 2.5 miles of pipes that are 1.5 inches in diameter or smaller, and included pipes as small as 3/4-inch diameter. Therefore, it is possible that the service lines are already included, to a degree, in the pipeline database. Service line replacements were not explicitly included in the cost estimate to avoid any double-counting.

6.2.2 OPCC Assumptions for Vertical Facilities

The OPCC for vertical facilities is based on the detailed recommendations provided in the Technical Memorandum, Pump Station and Reservoir Seismic Vulnerability Assessment (September 6th, 2021), provided in Appendix C. As detailed engineering layouts of the proposed improvements were not available due to the conceptual nature of this study, the OPCC is largely based on parametric factoring of known costs for similar systems and analogous projects with comparable corresponding features and sizing. The OPCC for the vertical facilities sites is based the estimating allowances and contingencies noted in Table 6-3.

Table 6-3 OPCC Markups for Vertical Facilities

Cost Component	Contingency Applied To	Vertical Facilities Contingency
Contractor and subcontractor overhead and profit (OH&P), including market condition due to current labor availability and supply chain issues; mobilization, general conditions and field overhead expense	Direct construction cost (labor, materials, and equipment)	Base cost
Construction contingency	Direct construction cost, after OH&P	30%
Professional services[1] Engineering, construction management, and inspection	Construction cost[2]	30-40%
Additional contingency at Mountain View and Fairmount Reservoir, due to complexity of improvements	Construction and engineering costs	$200,000

[1] Excludes right-of-way acquisition.

[2] Direct construction cost, after OH&P and construction contingency.

6.3 Pipeline System Prioritization and Cost Projections

6.3.1 Prioritization Approach

Pipeline work is prioritized based on risk using a combination of the LOF and COF scores. This resulted in the suggested phasing of improvements shown in Table 6-1. A summary of pipeline breaks and pipe length in miles for each LOF and COF is provided in Table 6-4, which is color-coded as follows:

- Red represents "high risk" and "very high risk" pipelines that have a COF and LOF of 5. "High risk" pipelines are those in pink that are not classified as "very high risk" and have an LOF of 4 paired with a COF of 5, or a COF of 4 paired with an LOF of 5.
- Orange represents "moderate to high risk" pipelines.
- Yellow represents "moderate risk" pipelines.
- Dark green represents "low risk" pipelines.
- Cells that are not color coded represent "very low risk" pipelines that have a COF of 1. These pipelines are not part of the system backbone.

Table 6-4 Pipeline Risk Matrix

	LOF									
	1		2		3		4		5	
COF	Breaks	Pipe Length (miles)	Breaks	Pipe Length (miles)	Breaks	Pipe Length (miles)	Breaks	Pipe Length (miles)	Breaks	Pipe Length (miles)
5	0	3	13	5	8	2	104	17	91	10
4	0	18	52	22	57	13	101	17	69	8
3	0	3	2	1	4	1	24	4	48	5
2	0	5	12	5	8	2	16	3	11	1
1	3	230	416	185	704	161	692	115	936	99

Pipelines with an LOF of 1 do not require hardening because the number of breaks are anticipated to be almost zero due to either the low potential for seismic geohazards and/or the high anticipated resilience of that pipeline to withstand earthquake damage. Pipes that have an LOF of 1 and a COF of 1 represent 230 miles of pipeline, but are only forecasted to have three breaks, which can be repaired quickly by staff following an earthquake.

Pipelines with an LOF of 2 and a COF of 1 should be the City's lowest priority for hardening. These pipes represent 185 miles of pipeline, which equate to approximately 20% of the City's pipeline system by length. The LOS goals allow for a longer duration of time for bringing 20% of customers back into operation following a CSZ earthquake, which gives the City time to repair leaks and breaks as needed to restore system operation. Therefore, for this risk category, pipe replacement was not included in the cost projections. Twenty percent of the City's pipeline system does not equate to 20% of customers served, but length of pipe in miles was used as a surrogate until the City develops a more in-depth analysis.

Within a given risk level, the City could further prioritize replacement based upon the existing pipeline materials using the K1 and K2 values from Table 4-4 (i.e., prioritize replacement of pipeline materials with higher K1 and K2 values over pipelines materials with lower K1 and K2 values). For example, cast iron pipe has historically been highly vulnerable to both PGD and PGV/shaking, because it is brittle and succeptible to cracking. The joints are typically leaded and rigid. Even small movements will cause them to leak. Larger movements cause the pipe bells to break and/or the joints to separate.

6.3.2 Pipeline Mitigations

Pipeline joint systems and materials heavily influence a pipeline's ability to withstand the effects of earthquakes. Pipeline joints within seismically vulnerable areas should be designed to allow movement and/or deformation without joint failure when subjected to seismic forces. Pipe material should be designed to withstand shear and compression forces without local buckling. The overall system (joints and pipe material) should accommodate a certain amount of strain. Table 6-5 presents the recommended approach for selection of various pipe materials under different conditions.

Table 6-5 Pipe Replacement Material Selection

Selection Criteria	Steel (Butt Welded)	Steel (Lap Welded)	DIP (Earthquake Resistant Joints)[6]	DIP (Mechanically Restrained Joints, not Wedges)	HDPE	PVCO[7] with Seismic Restrained Joints	PVCO[7] with Double Depth Bell
Cost per inch-Diameter/LF ($)[1]	45	54	54	45	32	15	15
Highway, creek, or rail crossing[2]	■	■	■	■	■	■	■
48" diameter or greater	■		■				
24"≤ diameter < 48"	■	■	■	■	■		
12"< diameter <24"	■	■	■	■	■		
12" diameter or smaller[3]	■	■	■	■	■	■	■
PGD > 4"	■		■	COF ≤ 3	■	■	COF ≤ 3
Corrosive soil conditions[4]	With corrosion protection	With corrosion protection	With corrosion protection	With corrosion protection	■	■	■
"Very Strong" ground shaking[5]	■	COF ≤ 3	■	COF ≤ 3	■	■	COF ≤ 3

[1] Does not include contingencies or engineering costs.

[2] Additional costs associated with trenchless construction.

[3] Except service lines, which are generally constructed of copper tubing.

[4] Steel corrosion potential is moderate or high, according to mapped corrosion of steel potential from the United States Department of Agriculture Natural Resources Conservation Service Web Soil Survey. https://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm

[5] Very strong ground shaking is characteristic of areas which have PGVs higher than 24 inches per second. Pipelines in the City are not anticipated to have PGV values higher than 24 inches per second.

[6] Earthquake resistant joints are restrained but allow longitudinal movement.

[7] PVCO is molecularly-oriented PVC (AWWA C-909).

Empirical leak and break rates associated with modern piping alternatives, such as welded steel, earthquake-restrained ductile iron pipe (DIP), mechanically restrained ductile iron pipe, high density polyethylene (HDPE), and molecular-oriented polyvinyl chloride (PVCO) are not readily available. Much more research on leaks and breaks is available for historic piping materials such as ductile iron, cast iron, and asbestos concrete, which characterize the majority of most water distribution systems. Therefore, it is difficult to quantify the impact of replacing older pipe materials with newer pipe materials. Future research, conducted following future earthquakes in areas that have seismically hardened systems, can help to clarify break rates associated with various modern joint and material systems.

PVCO has successfully undergone extreme earthquake testing at the seismic pipe lab at Cornell University. PVC (AWWA C-900) pipe is inherently brittle and has been known for cracks to propagate the full length of the pipe in non-earthquake conditions. PVC was installed in Christchurch, New Zealand, and subjected to the 2011 earthquake. It suffered significant damage which resulted in many utilities transitioning to use of HDPE or PVCO in liquefiable soils.

Another area of emerging research is to what degree cured-in-place pipe (CIPP) provides seismic resilience. Rehabilitating a pipe with CIPP is a cost-effective means of extending a pipeline's expected useful life. A CIPP liner converts a jointed pipe to a continuous pipeline, and more joint stability can reduce the rate of breaks and leaks, resulting in less potential for damage during an earthquake. City staff are encouraged to keep track of pipelines that are already CIPP-rehabilitated when prioritizing pipes for repair and stay on top of current research regarding CIPP and seismic performance.

6.3.3 Cost Projections

For the purposes of developing the cost projections for this Report, assumptions were made about the pipeline replacement material, as shown in Table 6-6. It is noted that PVC and HDPE do not currently meet City design standards, but they are more cost-effective than steel and ductile iron pipes. Due to the large number of pipes that would need to be replaced to support the system backbone and distribution system as a whole in the event of a CSZ earthquake, the City should consider using these materials in seismically vulnerable areas, if they are appropriate for the site conditions, to reduce costs. The actual pipeline material selected for replacement will be determined later, during design of the pipeline improvements. The costs for pipeline system improvements for the Center Street Bridges are not included in the cost projections.

Table 6-6 OPCC Assumed Replacement Materials for Pipelines

Pipe Size	Assumed Replacement Material
Mains (≤12")	PVC C-909 Brute Deep Bell
Distribution Pipelines (>12" and ≤42")	HDPE
Transmission Pipelines (>42")	Steel Pipe Butt Weld

A summary of the anticipated pipeline replacement costs in each risk category (not including replacement of LOF 1 pipes, which are not anticipated to fail, and COF 1/LOF 2 pipes, which should be repaired following an earthquake) is presented on Figure 6-1.

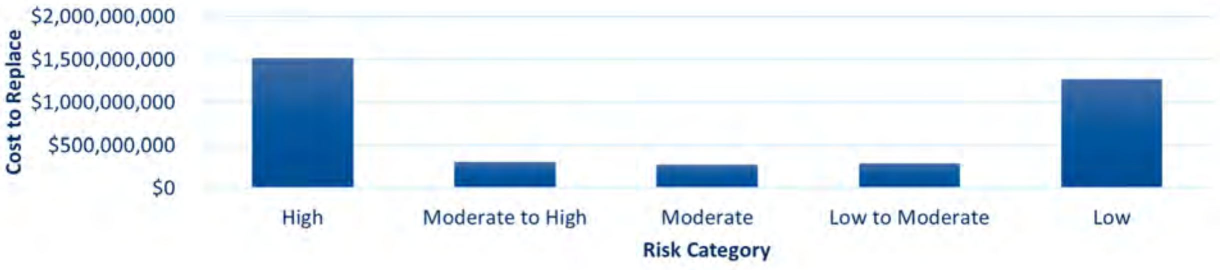


Figure 6-1 Summary of Pipeline Costs in Each Risk Category

6.4 Vertical Facilities Prioritization and Cost Projections

6.4.1 Prioritization Approach

Vertical facilities are prioritized using their risk scores, as shown in Table 5-2. Suggested phasing of improvements to these facilities is shown in Table 6-1. In the near term (in the earlier part of the Short-Term phase of Table 6-1), it is recommended that the City implement a seismic retrofit program to address life safety seismic deficiencies for water system structures that are frequently accessed by City staff and contractors.

6.4.2 Vertical Facility Mitigations

An approximate, high-level summary of recommended vertical facility mitigations are presented in Table 6-7 and Table 6-8. Refer to Section 4.0 of Appendix C for full mitigation concepts and details.

Table 6-7 Summary of Recommended Mitigations Measures at Reservoirs

Reservoir	Summary of Recommendations
Candalaria	• Perform an ASCE 41 Tier 2 assessment on the reservoir column reinforcement. • Install stainless steel plates to connect riser, base, and lid to the precast construction joints in the vault. • Repair any leaking precast joints with polyurethane resin or similar method in the vault. • Verify pipe materials in the reservoir. • Evaluate the adequacy of the overflow pipe and valve operator rise shafts to resist seismic forces in the vault. • Install lateral bracing of the overflow pipe and valve operator riser shafts in the vault. • Verify pipe and pump bracing in the vault, install as required.
Champion Hill	Reservoir • Perform a geotechnical study to evaluate liquefaction hazard. • Perform an ASCE Tier 2 assessment on the reservoir column reinforcement. • Anchor pipe support pedestals.
	Control Building • Perform a geotechnical study to evaluate liquefaction hazard. • Install new shaped blocking and Simpson A35 clips for masonry walls and roof truss connection. • Install blocking support and boundary nailing to support roof sheathing. • Install metal connector hardware to provide a vertical connection between the roof trusses and kicker brace frames. • Provide pipe, pump, and additional bracing for building elements. • Install flexible couplings between the pumps and connected piping. • Install blocking and metal connector hardware to provide connection from ceiling to walls for seismic force transfer.

Reservoir	Summary of Recommendations
Eola #1B	• Investigate extent and impact of circumferential concrete cracks. • Perform an ASCE Tier 2 assessment on the reservoir column reinforcement. • Investigate concrete deterioration near the lid connection of the valve vault. • Install stainless steel plates to connect riser, base, and lid to the precast construction joints in the vault. • Repair any leaking precast joints with polyurethane resin or similar method in the vault. • Assess pipe and valve's adequacy to resist seismic force.
Fairmount	Reservoir • Add 6-inch layer of shotcrete at the inside face of the perimeter walls and footings. • Install stainless steel connections along the roof expansion joints. • Install anchors between roof slab and the walls. • Investigate interaction between the Fairmont Reservoir and the Fairmont Reservoir Control Building. • Perform an ASCE Tier 2 assessment on the reservoir column reinforcement. • Verify pipe materials in the reservoir.
	Control Building • Conduct detailed structural seismic assessment. • Provide pipe, pump, and additional bracing for building elements. • Install flexible couplings between the pumps and connected piping. • Verify connection of the motor to the top of the steel motor support. • Replace any cast iron pipe and fittings. • Replace any piping, valves, or fittings with corrosion damage.
Grice Hill	Reservoir • Perform an ASCE Tier 2 assessment on the reservoir column reinforcement. • Install connection brackets to anchor pipe support pedestals.
	Control Building • Install new shaped blocking and Simpson A35 clips for masonry walls and roof truss connection. • Install blocking support and boundary nailing to support roof sheathing. • Install metal connector hardware to provide a vertical connection between the roof trusses and kicker brace frames. • Provide pipe, pump, and additional bracing for building elements. • Install flexible couplings between the pumps and connected piping.
Lone Oak	Control Building • Source design drawings and calculations and preform a follow up ASCE 41 Tier 1 evaluation. • Provide pipe, pump, and additional bracing for building elements.

Reservoir	Summary of Recommendations
Mill Creek #1	Reservoir • Perform an ASCE Tier 2 assessment on the reservoir column reinforcement. • Install connection brackets to anchor pipe support pedestals. • Install diagonal bracing between stair landing support posts.
	Control Building • Install new shaped blocking and Simpson A35 clips for masonry walls and roof truss connection. • Install blocking support and boundary nailing to support roof sheathing. • Source design drawings and evaluate the adequacy of the load path from the roof to the masonry walls. • Provide pipe, pump, and additional bracing for building elements. • Install flexible couplings between the pumps and connected piping. • Install blocking and metal connector hardware to provide connection from ceiling to walls for seismic force transfer.
Mountain View	• Install seismic restraint between the reservoir walls and foundation. • Operate the reservoir at a lower maximum elevation to reduce hydrodynamic forces and avoid a seismic retrofit. Or • Re-wrap the core wall with circumferential prestressing strands encased with shotcrete. • Install fiber reinforced polymer wrapping around columns. • Verify pipe material.

Note: This table is not fully inclusive. Refer to Section 4.0 of Appendix C for full mitigation concepts and details.

Table 6-8 Summary of Pump Station and Control Facilities Recommendations

Pump Station/ Control Facility	Summary of Recommendations
ASR #1 and #2	• Verify load path at roof step between the masonry walls. • Verify roof sheathing. • Install vertical steel angles where the east-west concrete masonry unit (CMU) walls interface with west wall of ASR #1 structure. • Install new shaped blocking and Simpson A35 clips for masonry walls and roof truss connection. • Verify masonry wall vertical reinforcement. • Provide pipe, pump, and additional bracing for building elements. • Install flexible couplings between the pumps and connected piping. • Verify concrete pillar reinforcement adequacy.
ASR #4	• Verify roof sheathing. • Investigate roof diaphragm capacity to transfer seismic forces due to hatch. • Install new shaped blocking and Simpson A35 clips for masonry walls and roof truss connection. • Verify masonry wall vertical reinforcement. • Provide pipe, pump, and additional bracing for building elements. • Install flexible couplings between the pumps and connected piping.

Pump Station/ Control Facility	Summary of Recommendations
ASR #5	• Install new shaped blocking and Simpson A35 clips for masonry walls and roof truss connection. • Investigate ceiling diaphragm connection to masonry walls. • Verify ceiling nail size and spacing. • Verify masonry wall vertical reinforcement. • Investigate the adequacy of free-standing masonry wall to resist seismic forces without additional bracing. • Investigate extent of corrosion damage to steel column and repair. • Provide pipe, pump, and additional bracing for building elements. • Install flexible couplings between the pumps and connected piping.
Boone Road	• Investigate gable end framing, sheathing nailing, and connection details to roof. • Install wood panel overlay to existing sheathing. • Install sub-diaphragm framing and connection hardware to repair roof and wall bracing. • Provide pipe, pump, and additional bracing for building elements. • Install flexible couplings between the pumps and connected piping. • Verify connection of the motor to the top of the steel motor support.
Creekside	• Verify existing roof sheathing to truss nailing. • Verify roof to masonry wall connection and install new shaped blocking and Simpson A35 clips for masonry walls and roof truss connection. • Install plywood, blocking, steel straps, and metal connector hardware to repair CMU wall bracing. • Verify the roof sheathing and gable end masonry wall op plate connection. • Provide pipe, pump, and additional bracing for building elements. • Install flexible couplings between the pumps and connected piping. • Verify connection of the motor to the top of the steel motor support.
Deer Park	• Verify the size and location of masonry wall reinforcement. • Replace roof and install out-of-plane bracing to perimeter and interior masonry walls. • Provide pipe, pump, and additional bracing for building elements. • Install flexible couplings between the pumps and connected piping.
Edwards	• Perform a geotechnical study to investigate liquefaction and lateral spreading. • Replace the entire structure. • Provide pipe, pump, and additional bracing for building elements. • Install flexible couplings between the pumps and connected piping. • Verify connection of the motor to the top of the steel motor support.
Limelight	• Investigate extent and impact of vertical cracks in masonry shear walls. • Verify roof sheathing and truss nailing. • Install new shaped blocking and Simpson A35 clips for masonry walls and roof truss connection. • Install plywood, blocking, steel straps, and metal connector hardware to repair CMU wall bracing. • Provide pipe, pump, and additional bracing for building elements. • Install flexible couplings between the pumps and connected piping.

Pump Station/ Control Facility	Summary of Recommendations
Mountain View	• Install plywood/sheathing, framing/blocking, and connector hardware to provide a load path between the roof and interior masonry walls. • Verify roof to masonry wall connection and install new shaped blocking and Simpson A35 clips for masonry walls and roof truss connection. • Install plywood, blocking, steel straps, and metal connector hardware to repair CMU wall bracing. • Provide pipe, pump, and additional bracing for building elements. • Install flexible couplings between the pumps and connected piping. • Verify connection of the motor to the top of the steel motor support. • Install anchorage/positive connection between the strut and masonry shear wall for seismic demands.
Salem/Keizer Intertie #1	• Perform a geotechnical study to investigate liquefaction and lateral spreading. • Investigate the gap between the City pump station and the City of Keizer building. • Install shaped blocking and boundary nailing to correct the gap in the roof sheathing. • Install new shaped blocking and Simpson A35 clips for masonry walls and roof truss connection. • Install flexible joints where water system piping penetrates through the pump station floor. • Install flexible couplings between the pumps and connected piping. • Verify connection of the motor to the top of the steel motor support. • Install anchorage/positive connection between the strut and masonry shear wall for seismic demands.
Turner Control Facility	• Perform a geotechnical study to investigate liquefaction and lateral spreading. • Verify the roof sheathing to masonry wall top plate connections. • Install fasteners between roof sheathing and outrigger. • Perform a geotechnical study to investigate liquefaction and lateral spreading. • Install flexible couplings between the pumps and connected piping. • Verify connection of the motor to the top of the steel motor support. • Install anchorage/positive connection between the strut and masonry shear wall for seismic demands.

Note: This table is not fully inclusive. Refer to Section 4.0 of Appendix C for full mitigation concepts and details.

6.4.3 Cost Projections

For each vertical facility assessed, costs were developed for (1) addressing known issues identified through the seismic vulnerability assessment, (2) additional studies recommended by the seismic vulnerability assessment, and (3) work identified from additional studies. These costs are broken into short and medium term CIP phases in Table 6-9.

Table 6-9 Short- and Medium-Term Vertical Facility CIP Projections

Facility	Known Issues	Additional Studies	Potential Additional Work[1]	Total
Short-Term CIP (Years 0-15)				
ASR 1&2	$180,000	$49,000	$100,000	$329,000
ASR 4	$100,000	$36,000	None	$136,000
ASR 5	$60,000	$65,000	$170,000	$295,000
Creekside PS	$120,000	$94,000	$80,000	$294,000
Deer Park PS	$130,000	$62,000	$190,000	$382,000
Mountain View PS	$230,000	$11,000	$30,000	$271,000
Salem Keiser Intertie #1	$140,000	$21,000	$10,000	$171,000
Turner Control Facility	$70,000	$29,000	$100,000	$199,000
Candalaria Reservoir	$10,000	$101,000	$240,000	$351,000
Champion Hill Reservoir	$100,000	$8,000	None	$108,000
Champion Hill Reservoir Control Bldg	$180,000	$6,000	$10,000	$196,000
Edwards PS	$190,000	$11,000	$810,000	$1,011,000
Fairmount Reservoir	$2,650,000	$29,000	$390,000	$2,869,000
Fairmount Res. Control Bldg	$140,000	$18,000	$30,000	$188,000
Grice Hill Res Control Bldg	$150,000	None	None	$150,000
Lone Oak Res. Cntrl Bldg	$30,000	$44,000	$10,000	$84,000
Mill Creek Reservoir	$40,000	$8,000	$940,000	$988,000
Mill Creek#1 Res. Cntrl. Bldg	$60,000	$44,000	$150,000	$254,000
Mountain View Reservoir	$3,790,000	None	$70,000	$3,660,000
Eolia 1B Seismic Valve	$200,000	None	None	$200,000
Subtotal – Short-Term CIP	**$8,570,000**	**$636,000**	**$3,330,000**	**$12,136,000**
Medium-Term CIP (Years 20-30)				
Boone Road PS	$110,000	$25,000	$140,000	$275,000
Limelight PS	$100,000	$67,000	$310,000	$477,000
Eola #1B Reservoir	$80,000	$8,000	$20,000	$108,000
Grice Hill Reservoir	$20,000	None	$20,000	$40,000
Lone Oak Reservoir	None	None	None	None
Subtotal – Medium-Term CIP	**$310,000**	**$100,000**	**$490,000**	**$900,000**
Total CIP	$8.88M	$0.74M	$3.82M	$13.04M

[1] This includes estimated costs for remedial measures that may arise from the additional studies; these additional studies would further define the nature, extent, and cost of this remedial work.

6.5 Seismic Capital Recommendations Summary

A summary of the priorities and total costs for the short, medium, and long term are presented in Table 6-10.

Table 6-10 Seismic Improvements Phasing and Cost Summary

Term	Priority	Risk Level of Facilities to Be Improved	Risk Level of Pipelines to Be Improved
Short (0 – 15 Years)	1. Preserve Water in the System 2. Convey Treated Water 3. Implement Alternative Supplies 4. Complete Studies to Understand System Hazards	Very High	Very High
		High	High
		Moderate to High	Moderate to High
		Moderate	
Total Cost (Short Term)		**$8.61 - 12M**	**$1.82B**
Medium (10 – 25 Years)	5. Harden the Rest of the Backbone	Low to Moderate	Moderate
		Low	Low to Moderate
Total Cost (Medium Term)		**$0.41 - 0.90M**	**$0.56B**
Long (20 – 50 years)	6. Harden Distribution System to Reduce the Number of Repairs	-	Low
Total Cost (Long Term)		**$0**	**$1.27B**

6.6 Opportunities for Further Study and System Improvements

It is recommended that the City consider the following noncapital improvements to further mitigate the risk of a CSZ earthquake:

- **Emergency Contractors**. Staffing shortages and the ability of the City to mobilize contractors can impact the City's ability to respond to an emergency. The provision of standing emergency contracts with pipeline contractors and maintaining adequate staffing levels can help to improve the City's resilience and promote a quicker response to an emergency. It is recommended that the City consider the use of emergency contracts.
- **Public Emergency Preparedness**. The public can take certain steps to mitigate the impacts of a natural disaster. For example, maintaining a 2-week water supply and understanding where to find an emergency shelter are two steps that can mitigate the impact of an earthquake. Public outreach can help to promote preparedness.
- **Funding Assessment**. It is recommended that the City conduct a funding assessment and apply for alternative financing to support seismic resiliency improvements. It is also recommended that the City analyze staffing and funding constraints to help fully develop a sustainable program.
- **Seismic Upgrade Program**. It is recommended that the City develop a program for transmission pipelines and distribution pipelines which specify replacement materials to be used to promote seismic resiliency if those pipes are at risk of damage during a CSZ earthquake. It is recommended that new subdivisions that are developed in seismically vulnerable areas use seismically resistant materials for new pipelines. It is noted that PVC and HDPE do not currently meet City design standards, but they are more cost-effective than steel and ductile iron pipes.

The City should consider revising the City standards to allow use of these materials in seismically vulnerable areas if they are appropriate for the site conditions.

- **Integrate Seismic Vulnerabilities with Water Master Plan**. The City is undertaking a Water Master Plan that will identify hydraulic and structural deficiencies. It is recommended that the pipelines and vertical facilities that are identified as high priority in this seismic resiliency study be similarly prioritized in the master plan. Furthermore, it is recommended that system outage scenarios and their impact to the City's backbone system are evaluated in the master plan or as a separate effort.

It is also recommended that the City consider the following future studies and system improvements to further mitigate the risk of a CSZ earthquake:

- **Valve Isolation Analysis**. The system can be modeled to determine: (1) valves that must be closed to isolate the backbone and (2) how to prioritize those valves considering the number of valves that can be closed each day in an emergency. The number of valves that can be closed in a day depends upon the number of field crews that are available during an emergency to perform this service. A valve isolation analysis can be used to develop a workflow and strategy for valve isolation and should consider both valve and hydrant flushing.
- **Operations & Maintenance (O&M) Inventory Assessment**. An O&M inventory assessment will enable the City to know what inventory of materials should be kept on hand in case of an emergency, such as pipe clamps, couplings, pipe materials, or chemicals. Materials stored for an emergency must also be stored safely in case they are not needed for a long time. Storage of materials can be costly, especially for large diameter pipes, but it is useful to have some materials on hand so that supply chain delays will not have large impacts on the City's disaster response. This assessment can also review the use of inflatable pipes on a temporary basis, such as to direct water across a street.
- **Center Street Bridge Improvements Design**. As a follow up to the recommendations provided in Subection 4.2.3 for the Center Street Bridge, additional design work is necessary to improve the pipeline performance during an earthquake and be able to adapt to differential settlement without pipe failure. The City is currently scoping the replacement of this line as part of ODOT's seismic retrofit of Center Street Bridge.
- **Development of Alternative Water Supplies**. It is recommended that the City consider implementation of alternative water supplies. The City currently operates four ASR wells and is considering constructing emergency well at additional locations. Wells located near the City's critical customers can offset some of the demand on the distribution system, which will not be hardened in the short term except for "very high," "high," and "moderate to high" pipeline segments. Because wells located in liquefiable soils are prone to seismic failure , it is recommended that the City site wells in areas with low liquefaction and landslide potential to safeguard the integrity of these wells during a seismic event.
- **Seismic Evaluation of Remaining Water System Structures**. It is recommended that the City conduct seismic evaluations of the remaining inventory of water system structures (pump stations, reservoirs, communications towers, etc.) as part of a future project. A key component of these evaluations is the assessment of Franzen Reservoir.

- **Hydrants and Seismic Shutoff Valves**. It is recommended that the City consider installing hydrants between the reservoirs and seismic isolation valves so that stored water can be accessed by the City staff and the City Fire Department. The majority of the City's reservoirs have seismic shutoff valves to preserve water storage. However, the reservoir sites with seismic shutoff valves seem to be lacking hydrants that are connected between the reservoir and the seismic valve. As a result, fire trucks may not currently have a way to access the water stored in the reservoirs after the seismic valves close. As part of the City's resilience implementation plan, it is recommended that a hydrant is installed between the connection between each of the reservoirs and its seismic shutoff valve.
- **Evaluate Improvement Alternatives**. It is recommended that the City evaluate improvement alternatives for the transmission main alignments and for opportunities to serve West Salem during a CSZ earthquake.

7.0 References

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Appendix A. Critical Social/Economic Needs

Appendix A
Critical Social/Economic Needs: Name and Address List

Parcel Description	Address	City GIS PLACE_TYPE
Hospitals		
SALEM HOSPITAL	1002 BELLEVUE ST SE	Health Care Clinic or Service
SALEM HOSPITAL	2455 FRANZEN ST NE	Health Care Clinic or Service
SALEM HOSPITAL	2561 CENTER ST NE	Health Care Clinic or Service
SALEM HOSPITAL	3300 & 3310 STATE ST	Health Care Clinic or Service
SALEM HOSPITAL	665 & 699 WINTER ST SE	Hospital / Health Care Complex
SALEM HOSPITAL	698 12TH ST SE	Health Care Clinic or Service
SALEM HOSPITAL	875, 939, & 1127 OAK ST SE	Hospital / Health Care Complex
SALEM HOSPITAL	1073 OAK ST SE	Health Care Clinic or Service
SALEM HOSPITAL	985 MISSION ST SE	Health Care Clinic or Service
SALEM HOSPITAL REGIONAL LABORATORY	869 MEDICAL CENTER DR NE	Health Care Clinic or Service
Urgent Care Centers		
KAISER PERMANENTE NORTH LANCASTER	2400 LANCASTER DR NE	Health Care Clinic or Service
MEND CLINIC ORTHOPEDIC URGENT CARE	2936 COMMERCIAL ST SE	Health Care Clinic or Service
SALEM CLINIC	2020 CAPITOL ST NE	Health Care Clinic or Service
SALEM CLINIC SOUTH	2531 BOONE RD SE	Health Care Clinic or Service
SALEM HEALTH MEDICAL CLINIC	1049 EDGEWATER ST NW	Health Care Clinic or Service
SOUTH SALEM IMMEDIATE CARE CLINIC	3777 COMMERCIAL ST SE	Health Care Clinic or Service
SWIFTCARE	560 Wallace Rd NW Suite 140	BV_ADDED
Dialysis Centers		
DAVITA DIALYSIS	1220 LIBERTY ST NE	Health Care Clinic or Service
DAVITA DIALYSIS	645 9TH ST NW STE 145	BV_ADDED
DAVITA DIALYSIS	421 LANCASTER DR NE	BV_ADDED
DAVITA DIALYSIS	4792 PORTLAND RD NE	BV_ADDED
DAVITA DIALYSIS	3550 LIBERTY RD S STE 100	BV_ADDED
FRESENIUS KIDNEY CARE	1060 2ND ST NW	Health Care Clinic or Service
FRESENIUS KIDNEY CARE	440 LANCASTER DR NE	BV_ADDED
City of Salem Critical Services		
CITY HALL	1320 EDGEWATER ST NW	Office Business
CITY OF SALEM INFORMATION TECHNOLOGY	295 CHURCH ST SE	Multi-Use Building
SALEM AIRPORT TERMINAL BUILDING / SALEM AIRPORT TOWER	2990 & 3000 25TH ST SE	Airport Terminal, Runway or Support Facility
SALEM CITY SHOPS COMPLEX	1388 - 1590 20TH ST SE 1395 - 1582 22ND ST SE	Salem City Facility
SALEM FIRE STATION 11	1970 ORCHARD HEIGHTS RD NW	Salem City Fire Station
SALEM FIRE STATION 8	4000 LANCASTER DR NE	Salem City Fire Station
SALEM FIRE STATION 7	5021 LIBERTY RD S	Salem City Fire Station
SALEM MAIN LIBRARY	1400 BROADWAY ST NE	Service Business
SALEM POLICE DEPT EMERGENCY SERVICES BUILDING	4730 LIBERTY RD S	Salem City Facility
SALEM POLICE FACILITY	333 DIVISION ST NE	Municipal Police Station
WILLAMETTE VALLEY COMMUNICATIONS CENTER	595 COTTAGE ST NE	Salem City Facility
State of Oregon Critical Services		
ANDERSON READINESS CENTER	3225 STATE ST	State Government Facility
ARMY AVIATION SUPPORT FACILITY	1921 TURNER RD SE	State Government Facility
CAPITOL BUILDING	900 COURT ST NE	State Government Facility
DEPARTMENT OF AVIATION	3040 25TH ST SE	State Government Facility
DEPT OF ADMINISTRATIVE SERVICES	155 COTTAGE ST NE	State Government Facility
DEPT OF ENERGY	550 CAPITOL ST NE	State Government Facility
DEPT OF FORESTRY	2600 STATE ST 2600 LEE ST SE	State Government Facility
HUMAN SERVICES BUILDING	500 SUMMER ST NE	State Government Facility
ODOT	455 & 885 AIRPORT RD SE 1158 & 1178 CHEMEKETA ST NE 4040 FAIRVIEW INDUSTRIAL DR SE	State Government Facility
ODOT AUTO MAINTENANCE	2480 TURNER RD SE	State Government Facility
ODOT MATERIALS LAB	800 AIRPORT RD SE	State Government Facility
ODOT MILL CREEK BUILDING	555 13TH ST NE	State Government Facility
ODOT SAFE HAVEN	1144 CENTER ST NE	State Government Facility
ODOT TRAFFIC SIGNAL DIVISION	2445 LIBERTY ST NE	State Government Facility
OREGON LOTTERY BUILDING	500 AIRPORT RD SE	State Government Facility
OREGON PUBLIC SAFETY ACADEMY	4190 AUMSVILLE HW SE	State Government Facility
OREGON STATE FAIRGROUNDS	2330 17TH ST NE	State Government Facility
OREGON STATE HOSPITAL	2600 CENTER ST NE	State Government Facility
OREGON STATE POLICE OFFICE	3545 & 3565 TRELSTAD AV SE	State Police Station / Facility
PUBLIC SERVICE BUILDING	255 CAPITOL ST NE	State Government Facility
PUBLIC UTILITY COMMISSION	201 HIGH ST SE	Office Business

Appendix A
Critical Social/Economic Needs: Name and Address List

Parcel Description	Address	City GIS PLACE_TYPE
State of Oregon Critical Services (Cont.)		
SANTIAM CORRECTIONAL INSTITUTION	4005 AUMSVILLE HW SE	State Government Facility
STATE DATA CENTER	530 AIRPORT RD SE	State Government Facility
STATE LIBRARY BUILDING	250 WINTER ST NE	Library / Research Facility
STATE MOTOR POOL	1100 AIRPORT RD SE	State Government Facility
SUPREME COURT BUILDING	1163 STATE ST	State Government Facility
TRANSPORTATION BUILDING	355 CAPITOL ST NE	State Government Facility
Marion County Critical Services		
MARION COUNTY COURTHOUSE	100 HIGH ST NE	County Government Facility
MARION COUNTY HEALTH	2045 SILVERTON RD NE	County Government Facility
MARION COUNTY HEALTH DEPT	3180 CENTER ST NE	County Government Facility
MARION COUNTY OFFICE BUILDING	555 COURT ST NE	Office Business
Correctional Facilities		
HILLCREST YOUTH CORRECTIONAL FACILITY	2450 STRONG RD SE	State Government Facility
MARION COUNTY CORRECTIONAL FACILITY	4000 AUMSVILLE HW SE	County Government Facility
MARION COUNTY JUVENILE DEPT DETENTION CENTER	2970 CENTER ST NE	County Government Facility
MILL CREEK CORRECTIONAL FACILITY	5400 , 5465, & 5471 TURNER RD SE	State Government Facility
OREGON STATE CORRECTIONAL INSTITUTION	3405 DEER PARK DR SE	State Government Facility
STATE PENITENTIARY	2605 STATE ST	State Government Facility
STATE PENITENTIARY MINIMUM	2809 STATE ST	State Government Facility
Emergency Shelters & Community Water Distribution Points		
AUBURN ELEMENTARY SCHOOL	4612 AUBURN RD NE	Public Elementary School
BATTLE CREEK ELEMENTARY SCHOOL	1640 WALN DR SE	Public Elementary School
BRUSH COLLEGE ELEMENTARY SCHOOL	2623 DOAKS FERRY RD NW	Public Elementary School
CHEMEKETA COMMUNITY COLLEGE	4000 LANCASTER DR NE	Community College
CORBAN UNIVERSITY	5000 DEER PARK DR SE	College / University Building
CROSSLER MIDDLE SCHOOL	1155 DAVIS RD S	Public Middle School
HOUCK MIDDLE SCHOOL	1155 CONNECTICUT ST SE	Public Middle School
JUDSON MIDDLE SCHOOL	4512 JONES RD SE	Public Middle School
LESLIE MIDDLE SCHOOL	3850 PRINGLE RD SE	Public Middle School
MCKAY HIGH SCHOOL	2440 LANCASTER DR NE	Public High School
NORTH SALEM HIGH SCHOOL	765 14TH ST NE	Public High School
PARRISH MIDDLE SCHOOL	802 CAPITOL ST NE	Public Middle School
PUTNAM UNIVERSITY CENTER	935 MILL ST SE	College / University Building
ROBERT W STRAUB MIDDLE SCHOOL	1920 WILMINGTON AV NW	Public Middle School
ROBERTS HIGH SCHOOL-STATE STREET CAMPUS	3620 STATE ST	Public Alternative High School
SALEM CONVENTION CENTER	200 COMMERCIAL ST SE	Assembly / Exhibition Hall
SOUTH SALEM HIGH SCHOOL	1910 CHURCH ST SE	Public High School
SPRAGUE HIGH SCHOOL	2373 KUEBLER RD S	Public High School
STEPHENS MIDDLE SCHOOL	4962 HAYESVILLE DR NE	Public Middle School
TOKYO INTERNATIONAL UNIVERSITY OF AMERICA	1300 MILL ST SE	College / University Building
WALDO MIDDLE SCHOOL	2805 LANSING AV NE	Public Middle School
WALKER MIDDLE SCHOOL	1075 8TH ST NW	Public Middle School
WEST SALEM HIGH SCHOOL	1655 DOAKS FERRY RD NW	Public High School
Community Water Distribution Points		
CASCADES GATEWAY PARK	2100 TURNER RD SE	Developed City, County or State Park / Area
LIMELIGHT WATER PUMP STATION	880 VAN BUREN DR NW	Public Water Pump Station
RIVER ROAD PARK	3045 RIVER RD N	Developed City, County or State Park / Area
SALEM CITY SHOPS BUILDING 16 WATER STORAGE	1440 20TH ST SE	Salem City Facility
SALEM FIRE STATION 1	370 TRADE ST SE	Salem City Fire Station
SALEM FIRE STATION 2	875 MADISON ST NE	Salem City Fire Station
SALEM FIRE STATION 3	1884 LANSING AV NE	Salem City Fire Station
SALEM FIRE STATION 4	200 ALICE AV S	Salem City Fire Station
SALEM FIRE STATION 5	1520 GLEN CREEK RD NW	Salem City Fire Station
SALEM FIRE STATION 6	2740 25TH ST SE	Salem City Fire Station
SALEM FIRE STATION 7	1970 ORCHARD HEIGHTS RD NW	Salem City Fire Station
SALEM FIRE STATION 8	4000 LANCASTER DR NE	Salem City Fire Station
SALEM FIRE STATION 9	5080 BATTLE CREEK RD SE	Salem City Fire Station
SALEM FIRE STATION 10	3611 STATE ST	Salem City Fire Station
SALEM FIRE STATION 11	5021 LIBERTY RD S	Salem City Fire Station
SALEM/KEISER INTERTIE #1 (CHERRY AVE BOOSTER)	4000 BLOCK CHERRY AVE NE	Public Water Pump Station
SOUTH RIVER ROAD WATER PUMP STATION	3285 RIVER RD S	Public Water Pump Station
TURNER CONTROL WATER FACILITY	7100 3RD ST SE	Public Water Facility
WEATHERS STREET PARK	4188 WEATHERS ST NE	Developed City, County or State Park / Area

Appendix A
Critical Social/Economic Needs: Name and Address List

Parcel Description	Address	City GIS PLACE_TYPE
Vulnerable Populations		
ADULT CARE HOME	1530 GABRIELA CT NE	Adult Care Home or Facility
AFH LICENSE #514816	3565 BELLE VISTA CT S	Adult Care Home or Facility
AVAMERE SKILLED NURSING FACILITY	3445 BOONE RD SE	Adult Care Home or Facility
BATTLE CREEK MEMORY CARE	1805 WALN DR SE	Adult Care Home or Facility
BERRY CARE	1665 BERRY ST SE	Adult Care Home or Facility
BONAVENTURE SENIOR LIVING CENTER	3411 BOONE RD SE	Retirement Center or Other
BROOKDALE SALEM ALZHEIMERS & DEMENTIA CARE(Clare Bridge)	1355 BOONE RD SE	Retirement Center or Other
BROOKSTONE ALZHEIMER SPECIAL CARE CENTER	5881 WOODSIDE DR SE	Retirement Center or Other
CAPITAL MANOR RETIREMENT	368 LOWER LAVISTA CT NW	Retirement Center or Other
CAPITAL MANOR RETIREMENT	1961 MANORVIEW LN NW	Retirement Center Residence
CAPITOL MANOR MAINTENANCE BLDG	2071 SALEM DALLAS HW NW	Retirement Center or Other
DANVILLE SERVICES OF OREGON LLC	4900 LIBERTY RD S	Adult Care Home or Facility
FARMINGTON SQUARE OFFICE	920 BOONE RD SE	Retirement Center or Other
FORDS WESTSIDE MANOR	1042 8TH ST NW	Retirement Center or Other
FOUR SEASONS RESIDENTIAL CARE FACILITY	2850-2855 EVERGREEN AV NE	Adult Care Home or Facility
GIBSON CREEK ASSISTED LIVING OFFICE	1615 BRUSH COLLEGE RD NW	Adult Care Home or Facility
HARMONY HOUSE	3062 HYACINTH ST NE	Adult Care Home or Facility
HAWTHORNE HOUSE	3042 HYACINTH ST NE	Adult Care Home or Facility
HIDDEN LAKES OFFICE	400 MADRONA AV SE	Retirement Center or Other
HOME INSTEAD SENIOR CARE	2015 25TH ST SE	Health Care Clinic or Service
INDEPENDENT LIVING CENTER AND DAYCARE FACILITY	2990 BOONE RD SE	Adult Care Home or Facility
JASON LEE MANOR	1551 CENTER ST NE	Retirement Center or Other
LANCASTER VILLAGE RETIREMENT COMMUNITY	1496 BRENNER ST NE	Retirement Center or Other
LANCASTER VILLAGE RETIREMENT COMMUNITY	4099 CYPRESS ST NE	Retirement Center or Other
LANCASTER VILLAGE RETIREMENT COMMUNITY	4138 - 4156 MARKET ST NE	Retirement Center or Other
LANCASTER VILLAGE RETIREMENT COMMUNITY	1492 BRENNER ST NE	Retirement Center or Other
MADRONA HILLS RETIREMENT APTS OFFICE	707 MADRONA AV SE	Retirement Center or Other
MEADOW CREEK VILLAGE	3988 12TH ST CUTOFF SE	Retirement Center or Other
MOSAIC SENIOR LIVING	2950 BOONE RD SE	Retirement Center or Other
ORCHARD HEIGHTS SENIOR COMMUNITY	695 ORCHARD HEIGHTS RD NW	Retirement Center or Other
OREGON SCHOOL FOR THE DEAF	999 LOCUST ST NE	Special Purpose School
REDWOOD HEIGHTS ASSISTED LIVING CO	4050 12TH ST CUTOFF SE	Retirement Center or Other
RODINA RETIREMENT CENTER	4107 FISHER RD NE	Retirement Center or Other
SALEM MASONIC TEMPLE CARE HOME	1601 BRUSH COLLEGE RD NW	Adult Care Home or Facility
SHANGRI LA CORP	1460 VISTA AV SE	Adult Care Home or Facility
SOUTHERN HILLS ASSISTED LIVING COMMUNITY	4795 SKYLINE RD S	Adult Care Home or Facility
SPRUCE VILLA INC SIZEMORE APTS	1915 SIZEMORE DR NE	Adult Care Home or Facility
SUNNY OAKS INC	2526 WILARK DR NW	Adult Care Home or Facility
SUNNY OAKS INC THE GROTTO	4375 RICKEY ST SE	Adult Care Home or Facility
SUNNYSIDE CARE HOME	4515 SUNNYSIDE RD SE	Retirement Center or Other
SWEET BYE N BYE ASSISTED LIVING	2520 CORAL AV NE	Retirement Center or Other
SWEET BYE N BYE RCF	2480 CORAL AV NE	Adult Care Home or Facility
THARSEL NURSING HOME	2210 LANSING AV NE	Adult Care Home or Facility
THE RIDGE AT MADRONA HILLS CLUBHOUSE	678 RATCLIFF DR SE	Retirement Center or Other
THE SPRINGS AT SUNNYVIEW RETIREMENT COMMUNITY	1950 45TH AV NE	Retirement Center or Other
THE WOODS AT WILLOWCREEK	4398 GLENCOE ST NE	Adult Care Home or Facility
TIERRA ROSE SENIOR LIVING COMMUNITTY	4254 WEATHERS ST NE	Retirement Center or Other
TOUCH OF LOVE SENIOR CARE	4190 SUNNYVIEW RD NE	Adult Care Home or Facility
WILLSON HOUSE	1625 CENTER ST NE	Retirement Center or Other
WINDSONG OF EOLA HILLS	2030 WALLACE RD NW	Adult Care Home or Facility
WOODLAND RESIDENCE INN OFFICE	4710 SUNNYSIDE RD SE	Retirement Center or Other

Appendix B. Seismic Geohazard Evaluation Report

SUBMITTED TO:
Black & Veatch
19801 SW 72nd Ave Suite 200
Tualatin, Oregon, 97602

BY:
Shannon & Wilson, Inc.
3990 SW Collins Way, Ste 100
Lake Oswego, Oregon,
97035

503-210-4750
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DRAFT

SEISMIC GEOHAZARD EVALUATION REPORT

City of Salem Seismic Resilience Study

SALEM, OREGON

May 2021

Shannon & Wilson No: 105679-501

SHANNON & WILSON

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Submitted To: Black & Veatch
19801 SW 72nd Ave Suite 200
Tualatin, Oregon, 97602
Attn: Ho-ping Wei, PE

Subject: DRAFT SEISMIC GEOHAZARD EVALUATION REPORT, CITY OF SALEM SEISMIC
RESILIENCE STUDY, SALEM, OREGON

Shannon & Wilson prepared this report and participated in this project as a subconsultant to Black & Veatch. Our scope of services was specified in Agreement Number 406828.12.1000 with Black & Veatch dated October 16, 2020. This report presents our Seismic Geohazard Evaluation and was prepared by the undersigned.

We appreciate the opportunity to be of service to you on this project. If you have questions concerning this report, or we may be of further service, please contact us.

Sincerely,

SHANNON & WILSON, INC.

Elliott Mecham, PE Senior Associate	Kevin Wood, PE Senior Engineer

DJS:KJW:ECM:WJP/las

EXECUTIVE SUMMARY

Based on our regional seismic hazard mapping and review of existing information provided to us or obtained from publicly available sources, we have assigned hazard rankings for various seismic hazards at each of the critical facilities provided to us. The hazard rankings for the various seismic hazards we considered are summarized in Exhibit ES-1. Numerous assets are located in areas where rock is mapped as the geologic surface unit. The risk of permanent ground deformation from liquefaction related hazards at rock sites is considered low and the primary seismic hazard is strong ground motions. Assets where rock is mapped and we have ranked a low risk of liquefaction and landslide include Grice Hill Reservoir, Hemlock Well, Mountain View Reservoir and Pump Station, Limelight Pump Station, Fairmount Reservoir, Candalaria Reservoir, the South Salem Repeater Tower, the ASR Wells, Skyline Repeater Tower, Lone Oak Reservoir, Creekside Pump Station, Deer Park Pump Station, and Mill Creek Reservoir.

We assigned a moderate hazard ranking to the EOLA 1B Reservoir for landslides due to the proximity of the reservoir to an existing landslide and the lack of available site-specific subsurface information. If subsurface information is provided to us for this site, we can reassess the landslide hazard and ranking at this site.

We assigned moderate and high hazard rankings to Croisan Lower Pump Station for liquefaction and landslides based on the predicted ground deformations, our site reconnaissance, and due to the site being near the contact between a large existing landslide and volcanic rock, and the lack of available site-specific subsurface information. If subsurface information is provided to us for this site, we can reassess the seismic hazards and rankings at this site.

Based solely on the geologic mapping and modeling, the potential permanent ground deformation was low at the Edwards S1 Pump Station. However, the assessment is based on the presence of coarse-grained flood deposits at the site as indicated on the geologic map and the nearest publicly available well logs indicate that site is mantled with fine-grained flood deposits. Because the predicted settlements in the mapping model are based on the assumption that the pump station is underlain by gravel as mapped which appears not to be correct based on the closest available well log, the hazard may not be adequately defined by the hazard mapping. Additionally, we understand that there haves been uncontrolled releases of water at this site in the past that has resulted in the manifestation of surface settlement around the building foundations. Due to uncertainties associated with the liquefication potential and subgrade, we consider the potential for permanent ground deformation from landslides and liquefaction to be moderate to high during a seismic event.

We assigned a moderate hazard rating to the Champion Hill Reservoir for potential liquefaction and landslides due to uncertainty in the subsurface conditions from a lack of available site-specific subsurface explorations. Based solely on the geologic mapping and the hazard modeling, the potential for geohazards was considered to be low due to rock being mapped at the site. However, the nearest publicly available well logs indicate that site is mantled with fine grained flood deposits, which are at a higher risk of permanent ground surface deformations during a seismic event. If subsurface information is provided to us for this site, we can reassess the seismic hazards and ranking at this site.

We assigned moderate and high hazard rankings to the Franzen Reservoir and Repeater Tower for potential landslides and fault rupture. The hazard rankings are based on our understanding from the existing basis of design reports provided to us and our understanding of past instability along the earthen embankments.

Exhibit ES-1: Summary of Geotechnical Seismic Hazard Rankings

Site ID	Locations	Site Class	Liquefaction Settlement Hazard	Landslide Hazard	Fault Rupture Hazard
1	Salem-Keizer Intertie/Cherry Ave Booster Pump Station	D	M	L	L
2	Grice Hill Reservoir and Repeater Tower	B	L	L	L
3	Hemlock Well	B	L	L	L
4	Mountain View Reservoir and Pump Station	B	L	L	L
5	EOLA 1B Reservoir	B	L	M	L
6	Limelight Pump Station	B	L	L	L
7	Fairmount Reservoir	B	L	L	L
8	Candalaria Reservoir	B	L	L	L
9	South Salem Repeater Tower	B	L	L	L
10	Croisan Lower Pump Station	C/D	M	H	L
11	Edwards S1 Pump Station*	D	H	M	L*
12	ASR Wells	B	L	L	L
13	Skyline Repeater Tower	B	L	L	L
14	Lone Oak Reservoir	B	L	L	L
15	Creekside Pump Station	B	L	L	L
16	Champion Hill Reservoir	B	M	M	L
17	Boone Road Pump Station	D	L	L	L
18	Deer Park Pump Station	B	L	L	L
19	Mill Creek Reservoir	B	L	L	L
20	Turner Control Facility	D	L	L	L
21	Franzen Reservoir and Repeater Tower*	B	L	H	M
22	Geren Island Water Treatment Plant	D	L	L	L

NOTE: L = Low, M = Moderate, H = High

*See discussion in main text. Geologic maps may not adequately capture geohazard.

Note that the sites highlighted in red did not have subsurface explorations available for review, and nearby well logs could not be found. Therefore, the sites highlighted in red in Exhibit ES-1 are based on the regional seismic hazard mapping only.

For the pipelines, the main hazards based on the mapping appears to be localized liquefaction, lateral spreading at the Sunset Park Willamette River crossing, and fault rupture where the pipelines cross the Turner and Mill Creek Faults and Waldo Hills Fault. Based on the mapping, the potential for localized liquefaction is highest at the Willamette River Crossings, near Turner, Oregon, and near the Geren Island Water Treatment Plant (WTP). However, existing subsurface information and Geotechnical Engineering Reports performed at Geren Island WTP show that the mapping-based liquefaction hazard may overestimate the actual hazard. This is due to the relatively high percentage of gravels underlying that site.

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Appendix I: Existing Information Site 18 – Deer Park Pump Station
Appendix J: Existing Information Site 19 – Mill Creek Reservoir
Appendix K: Existing Information Site 20 – Turner Control Facility
Appendix L: Existing Information Site 21 – Franzen Reservoir and Repeater Tower
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Appendix N: Existing Information Center Street Bridge Crossing
Important Information

1 SCOPE OF SERVICES

The purpose of the project is to provide a seismic resiliency analysis of the City of Salem (City) water treatment, transmission, and distribution system and to develop recommendations for mitigation and future infrastructure design. Shannon & Wilson's scope of work consisted of the following:

- Gather existing geologic/geotechnical and seismic data in the greater Salem area to develop a preliminary understanding of subsurface conditions and potential seismic hazards, including local and regional readily available geologic publications and maps, DOGAMI seismic hazard maps, Oregon Department of Water Resources well logs at select locations and geotechnical boring information and reports, as available.
- Evaluate existing geologic/geotechnical and seismic data in the greater Salem area to develop a thorough understanding of subsurface conditions and potential seismic hazards.
- Prepare seismic hazard maps including Seismic Hazard Maps based on the Magnitude 9.0 Cascadia Subduction Zone (CSZ) scenario defined in the Oregon Resilience Plan and local geology. The maps include peak ground velocity, peak ground acceleration, 0.3- and 1.0-second spectral accelerations, probability of liquefaction, liquefaction induced settlement, and landslide induced permanent ground deformation based on the methodology developed by HAZUS.
- Perform screening level liquefaction analyses on available geotechnical borings provided by the City using methods developed by Boulanger and Idriss (2014).
- Perform site visits to facilities identified by the City as critical.
- Evaluate the seismic geohazard rankings and assigned hazard rankings to the backbone assets identified by Black & Veatch and the City.

2 SEISMIC HAZARD MAPPING

2.1 Approach

The GIS map layers developed for this project are primarily based on published geologic maps; variations from actual site conditions should be expected. Also, the analyses, methods, and approaches applied herein were developed and used by the Oregon Department of Geology and Mineral Industries (DOGAMI) and the Federal Emergency Management Agency (FEMA) for planning purposes only. FEMA methodology referenced by DOGAMI refers to the Hazus® -MH 2.1 Technical Manual (FEMA, 2011). This manual has since been updated, (Hazus® -MH 4.2 Technical Manual (FEMA, 2020), and these manuals were compared so that current, updated methodologies would be used where applicable. While the 2020 Hazus® manual expanded on analyses in the 2011 manual, for all of the analyses done for this regional mapping, the two manuals do not differ in their methodologies. Also, note that these types of analyses are not the same as those used for site-specific, code-based geotechnical design.

2.2 Existing Information Review

2.2.1 Regional Seismological Setting

Earthquakes in the Pacific Northwest occur largely as a result of the subduction of the Juan de Fuca plate beneath the North American plate along the Cascadia Subduction Zone (CSZ). The CSZ is located approximately parallel to the coastline from northern California to southern British Columbia. The compressional forces that exist between these two colliding plates cause the oceanic Juan de Fuca plate to descend, or subduct, beneath the continental plate at a rate of about 1.5-inches per year (DeMets and others, 1990). This process leads to volcanism in the North American plate and stresses and faulting in both plates throughout much of the western regions of southern British Columbia, Washington, Oregon, and northern California. Stress between the colliding plates is periodically relieved through great earthquakes at the CSZ plate interface.

Within the regional tectonic framework and historical seismicity, three broad earthquake sources are identified:

- Subduction Zone Interface Earthquakes originate along the CSZ, which is located 25 miles beneath the coastline. Paleoseismic evidence and historic tsunami records from Japan indicate that the most recent subduction zone interface event was in 1700 AD and was an approximately magnitude 9 earthquake that likely ruptured the full length of the CSZ.
- Deep-Focus, Intraplate Earthquakes originate from within the subducting Juan de Fuca oceanic plate as a result of the downward bending and tension in the subducted plate. These earthquakes typically occur 28 to 38 miles beneath the surface. Such events on the CSZ are estimated to be as large as magnitude 7.5. Historic earthquakes include the 1949 magnitude 7.1 Olympia earthquake, the 1965 magnitude 6.5 earthquake between Tacoma and Seattle, and the magnitude 6.8 2001 Nisqually earthquake. The highest rate of CSZ intraslab activity is beneath the Puget Sound area, with much lower rates observed beneath western Oregon.
- Shallow-Focus Crustal Earthquakes are typically located within the upper 12 miles of the earth’s surface. The relative plate movements along the CSZ cause not only east-west compressive strain but dextral shear, clockwise rotation, and north-south

compression of the leading edge of the North American Plate (Wells and others, 1998), which is the cause of much of the shallow crustal seismicity of engineering significance in the region. The largest known crustal earthquake in the Pacific Northwest is the 1872 North Cascades earthquake with an estimated magnitude of about 7. Other examples include the 1993 magnitude 5.6 Scotts Mill earthquake and magnitudes 5.9 and 6.0 Klamath Falls earthquakes.

2.2.2 Oregon Resilience Plan

The Oregon Resilience Plan is a result of Oregon House Resolution 3, adopted in April 2011. The House Resolution directed the Oregon Seismic Safety Policy Advisory Commission "to lead and coordinate preparation of an Oregon Resilience Plan that reviews policy options, summarizes relevant reports and studies by state agencies, and makes recommendations on policy direction to protect lives and keep commerce flowing during and after a Cascadia earthquake and tsunami" (OSSPAC, 2013). A task group then developed a Cascadia Earthquake Scenario for use by other work groups as a basis for assessing the effects of the scenario on various sectors of society or parts of the built environment.

This assessment is for a magnitude 9.0 CSZ earthquake, as defined in the Oregon Resilience Plan. Other magnitudes of CSZ events and earthquakes from other sources are not considered. However, at the request of Black & Veatch, we have provided design ground displacements from fault rupture at the pipeline crossings of shallow Class A faults.

2.2.3 Geology

The project site lies within the Willamette Valley physiographic province (Orr and others, 1992). The local geology has been mapped by numerous authors including Tolan and others (2000) and O'Connor and others (2001). A simplified geologic map of the study area is presented in Figure 2 and is based on DOGAMI publications OGDC-6 (Smith and Row, 2015) and SLIDO 4.0 (Franczyk and others, 2019).

Today the Willamette Valley is a broad alluvial plain bounded by the Columbia River to the north, the Cascade Range to the east, and the Coast Range to the west and south. Before it was a terrestrial valley, the region was a broad continental shelf, extending westward from the proto-Cascades into the ocean (Orr and others, 1992). Around 50 million years ago, an oceanic island chain slowly collided with the coastline as the oceanic crust that carried it was subducting beneath the North American tectonic plate. This accreted island chain ultimately formed the Coast Range and shaped the present-day Willamette Valley by creating the western and southern boundary.

Structurally, the valley is a tectonic fore-arc basin created by down warping and faulting of the underlying Columbia River Basalt Group bedrock as the Coast Range and Cascades were being uplifted (Gannett and others, 1998). From the creation of the sedimentary basin to the beginning of the ice age, the valley was inundated by deposition from the surrounding uplands including Pleistocene (2.6 million to 11,700 years ago) sand and gravels, and mud and debris flows from volcanic eruptions in the Cascades (O'Conner and others, 2001). These Pleistocene sand and gravels formed large widespread sheets and alluvial fan complexes which extended into the Valley floor where major Willamette tributaries exited from the Cascade Range. In the central and southern Willamette Valley, these Pleistocene sand and gravels directly correspond to previously mapped Pleistocene alluvial deposits referred to as Linn Gravel and the Rowland Formation (O'Conner and others, 2001). Estimated thickness of the Pleistocene sand and gravel deposits from drill logs indicate near surface deposits of 40 to greater than 100 meters thick at alluvial fan apexes, and 10 to 20 meters thick in the distant areas away from the Cascades or Coast ranges (O'Conner and others, 2001).

During the late stages of the last great ice age, between about 18,000 and 15,000 years ago, a lobe of the continental ice sheet repeatedly blocked and dammed the Clark Fork River in western Montana, which then formed an immense glacial lake called Lake Missoula. The lake grew until its depth was sufficient to buoyantly lift and rupture the ice dam, which allowed the entire massive lake to empty catastrophically. Once the lake had emptied, the ice sheet again gradually dammed the Clark Fork Valley and the lake refilled, leading to 40 or more repetitive outburst floods at intervals of decades (Allen and others, 2009). During each short-lived episode, floodwaters washed across the Idaho panhandle, through the eastern Washington scablands, and through the Columbia River Gorge. When the floodwater emerged from the western end of the gorge, it spread out over the Portland Basin and up the Willamette Valley as far south as Junction City, depositing a tremendous load of sediment (O'Conner and others, 2001). In the Salem area, these deposits are mostly composed of silt and clay, and mapped as fine-grained Missoula Flood deposits by O'Connor and others (2001). These fine-grained flood deposits blanketed the earlier Pleistocene sand and gravel alluvium obscuring the underlying gravels beneath a layer of silt and clay. In more recent times, portions of the site have been cut, graded, or filled during the course of development.

2.2.4 Available Mapping

DOGAMI developed a publication based on the Oregon Resilience Plan CSZ scenario for the state of Oregon. The publication, Open-File Report O-13-06, primarily consists of GIS data of site conditions, ground motions, ground deformations, and other hazards associated with a magnitude 9.0 event on the CSZ (Madin and Burns, 2013). Datasets of interest for this project include the following:

- Shear Wave Velocity within 30 meters of the Ground Surface (Vs30)
- Bedrock and Site Peak Ground Acceleration (PGA)
- Bedrock and Site 1-second Spectral Acceleration (SA1)
- Bedrock and Site Peak Ground Velocity (PGV)
- Liquefaction Susceptibility, Probability, and Permanent Ground Deformation (PGD)
- Earthquake-Induced Landslide Susceptibility, Probability, and PGD

The provided methodology indicates that, within the project area, the majority of these datasets were derived based on the Relative Earthquake Hazard Maps of the Salem East and Salem West Quadrangles, Marion and Polk Counties, Oregon (GMS-105; Wang and Leonard, 1996); the Oregon Geologic Data Compilation Release 5 (OGDC-5); and the Statewide Landslide Information Database for Oregon Release 2 (SLIDO-2; Burns and others, 2011). The bedrock ground motions included in the publication were provided to DOGAMI by the U. S. Geological Survey (USGS) and are based on the USGS Cascadia M 9.0 scenario ShakeMap®.

Following the publication of O-13-06, DOGAMI published the Oregon Geologic Data Compilation Release 6 (OGDC-6; Smith and Roe, 2015) and Release 4.0 of the Statewide Landslide Information Database for Oregon (SLIDO-4.0, Franczyk and others, 2019). These recent publications have not yet been incorporated into DOGAMI's CSZ scenario datasets.

Bedrock 0.3-second spectral acceleration data were downloaded from the USGS website for the Cascadia M 9.0 scenario ShakeMap® (USGS, 2017). Data for the 0.2-second spectral acceleration, as used in building codes, were not available. For preliminary planning purposes, the 0.2-second spectral acceleration can be approximated as the 0.3-second spectral acceleration.

2.3 Shear Wave Velocity, Vs30

For the study area around Salem, there are published DOGAMI maps which show both Vs (approximate weighted average shear wave velocity of the geologic unit) and Vs30 values (time-averaged shear wave velocity in the upper 30 meters of the geologic profile). However, the published Vs30 values for the study area do not incorporate shear wave velocity measurements from the Salem area. Instead, they represent averages from measurements from similar geologic units taken from across the state, primarily the Portland Metropolitan area. Therefore, we used Vs values from the DOGAMI GMS-105 publication. While Vs and Vs30 values can differ, because the data from GMS-105 represents actual values from the study area, for this project, we are assuming that Vs and Vs30 values are approximately the same. The values used for the geologic units within the study area are shown below and on Figure 3.

- Volcanic Rock: 968 m/s
- Sedimentary Rock: 920 m/s
- Landslide deposits: 360 m/s
- Terrace Deposits: 250 m/s
- Recent Alluvium: 250 m/s
- Missoula Flood Deposits: 190 m/s

2.4 Liquefaction Hazard

The liquefaction susceptibility map provided in O-13-06 is a compilation of liquefaction susceptibility maps from other DOGAMI publications. Within the Salem area, this primarily includes GMS-105 (Wang and Leonard, 1996). Explanatory text for GMS-105 indicates that susceptibility categories (0-5) were based on the available thickness of liquefiable material. Conservative groundwater levels were also used so as to not underestimate the liquefaction susceptibility.

Even though the map provided in O-13-06 indicates that the GMS-105 map was used, comparison of the original map and the one provided indicated this was not the case. Therefore, the O-13-06 map was not used. Instead, the raw data from GMS-105 was used for the area within Salem, and outside of it we used our geologic map (Figure 2), updated to include all mapped landslides, and employed the Youd and Perkins (1978) methodology, as well as knowledge of regional liquefaction susceptibility, to assign new liquefaction susceptibilities and create a unified map. To do this, we considered how Youd and Perkins would have classified a unit, and then qualitatively fit that with the Wang and Leonard (1996) susceptibility categories. During this process, the Wang and Leonard susceptibility categories 4 and 5 were merged. In areas where a susceptibility category of 5 was given, there were no apparent site-specific studies as recommended by the methodology. Furthermore, GMS-105 does not include an underlying geologic map in GIS form. Instead, it shows a generalized geologic map, which was amended based on limited site visits, aerial photograph interpretation, limited field reconnaissance, and available subsurface data. Therefore, the categories were combined to create a unified map. The resulting map is shown on Figure 4

2.5 Landslide Susceptibility

We generally followed the methodology and Geologic Group assignments as described in O-13-06, using the compiled geologic map shown on Figure 2 and discussed above, as the base map. We assigned Geologic Group C (relatively weak material) to areas mapped as Alluvium, Missoula Flood Deposits, Terrace Deposits, and Landslide Deposits. All other geologic units, including Volcanic Rock and Sedimentary Rock, were assigned Geologic Group B. We calculated a slope map from bare earth lidar data of the area to complete the landslide susceptibility map because DOGAMI's slope map was not included in O-13-06. In order to give what we believe are upper and lower limits of landslide susceptibility, maps accounting for both dry and wet conditions were generated. Dry conditions assume that the groundwater is below the level of sliding, while wet conditions assume that the groundwater level is at ground surface. The landslide susceptibility maps are shown on Figures 5 and 6.

2.6 PGA, SA1, SA0.3, and PGV

The site amplification factors in O-13-06 were calculated based on site class and the appropriate Vs30 value for each site, as determined from the Vs30 map. We calculated the PGA and SA1 site amplification factors for the Salem area from the Vs30 dataset described above using the approach referenced in O-13-06 (Boore and Atkinson, 2008) and applied them to the bedrock PGA and SA1 maps provided with O-13-06 to produce PGA, SA1, and PGV maps.

Maps of Peak Ground Acceleration, 1-Second Spectral Acceleration, and Peak Ground Velocity are shown on Figures 7, 9, and 10, respectively. The same methodology was used for the 0.3-Second Spectral Acceleration map, shown in Figure 8, using the bedrock SA0.3 map from the USGS scenario. It should be noted that current USGS & DOGAMI mapping does not include mapping for the 0.2-second spectral acceleration, but it does include spectral acceleration for a period of 0.3 seconds. For preliminary planning purposes the 0.2-second spectral acceleration can be approximated as the 0.3-second spectral acceleration.

2.7 Probability of Liquefaction

We used the refined liquefaction hazard map described above and followed the methods presented in O-13-06 to develop a map of liquefaction probability. The resulting map is shown on Figure 11.

2.8 Liquefaction-Induced PGD

2.8.1 Lateral Spreading

We used the refined liquefaction hazard map described above and followed the methods presented in O-13-06 to calculate permanent ground deformations from liquefaction-induced lateral spreading. The map of estimated PGD due to lateral spreading is included on Figure 12.

2.8.2 Settlement

DOGAMI did not include a map of predicted ground settlement associated with liquefaction in O-13-06. We calculated estimated liquefaction-induced settlements using the methodology in Chapter 4 of the Hazus® -MH 4.2 Technical Manual (FEMA, 2020), using the refined liquefaction hazard map discussed above.

The FEMA method associates each susceptibility category with a unique settlement amplitude value. Each of the values is assumed to have an uncertainty with a uniform probability distribution from one-half to two times the respective value. The map of estimated PGD due to liquefaction-induced settlement is included on Figure 13.

2.9 Probability of Earthquake-Induced Landslides

We used the refined landslide susceptibility and PGA maps described above and followed the methods presented in O-13-06 to calculate and map the probability of earthquake-induced landslides. To give what we believe are upper and lower limits of the probability of earthquake-induced landslides, we calculated probabilities in both wet and dry conditions. This was done by populating tables 4.16 and 4.17 in Chapter 4 of the Hazus® -MH 4.2 Technical Manual (FEMA, 2020). The resulting maps are shown on Figures 14 and 15.

2.10 Earthquake-Induced Landslide PGD

The earthquake-induced landslide PGD map is based on the methodology in Hazus® -MH 2 Technical Manual (FEMA, 2011), which is referenced in O-13-06. It should be noted that the Hazus methodology remains the same in the 4.2 Technical Manual (2020). We retained the acceleration term that DOGAMI chose to remove from FEMA equation 4-14 because the acceleration is in “decimal fraction of g’s,” not cm/sec^2, as DOGAMI indicated.

Additionally, we observed that the equation given by DOGAMI for the displacement factor did not produce a curve similar to the FEMA Figure 4.13 relationship. In examining the

DOGAMI equation, we saw that if the first constant was made negative, a curve similar to the FEMA Figure 4.13 relationship was seen. Therefore, we based our calculations on this slightly amended and corrected relationship to match the source FEMA publication. As we did for all landslide maps, we generated permanent ground deformation maps for both wet and dry conditions. These maps were based on probability inputs generated when calculating the probability of earthquake-induced landslides. Our maps of estimated earthquake-induced landslide permanent ground deformation are shown on Figures 16 and 17.

2.11 Surface Faulting

The United States Geologic Survey defines four categories of faults, Class A through D, based on evidence of tectonic movement known or presumed to be associated with large earthquakes during Quaternary time (within the last 2.6 million years). For Class A faults, geologic evidence demonstrates that a tectonic fault exists and that it has likely been active within the Quaternary period. The Lower Transmission Line crosses two Class A Faults identified in the United States Geologic Study Fault and Fold Data Base at the locations shown on Figure 2, Geologic Map. The Class A faults consist of the Turner Creek and Mill Creek Faults (the southern fault) and the Waldo Hills Fault (the northern fault).

Exhibit 2-1: USGS Fault Information for Mapped Faults Crossed by Transmission Mains

Fault Name	USGS Fault Number	Fault Class	Approximate Length	Sense of Slip	Slip Rate Category[1]	Time Since Last Deformation[2]
Turner and Mill Creek Faults	871	A	11.2 miles	Strike Slip	< 0.2 mm/yr	< 1.6 Ma
Waldo Hills Fault	872	A	7.5 miles	Normal	< 0.2 mm/yr	< 1.6 Ma

NOTES:

mm = millimeters; yr = year.

Ma = “Mega-annum” or million years ago.

The American Lifelines Alliance (ALA) water pipeline seismic design guidelines specify that large diameter transmission pipelines should be designed to cross active faults with evidence of fault movement within the Holocene geologic time period (i.e. less than approximately 11,000 years). While there is currently no evidence of Holocene tectonic activity along either the Turner and Mill Creek Faults or the Waldo Hills Fault, the ALA guidelines suggest considering a hypothetical displacement of approximately 10 percent of the maximum estimated fault movement due to a surface rupture.

Using the regression equations published in Wells and Coppersmith (1994), maximum hypothetical earthquake magnitudes of 6.3 and 6.6 were determined for the Waldo Hills

Fault and Turner and Mill Creek Faults, respectively. Applying the Wells and Coppersmith (1994) magnitude-fault displacement relationship calculates a maximum displacement of approximately 19 and 21 inches for the Waldo Hills Fault and Turner and Mill Creek Faults, respectively. Thus, 10 percent of the maximum estimated fault movement along both the Waldo Hills Fault and Turner and Mill Creek Faults is approximately 2 inches.

2.12 Seismic Hazards at Critical Infrastructure

The locations of selected infrastructure have been provided by Black & Veatch. The approximate locations of the selected infrastructure are shown on Figures 1 through 17, and a summary of the GIS map results for seismic hazards at these specific locations are shown on the attached Table 1.

3 SITE RECONNAISSANCE AND DOCUMENT REVIEW

Site reconnaissance was completed in two stages, on March 30, 2021 and April 16, 2021. A Shannon & Wilson geology staff member and a senior geotechnical engineer completed the reconnaissance. Descriptions of findings are provided in the following subsections, and information related to the on-site structures is primarily from the Black & Veatch 2001 seismic resiliency study. For information regarding the seismic geohazards at each of the critical facilities, see Table 1. We present the results of our site reconnaissances in the following sections in the same order that they are listed on the figures and in Table 1.

3.1 Site 1 - Salem-Keizer Intertie/Cherry Ave Booster Pump Station

The Salem-Keizer intertie is located at the Cherry Avenue Booster in Keizer, Oregon. The pump station houses a single pump, with a capacity of approximately 5 million gallons per day. During our site reconnaissance, it was observed that the pump station is on flat ground, with no observed geologic hazards.

Existing subsurface information was not available for this pump station. Based on the geologic mapping, the site is underlain by alluvial deposits. Therefore, the regional seismic hazard mapping indicates there are seismic related hazards as an issue for this site. As the site is flat, the regional seismic hazard mapping is showing liquefaction as the main hazard for this site.

We reviewed a publicly available water well log completed for the Keizer Water District within 350 feet of the site. The water well log indicates sandy clay to 22 feet, which is underlain by sands and gravels. Cemented gravel is noted at a depth of 75 feet. The static groundwater table based on this log is at 25 feet below the ground surface. A log of the exploration we reviewed is included in Appendix A.

If subsurface conditions underlying the pump station are similar to what was encountered in the nearby exploration, then the potential for liquefaction and associated hazards is considered to be moderate, which is consistent with the regional seismic mapping.

Exhibit 3-1: Photo of Salem-Keizer Intertie/Cherry Avenue Booster PS During Site Visit

3.2 Site 2 - Grice Hill Reservoir and Transmission Tower

Grice Hill Reservoir is a 20-foot-high reinforced concrete reservoir with a nearby Transmission Tower. The reservoir is located at the western extent of the Salem urban growth boundary and has a capacity of 2.3 million gallons. During our site visit, we observed that the reservoir is on relatively flat ground, and evidence or indicators of potential geologic hazards were not observed.

Subsurface information from the City and Black and Veatch was not available for this reservoir. Based on the geologic mapping, the site is underlain by volcanic rock. Therefore, the regional seismic hazard mapping does not indicate seismic geohazards at this site, except for strong ground motions.

We reviewed publicly available water well logs from two nearby residences that are approximately 700 to 800 feet south of the reservoir on 27th Place NW. The water well logs indicate that approximately 55 to 60 feet of clay overlies the basalt rock. However, both water well logs indicate that groundwater is near the contact between the clay and rock. Logs of the explorations that we reviewed are included in Appendix B.

If subsurface conditions underlying the reservoir are similar to what was encountered in the nearby explorations, the potential for permanent ground deformation from liquefaction and seismic slope instability is low, which is consistent with the regional seismic geohazard mapping.

Exhibit 3-2: Photo of Grice Hill Reservoir During Site Visit

3.3 Site 3 - Hemlock Well

Hemlock well is located near 1398 Hemlock Street NW. During our site reconnaissance, we observed that the site is on relatively flat ground with no observable geologic hazards. Based on the geologic mapping, the site is underlain by volcanic rock. Therefore, the regional seismic hazard mapping does not indicate permanent ground deformations from liquefaction induced settlement or seismic slope instability.

Exhibit 3-3: Photo of Hemlock Well During Site Visit

3.4 Site 4 - Mountain View Reservoir and Pump Station

Mountain View Reservoir is a buried, circular, prestressed concrete wire-wrapped reservoir, that was constructed in 1971. The reservoir tank has a capacity of approximately 10 million gallons. Just northeast of the reservoir is Mountain View Pump Station, constructed in 1995. This pump station is built on a 6-inch-thick reinforced concrete slab.

Only approximately 6 inches of the tank are exposed. Therefore, it could not be fully observed during our site visit. During our site visit, no evidence of slope instability or other geologic hazards were observed at either the reservoir or pump station.

The borings used during design were not available for this reservoir. Based on the geologic mapping, the site is underlain by volcanic rock. We reviewed publicly available geotechnical explorations completed for 1500 Orchard Heights Rd NW, which is approximately 700 to 800 feet northwest from the reservoir. The explorations were completed in 2011 and indicate that approximately 15 feet of clay overlies weathered basalt. A water well from 1999 for 1657 Orchard Heights Rd NW, which is approximately 500 feet north of the reservoir indicates that groundwater may be relatively deep (i.e. greater than 100 feet). Logs of the explorations that we reviewed are included in Appendix C.

If the subsurface conditions underlying the reservoir are similar to what was encountered in the nearby explorations, then the potential for liquefaction and associated hazards is considered to be low, which is consistent with the regional seismic mapping and the available. Therefore, the primary geologic hazard identified at this site for the Cascadia Subduction Zone event is strong ground motions.

Exhibit 3-4: Photo of Mountain View Reservoir and Pump Station During Site Visit

3.5 Site 5 - EOLA 1B Reservoir

EOLA 1B reservoir is a partially-buried reinforced concrete tank that was constructed in 2001. The tank has a capacity of 0.77 million gallons, with approximately 1.5 to 3 feet exposed above ground.

During the site visit, we observed that the reservoir is approximately 450 feet north of the mapped headscarp of a landslide above Doaks Ferry Road. However, at the reservoir, the ground is only gently-sloping to the south, and no on-site slope instability, such as road cracking, was observed.

Subsurface information was not available for the EOLA reservoir. Based on the geologic mapping, the site is underlain by volcanic rock. Therefore, the regional seismic hazard mapping does not indicate permanent ground deformation from liquefaction induced settlement or seismic slope instability. However, because of the proximity of the reservoir to an existing landslide, we recommend site specific geotechnical data be considered to further assess the geohazards. If the City has existing as-built information or geotechnical borings at this reservoir we request that they be provided to the project team to better assess the seismic geohazards.

Exhibit 3-5: Photo of EOLA 1B Reservoir During Site Visit

3.6 Site 6 - Limelight Pump Station

Limelight Pump Station is a reinforced masonry structure with a flexible roof diaphragm located just north of Glen Creek Reservoir. Built in 1998, the structure rests on a 6-inch reinforced concrete slab. Housing three pumps, the pump station has a total capacity of 5.18 million gallons per day. During our site reconnaissance, no evidence of slope instability or other geologic hazards were observed.

Existing subsurface information was not available for this pump station. Based on the geologic mapping, the site is underlain by volcanic rock. Therefore, the regional seismic hazard mapping does not indicate permanent ground deformations from liquefaction induced settlement or seismic slope instability.

Exhibit 3-6: Photo of Limelight Pump Station During Site Visit

3.7 Site 7 - Fairmont Reservoir

Fairmont Reservoir is a partially-buried rectangular reinforced concrete reservoir, constructed in 1937, making it Salem's oldest reservoir. The reservoir tank, which has a capacity of approximately 10 million gallons, is divided into two cells. The total height of the reservoir is 22 feet.

With approximately 2 feet of the reservoir exposed above the ground surface. it could not be fully observed during our site visit. In the immediate vicinity around the reservoir, the ground is flat, and there are no signs of slope instability.

Subsurface information was not available for this reservoir. Based on the geologic mapping, the site is underlain by volcanic rock. Therefore, the regional seismic hazard mapping does not indicate permanent ground deformation from liquefaction induced settlement or seismic slope instability. The primary geologic hazard identified at this site for the Cascadia Subduction Zone event is strong ground motions.

Exhibit 3-7: Photo of Fairmont Reservoir During Site Visit

3.8 Site 8 - Candalaria Reservoir

Candalaria Reservoir is a buried rectangular reinforced concrete reservoir that was constructed in 1940. The tank is 15 feet tall and has a capacity of 0.56 million gallons. The reservoir is currently beneath a small park where the ground is typically flat but slopes to the north, just beyond of the reservoir. The Candalaria Reservoir was included in a 2004 study performed by GRI for proposed seismic improvements. We understand from this study, that the proposed seismic improvements would likely consist of adding "seismic" valves to the reservoir. However, we do not know if the proposed improvements to the reservoir were completed.

As part of this 2004 study, a boring was completed to assess the subsurface conditions. The boring was designated B-2, and the location is shown on the site plan included in Appendix D. This boring encountered an approximately 28-foot-thick layer of hard silt with weathered basalt fragments overlying basalt. Average SPT blow counts in the silt ranged from 40 to refusal. One sample taken within the silt directly overlying the basalt had a blow count of 10. Groundwater was not indicated on the boring log, and a nearby water well installed in 1999 indicated that static groundwater was at a depth of 30 feet below the ground surface.

The tank is buried and could not be observed during the site visit. However, we observed the slopes in the immediate vicinity around the reservoir and our reconnaissance did not reveal signs of on-site slope instability nor did we observe evidence of soil creep. Based on our site visit, existing subsurface information provided to us and assumed groundwater conditions from publicly available resources, we consider the potential for seismic related permanent ground deformation due to liquefaction or seismic slope instability at this site to be low, which is consistent with the regional seismic hazard mapping. The primary geologic hazard identified at this site for the Cascadia Subduction Zone event is strong ground motions (i.e., ground shaking).

Exhibit 3-8: Photo of Candalaria Reservoir During Site Visit

3.9 Site 9 - South Salem Repeater Tower

The South Salem Repeater Tower is located at 955 Downs Street S. Based on the geologic mapping, the site is underlain by volcanic rock. Therefore, the regional seismic hazard mapping does not indicate permanent ground deformations from liquefaction induced settlement or seismic slope instability. We reviewed a publicly available geotechnical exploration completed at the site for the City of Salem. The log of the exploration indicates that the site is underlain by weathered basalt to a depth of 45 feet, which is consistent with the regional seismic hazard mapping.

3.10 Site 10 - Croisan Lower Pump Station

Croisan Lower Pump Station is a wood frame structure that sits on a 6-inch reinforced concrete slab. According to geologic mapping, this pump station sits on the headscarp of a landslide. Information from the Statewide Landslide Information Database for Oregon (SLIDO Release 4) indicates it is a deep-seated landslide with a length exceeding 1,000 feet and an estimated area of approximately 382,000 square feet. Based on our site reconnaissance, we estimate pavement cracks are typically 1/4 to 1/2 inch wide and oriented parallel and perpendicular to the roadway. The cracks where observed throughout the roadway leading to the pump station, as well as in Croisan Mountain Drive above the pump station.

Subsurface information was not available for this pump station. Based on the geologic mapping, the site is near the contact between volcanic rock and the mapped landslide. The values included in Table 1 are based on the pump station being located in landslide deposits.

If the City has existing as-built information or geotechnical borings at this pump station, we request that they be provided to the project team to better assess the seismic geohazards.

Exhibit 3-9: Photo of Croisan Lower Pump Station During Site Visit

Exhibit 3-10: Photo of Cracking Observed in Driveway During Site Visit

3.11 Site 11 - Edwards Pump Station

Edwards Pump Station was built in 1961 of non-reinforced SCR bricks. There are three pumps inside the pump station . The pump station is located within approximately 25 feet of a small creek with creek bank heights estimated to be less than 5 feet but is otherwise located on relatively level ground.

Based on our conversations with the City of Salem, we understand that past uncontrolled, pressurized water releases lifted the pavement outside the pump station above the base rock and subgrade. We observed locations where the pavement adjacent to the west side of the pump station building settled and cracks radiating around the pavement settlement formed. We also observed some cracking in the southwest corner of the pump station building. Exact measurements of the settlement areas and cracks were not performed; however, we estimate the pavement settlement adjacent to the building to be less than 6 inches and the width of the cracks to be less than 1/2 inch in width. A photo of the pump station and a close-up photo of the cracking and settlement observed in the pavement on the west side of the pump station are included below.

Subsurface information from the City was not available for this pump station at the time of our report. Based on the geologic mapping, the site is underlain by Missoula flood deposits. The Missoula flood deposits in this area are mapped as coarse-grained deposits with a very low liquefaction hazard.

We reviewed a publicly available geotechnical exploration that was completed in 1995 for the City of Salem near the intersection of Madrona Street and Madrona Court, which is approximately 250 feet southeast of the site. The exploration indicates that a 13-foot-thick layer of clay overlies gravel to a depth of 25 feet. Groundwater conditions were not indicated on the log; however, we expect them to closely follow those in the nearby creek. A log of the exploration we reviewed is included in Appendix F.

Because the predicted settlements in the HAZUS model are based on the assumption that the pump station is underlain by gravel which appears not to be correct based on the closest available well log, the hazard may not be adequately defined by the hazard mapping. The actual potential for liquefaction and movement to the nearby creek would be a function of the plasticity of the fine-grained material above the gravel. If the fine-grained material has consistent medium or high plasticity the liquefaction potential may be low. However, if layers of saturated, low plasticity silt or loose sand are also present, a liquefaction hazard may be present. Additionally, portions of the subgrade in the pavement next to the building appear to be negatively affected by past uncontrolled water releases at the pump station. If the areas of disturbed soil or voids extend below the building foundations, then portions of the foundations may have significantly less subgrade support than at locations where undisturbed native soil is present during both seismic and static conditions. Due to uncertainties associated with the liquefication potential and subgrade we consider the potential for permanent ground deformation to be moderate to high during a seismic event.

Exhibit 3-11: Photo of Edwards Pump Station During Site Visit

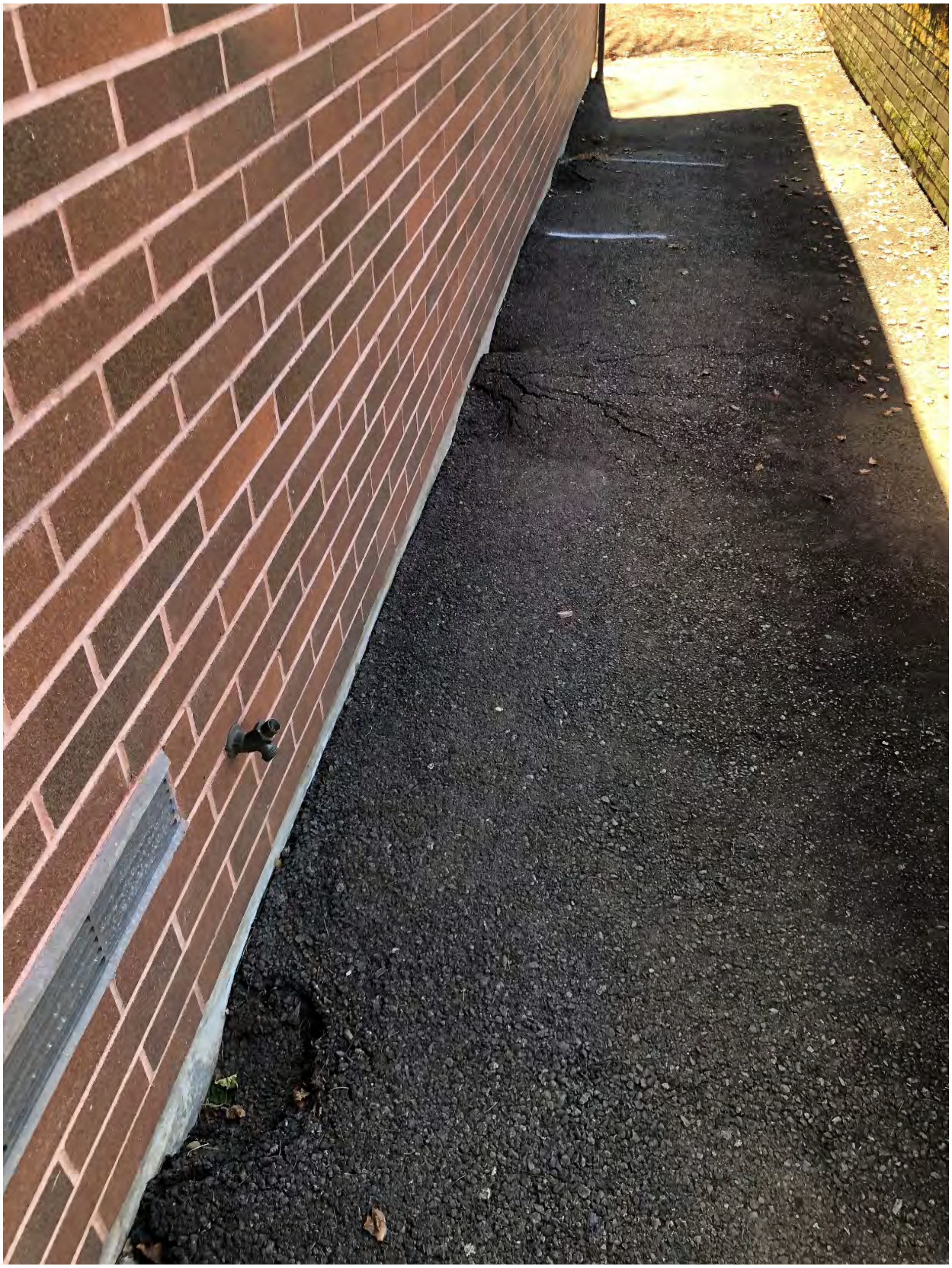

Exhibit 3-12: Photo of Cracking and Settlement along Edwards Pump Station West Wall During Site Visit

3.12 Site 12 - ASR Wells

The ASR Wells for the City of Salem are located in Woodmansee Park. At the ASR wells, of which there are five, treated water from the North Santiam River is pumped deep into the underground aquifer so that it can be used during times of the year where usage is higher. The ASR system is currently under construction, undergoing improvements and expansions to the system. As it currently stands, the ASR capacity is 8.71 million gallons per day. Because the ASR wells are located on relatively flat ground, there were no observed geologic hazards during our site reconnaissance.

Existing subsurface information was not available for this pump station. Based on the geologic mapping, the site is underlain by volcanic rock. Therefore, the regional seismic hazard mapping does not indicate permanent ground deformations from liquefaction induced settlement or seismic slope instability.

Exhibit 3-13: Photo of Woodmansee Park During Site Visit

3.13 Site 13 - Skyline Repeater Tower

Based on the geologic mapping, the site is underlain by volcanic rock. Therefore, the regional seismic hazard mapping does not indicate permanent ground deformations from liquefaction induced settlement or seismic slope instability.

3.14 Site 14 - Lone Oak Reservoir

Lone Oak Reservoir is a partially-buried reinforced concrete reservoir. Approximately 2 to 6 feet of the reservoir, which is 25 feet high, are exposed at the surface. The tank has a total capacity of 5.64 million gallons. At the reservoir site, there is a gentle slope to the south. During our site reconnaissance, no evidence of slope instability or soil creep was observed.

Existing subsurface information was not available for this reservoir. Based on the geologic mapping, the site is underlain by volcanic rock. Therefore, the regional seismic hazard mapping does not indicate permanent ground deformations from liquefaction induced settlement or seismic slope instability.

We reviewed publicly available geotechnical exploration logs from the intersection of Lone Oak Road SE and Mildred Lane SE, which is the intersection adjacent to the reservoir and were completed for the City of Salem Public Works Department. Two explorations were completed, and both included a monitoring well. Both explorations encountered residual soil to depths of 40 feet and both did not encounter groundwater. Logs of the explorations that we reviewed are included in Appendix G.

If subsurface conditions underlying the reservoir are similar to what was encountered in the nearby explorations, then we consider the potential for liquefaction and associated hazards to be low, which is consistent with the regional seismic mapping.

Exhibit 3-14: Photo of Lone Oak Reservoir During Site Visit

3.15 Site 15 - Creekside Pump Station

Creekside Pump Station is located just south of Lone Oak Reservoir. The pump station, constructed in 1998, contains three pumps and has a capacity of approximately 6 million gallons per day. The pump station sits on relatively flat ground at the bottom of a hill. During site reconnaissance, no signs of slope instability were observed.

Existing subsurface information was not available for this pump station. Based on the geologic mapping, the site is underlain by volcanic rock. Therefore, the regional seismic hazard mapping does not indicate permanent ground deformations from liquefaction induced settlement or seismic slope instability.

Exhibit 3-15: Photo of Creekside Pump Station During Site Visit

3.16 Site 16 - Champion Hill Reservoir

Champion Hill Reservoir is a 2.3-million-gallon reinforced concrete reservoir. The reservoir is located just south of the Salem city limits and is surrounded by vineyards. In the area around the tank, the ground is gently-sloping to the south. However, no signs of slope instability or soil creep were observed during our site reconnaissance.

Existing subsurface information was not available for this reservoir. Based on the geologic mapping, the site is underlain by volcanic rock. Therefore, the regional seismic hazard mapping does not indicate seismic related hazards as an issue for this site.

We reviewed publicly available geotechnical exploration logs from a site that is approximately 800 feet north of the intersection between Hylo Road SE and Champions Hill Road SE, which would be approximately 200 feet south of the reservoir. Three explorations were completed at this site with one exploration including a monitoring well. Two of the explorations indicate that approximately 40 to 63 feet of silt overlies decomposed basalt. The third exploration was performed to 25 feet and included a monitoring well. Groundwater was not observed within the monitoring well and was not noted on the other explorations. Water well logs that are within approximately 0.5 miles from the reservoir site indicate that groundwater is relatively deep (i.e. greater than 100 feet below the ground surface). Logs of the explorations that we reviewed are included in Appendix H.

If the reservoir is potentially founded on silty soil overlying rock and perched water was present, then the silt may be susceptible to liquefaction depending on its plasticity. However, if the static groundwater is similar to the nearby water wells, then the potential for liquefaction and associated seismic hazards is considered to be low, which is consistent with the regional seismic mapping. If the City has existing as-built information or geotechnical borings at this reservoir, we request that they be provided to the project team to better assess the seismic geohazards.

Exhibit 3-16: Photo of Champion Hill Reservoir During Site Visit

3.17 Site 17 - Boone Road Pump Station

Boone Road Pump Station was originally constructed in 1977 with modifications made in 1994 and again after 2001. Three pumps, with a total capacity of 12.96 million gallons per day are inside two structures on site. The pump station sits on flat ground and the were no observed slope or geologic hazards observed during our site reconnaissance.

Existing subsurface information was not available for this pump station. Based on the geologic mapping, the site is underlain by terrace deposits. However, the mapped terrace deposits in this area are mapped as coarse-grained deposits with a very low liquefaction hazard.

Exhibit 3-17: Photo of Boone Road Pump Station During Site Visit

3.18 Site 18 - Deer Park Pump Station

Deer Park Pump Station sits on an 8-inch-thick reinforced concrete slab foundation. The pump station, which was built in 1982, houses three pumps with a total capacity of approximately 5 million gallons per day. The pump station is on relatively flat ground and no geologic hazards were observed during our site reconnaissance.

Existing subsurface information was not available for this pump station. Based on the geologic mapping, the site is underlain by volcanic rock. Therefore, the regional seismic hazard mapping does not indicate permanent ground deformations from liquefaction induced settlement or seismic slope instability.

We reviewed publicly available geotechnical explorations completed in 1998 for the Oregon Department of Corrections at 5485 Turner Road SE, which is 300 to 500 feet west of the pump station site. The explorations indicated 6 to 10 feet of clay to silty clay overlying weathered basalt. Monitoring wells were also installed and indicate static groundwater ranges from 15 to 18 feet below the ground surface. Logs of the explorations we reviewed are included in Appendix I.

If subsurface conditions underlying the pump station are similar to what was encountered in the nearby exploration, then we consider the potential for liquefaction and associated hazards to be low, which is consistent with the regional seismic mapping.

Exhibit 3-18: Photo of Deer Park Pump Station During Site Visit

3.19 Site 19 - Mill Creek Reservoir

Mill Creek Reservoir is a 2.3-million-gallon reinforced concrete tank. The reservoir is adjacent to College Reservoir, near Corban University. The reservoir is near the top of a hill, where it slopes to the southwest. We did not observe indicators or evidence of slope instability around the tank.

We reviewed publicly available water well and geotechnical exploration logs completed for sites near the reservoir. We found two explorations completed for 5358 Deer Park Dr SE in 2015, which is approximately 300 to 400 feet south of the reservoir. These two explorations indicated sandy silt overlying weathered basalt. The contact with the weathered basalt varied from 9 to 18 feet below the ground surface. Neither of these explorations indicated observations of groundwater. A water well was completed for 5583 Jenniches Ln SE in 2005, which is approximately 0.5 mile southeast of the reservoir indicated groundwater was at a depth of 62 feet below the ground surface. Logs of the explorations that we reviewed is included in Appendix J.

If subsurface conditions underlying the reservoir are similar to what was encountered in the nearby explorations, the potential for permanent ground deformation from liquefaction and seismic slope instability is low, which is consistent with the regional seismic mapping. The primary hazard at the site is strong ground motions.

Exhibit 3-19: Photo of Mill Creek Reservoir During Site Visit

3.20 Site 20 - Turner Control Facility

We understand from review of a Geotechnical Engineering Report prepared by Foundation Engineering, Inc., (FEI) that the Turner Control Facility was being designed for replacement with a larger structure in 2005. The existing Turner Control Facility is on relatively flat ground.

As part of the previous Geotechnical Engineering Report, one boring, designated BH-1, was drilled near the Turner Control Building on March 10, 2005. The borehole was advanced to approximately 16.6 feet prior to encountering practical refusal. Another exploration, designated BH-12, was performed by FEI southwest of the control building on May 1, 1998 and was advanced to a maximum depth of 21.5 feet.

Subsurface conditions encountered in boring BH-1 consisted of alluvial soils that were comprised of very stiff, silty clay to a depth of approximately 5 feet, which was underlain by dense to very dense gravel to the bottom of the hole at 16.6 feet. Subsurface conditions encountered in boring BH-12 also consisted of alluvial soils that were comprised of medium stiff silt to approximately 3.5 feet, which was underlain by dense to very dense sandy gravel with cobbles to approximately 19 feet. The FEI report indicates that the gravels were underlain by dense sand from 19 to 21.5 feet.

Based on the geologic mapping, the site is underlain by terrace deposits. The regional hazard mapping indicates that the terrace deposits underlying the Turner Control Building have a low liquefaction hazard, which is consistent with the subsurface conditions encountered in the previous explorations.

3.21 Site 21 - Franzen Reservoir and Transmission Tower

Franzen Reservoir, located in the hills above Turner Oregon, was built in 1951 and has a capacity of just over 92 million gallons. A transmission tower was later constructed on the site. The reservoir consists of two cells. We reviewed the following documents provided for Franzen Reservoir:

- Squire Associates, 2001, Geotechnical Schematic Design Report Franzen Reservoir Rehabilitation Project;
- Squire Associates, 2002, Geotechnical Basis of Design Report Franzen Reservoir Rehabilitation Project; and
- City of Salem Public Works, 2008, Slump Failure at Franzen Reservoir.

The 2002 Geotechnical Basis of Design Report and study performed by Squire Associates included 22 borings, 18 test pits, 12 shallow hand augers, and four seismic refraction survey lines. The report included an evaluation of slope stability including seismic slope stability as well as evaluation of other seismic related geologic hazards such as fault rupture and liquefaction. That study identified what they believed to be an unknown fault with indeterminate activity extending through the middle of the reservoir based on offsets in the geologic units encountered in the explorations. They concluded that this unmapped fault is unlikely to experience surface ruptures for earthquake magnitudes less than M5.6, which is what they considered to be the maximum magnitude for this unmapped fault.

This study also concluded that liquefaction was not a hazard as the site is primarily underlain by residual soil, and coarse-grained subunits contained plastic fines contents ranging between 30 to 40 percent. Seismic slope stability was also performed and concluded that slopes were generally stable under the design seismic loading condition but that the downstream slope may experience deformations of up to 6 inches. A site plan and profile drawings from this study are included in Appendix L.

According to a 2008 memorandum prepared by the City of Salem, two slump failures occurred within the cut slope of the west cell of the reservoir. Each failure was approximately halfway down the slope from the top of the reservoir. Each slumped area is about 10 feet x 10 feet in size. Based on information contained within the memorandum, the failures were discovered when the plastic liner was removed for routine maintenance. No definitive cause of failure was stated in memorandum; however, the memo indicates that the original geotechnical engineer for the 2004 upgrades of the reservoirs (Barry Meyers) visited the site and that based on the type of failure groundwater was not anticipated to be the cause. The memo also indicates that a similar failure occurred at or near the site during the first year of the reservoir's operation. We understand that the failures were repaired. During our site visit the reservoir was covered, and we did not observe evidence slope instability; however, the reservoir cut walls were covered with a plastic liner. No information was available on how the cells were repaired at the time of this report.

Based on our site visit and existing subsurface information provided to us, the potential for liquefaction induced settlement of the native soils beneath the reservoir is low, which is consistent with the regional seismic mapping. However, because the failures on the embankment wall have occurred during static conditions (i.e. no-ground motion) it is our opinion the potential for permanent ground deformation from seismic slope instability is not adequately captured in the HAZUS model. Additional slope modeling outside of the of the current scope of the seismic geohazard evaluation would be required to better quantify the hazard, but it is our qualitative assessment based on the historic slumping the potential for seismic slope instability of the embankment may be moderate to high.

Exhibit 3-20: Photo of Franzen Reservoir During Site Visit

Exhibit 3-21: Photo of Slump Failure along Cut Slope of West Cell from 2008 (Photo Provided by the City of Salem)

3.22 Site 22 - Geren Island Water Treatment Plant and Transmission Tower

Geren Island is located in the Santiam River approximately 20 miles east of Salem near Stayton. At the Geren Island Water Treatment Plant, which was constructed in 1937, water from the North Santiam River is taken from the river and filtered through sand filters and disinfected with chlorine. The treatment plant is the main source of drinking water for the City of Salem and is also an active construction site where a new ozone treatment facility, scheduled to be finished in 2021, is being built. Over the course of several decades numerous improvements have been made to improve treated water quality, capacity and reliability. Not including the improvements currently under construction, the Geren Island Water Treatment Plant includes the following elements:

- Surface water intakes and metering facilities, including one active intake referred to as the "Middle Intake" and two intakes that have been abandoned as the depth and shape of the channel has changed over time and the intakes are no longer viable;

- Three groundwater supply wells;
- Pre-treatment facilities consisting of primary coagulant, pH adjustment and two roughing filters;
- Three slow sand filters (2-cells each);
- Post-treatment facilities consisting of primary disinfection, pH adjustment, and fluoridation; and
- Office buildings for staff, located adjacent to post treatment facilities.

The water treatment plant site is relatively flat. However, there are embankments for the side walls of the various filter facilities on-site, and there are slopes along the banks of the North Santiam River Channels. During our site visit, we did not observe signs of slope instability.

Based on the geologic map, the site is underlain by alluvial soils. Therefore, the regional seismic geohazard mapping indicates high liquefaction and other associated hazards (i.e. lateral spreading) are present at this site.

Existing explorations for Geren Island were provided to us or were found within our records. We reviewed the following geotechnical reports for Geren Island:

- Shannon & Wilson, 1987, Geotechnical Studies Geren Island Water Intake Facilities;
- Foundation Engineering, Inc., 1996, Geren Island Treatment Facility Improvements Geotechnical Investigation;
- Squire, 2004, Foundation Investigation at Geren Island Corrosion Control Facility - Soda Ash Storage Silo(s) and Equipment Building; and
- McMillen Jacobs Associates, 2019, Geren Island Water Treatment Plant Improvement Project Phase 1 - Ozone Facility.

These four reports include logs of 12 test pit explorations and 10 borings. A site plan showing the location of known previous explorations is included in Figure 18. Available logs of the explorations we reviewed are included Appendix M.

Subsurface conditions indicated on the exploration logs indicate that most of the site is primarily underlain by gravel alluvium. Standard Penetration Test samples collected within the gravel indicate that it is dense to very dense. Some of the test pits and boring logs indicate that there are localized areas of loose to medium dense silty sand overlying the gravels.

We evaluated the liquefaction potential of the soils in the borings in accordance with methods described by Boulanger and Idriss (2014) for a magnitude 9 earthquake and the peak ground acceleration shown in Table 1 (0.16 g). These analyses indicate that the factors of safety against liquefaction for a magnitude 9 CSZ event are greater than 1.0. Our analysis assumed a groundwater depth of 8 feet below the ground surface based on measured groundwater conditions in the borings, which is below the bottom of the localized areas of loose to medium dense sand noted on the test pit and boring logs. If groundwater levels are higher than assumed in our analyses, the loose to medium dense sand would show zones with factors of safety against liquefaction of less than 1.0. We also note that the subsurface conditions on the site vary from loose sand to very dense gravel, with the density and particle size of the alluvial deposits which form the island related to the energy and flow in the Santiam River during deposition. Consequently, the soil type, density, and strength characteristics can change over relatively short vertical and horizontal distances.

Liquefaction potential analysis of the available boring logs did not identify a liquefaction hazard for the Cascadia Subduction Zone ground motions considered in this study. Therefore, the seismic geohazard mapping overestimates the liquefaction induced settlement on those portions of the island where borings are available. However, in areas where there are no borings to indicate a low liquefaction hazard, we recommend that the liquefaction geohazard information indicated on the seismic geohazard maps be assumed.

Exhibit 3-22: Photo Showing Area Near New Ozone Facility

Exhibit 3-23: Photo Showing Middle Intake at Geren Island Water Treatment Plant

Exhibit 3-24: Photo Showing Geren Island Transmission Tower.

We note that the Geren Island WTP is downstream of Detroit Lake, a Lake impounded by the Detroit Dam, and the Big Cliff Reservoir, a reservoir impounded by the Big Cliff Dam. These reservoirs and dams are owned and operated by the U.S. Army Corps of Engineers (USACE). The Water Treatment Plant is also downstream of the Upper Bennett Dam and portions of Water Treatment Plant Facilities are downstream of the Lower Bennett Dam. The Lower and Upper Bennett Dams are co-owned by the Santiam Flood Control District and the City of Salem. Our study did not include an evaluation of the Upper and Lower Bennett Dams or the dams owned by the USACE.

3.23 Sites 23 and 24 - Upper and Lower Transmission Mains

We understand that the City of Salem's water transmission backbone is separated into an upper and a lower segment. The upper and lower transmission mains are further divided into two lines. The upper transmission lines extend from the Geren Island WTP and terminate in Turner at the Turner Control Valves, which is northwest of Franzen Reservoir. Line 1 for the upper transmission main is 36 inches in diameter, and Line 2 for the upper transmission main is 54 inches in diameter. The lower transmission lines extend from the Turner Control Valves and Line 1 terminates at the Fairmont Reservoir and Line 2 terminates at the Mountain View Reservoir. Line 2 of the lower transmission main crosses under the Willamette River at Sunset Park.

Based on the geologic mapping, both lines of the upper transmission main segment are primarily within terrace deposits. Based on the regional seismic geohazard mapping, the terrace deposits in this area are characterized as coarse-grained sediments that have a low liquefaction hazard. There are portions of the upper transmission mains that are within mapped areas of alluvium, specifically near Geren Island and just south of Turner. Based on the regional seismic geohazard mapping, the alluvial soils have a high liquefaction hazard.

Based on geologic mapping, both lines of the lower transmission main segment are primarily within terrace deposits and Missoula flood deposits. The regional seismic geohazard mapping indicates that the terrace and Missoula flood deposits have a low liquefaction potential in the HAZUS model. However, there are areas in the north part of Salem where the Missoula flood deposits have a moderate liquefaction hazard, and Line 2 of the lower transmission main crosses through these regions. Line 2 of the lower transmission main also crosses through mapped alluvium as it approaches the Willamette River. This area is mapped as having a moderate to high liquefaction hazard.

The other liquefaction-related hazard for Line 2 of the lower transmission main segment is lateral spreading near the Willamette River crossing. There is also a potential for lateral spreading where the upper transmission main segment crosses the North Santiam River. Lateral spreading can occur if soil liquefaction develops during a seismic event and the ground acceleration (inertial force) surpasses the yield acceleration (shear strength) of the liquefied soil. The displacements are cumulative and permanent and can occur on mild slopes or level ground adjacent to a much steeper slope or vertical face (free face).

Existing information related to this river crossing was not provided. Also, the depth of the pipeline was unknown to Shannon & Wilson at the time of this report. Based on a paper published by Youd in 2018, if a pipeline was buried 1H below the bottom of a channel, then the shear zone generated by lateral spread is typically above and non-damaging to the pipe.

Based on the regional seismic hazard mapping, permanent ground deformations due to lateral spreading are estimated to be up to 11 inches at distances of up to 350 feet west of the Willamette River crossing and 4 inches at distances of up to 1,000 feet east of the Willamette River crossing. Permanent ground deformations due to lateral spreading are estimated to be up to 9 inches at distances of up to 1,000 feet on either side of the North Santiam River crossing.

We also understand that there are lower Willamette River crossings at Marion Street and Center Street and that the pipelines for these crossings are supported by the Marion and Center Street bridges that are owned and maintained by ODOT. Our study did not include an evaluation of the lower Willamette River bridge crossings owned by ODOT. However, Shannon & Wilson is involved with a planned seismic retrofit of the Center Street bridge for ODOT and has submitted a draft Preliminary Geotechnical Memorandum (Shannon & Wilson, 2018).

The draft Preliminary Geotechnical Memorandum prepared by Shannon & Wilson did not include subsurface explorations, and preliminary results were based on historic explorations performed for the existing bridge. Based on the Preliminary Geotechnical Memorandum, soils underlying the West Approach and River Spans are susceptible to liquefaction. Up to 8 inches of liquefaction-induced settlement is estimated for the West Approach and River Spans. At the time the Preliminary Memorandum was prepared, Shannon & Wilson determined that there was not sufficient SPT data to perform a liquefaction analysis for the East Approach. However, Shannon & Wilson did include review of one existing test hole performed for the East Approach, which showed low liquefaction susceptibility. Lateral spreading was also noted as a hazard for the west riverbank, but a low potential for the east riverbank. Existing geotechnical data for the bridge from Historic Record Drawings provide by ODOT are included in Appendix N.

We recommend that the project team communicate with ODOT to understand the expected performance of the bridges and use that information to estimate the performance of the pipelines supported by the bridges.

Also, note that based on fault mapping, the upper and lower transmission mains appear to cross two Class A faults. A discussion of the faults and potential for surface rupture are included in Section 2.11 of this report.

4 LIMITATIONS

Our interpretations, conclusions and geotechnical considerations are based on a desktop study including review of publicly available information prepared by others, and a single site visit. No explorations were performed to evaluate geotechnical site conditions and make interpretations. Should proposed development of sites within the study area occur, we recommend that appropriate explorations and site characterization testing and evaluation be done, a detailed site-specific geotechnical study be performed, and geotechnical firms with experience in both static and seismic conditions perform the work.

Within the limitations of scope, schedule, and budget, the conclusions presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. Shannon & Wilson makes no other warranty, either express or implied. These conclusions were based on Shannon & Wilson's understanding of the project as described in this report and the site conditions as observed at the time of our field reconnaissance.

This report was prepared for the exclusive use of Black & Veatch and City of Salem, Oregon. The scope of Shannon & Wilson's present work did not include environmental assessments or evaluations regarding the presence or absence of hazardous or toxic substances in the soil, surface water, groundwater, or air, on or below or around this sites, or for the evaluation or disposal of contaminated soils or groundwater should any be encountered.

Shannon & Wilson has prepared "Important Information About Your Geotechnical/Environmental Report" to assist you and others in understanding the use and limitations of our reports and is attached at the end of this report.

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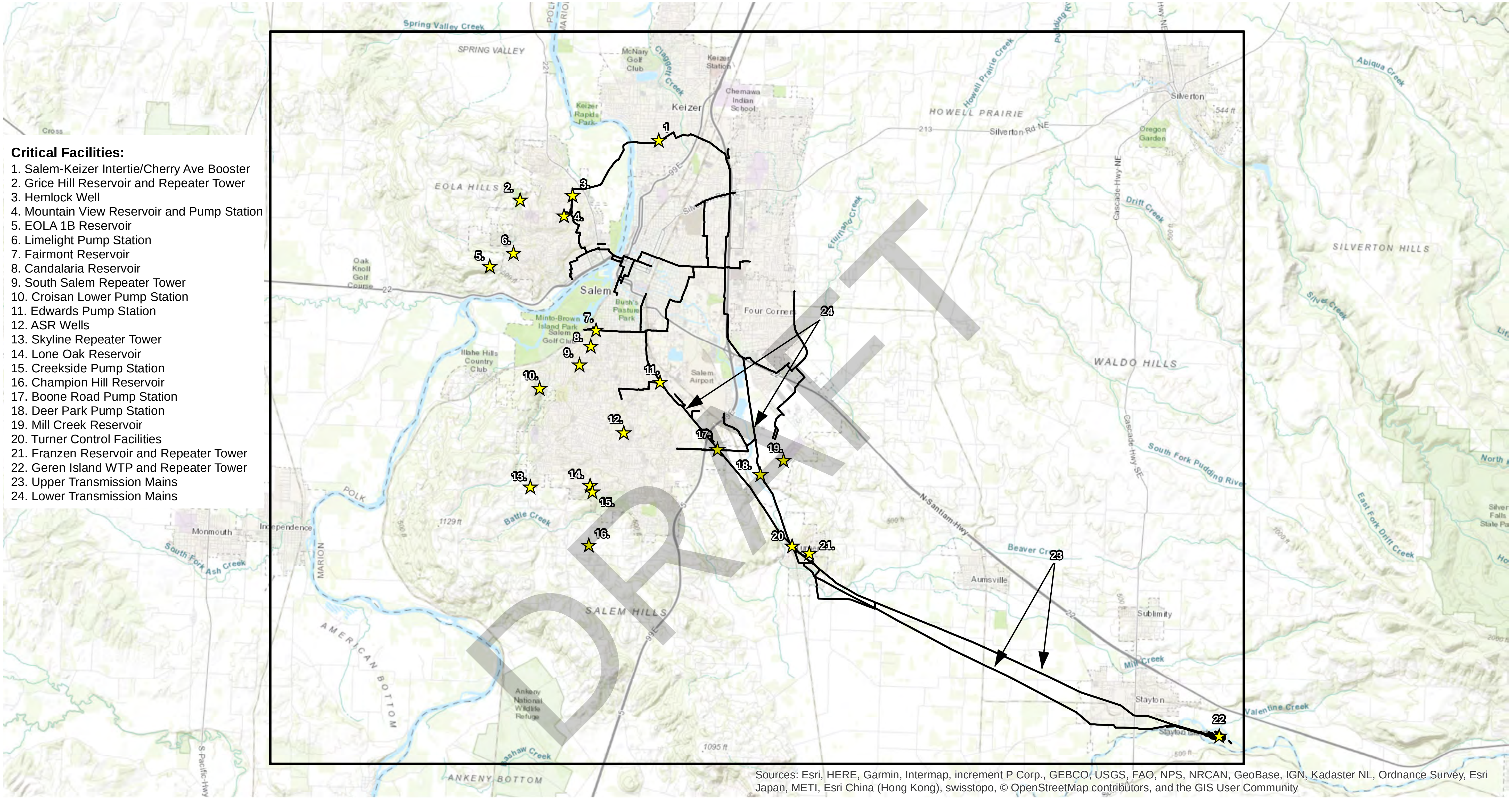

Critical Facilities:

1. Salem-Keizer Intertie/Cherry Ave Booster
2. Grice Hill Reservoir and Repeater Tower
3. Hemlock Well
4. Mountain View Reservoir and Pump Station
5. EOLA 1B Reservoir
6. Limelight Pump Station
7. Fairmont Reservoir
8. Candalaria Reservoir
9. South Salem Repeater Tower
10. Croisan Lower Pump Station
11. Edwards Pump Station
12. ASR Wells
13. Skyline Repeater Tower
14. Lone Oak Reservoir
15. Creekside Pump Station
16. Champion Hill Reservoir
17. Boone Road Pump Station
18. Deer Park Pump Station
19. Mill Creek Reservoir
20. Turner Control Facilities
21. Franzen Reservoir and Repeater Tower
22. Geren Island WTP and Repeater Tower
23. Upper Transmission Mains
24. Lower Transmission Mains

Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

LEGEND

☆	Critical Facilities
—	City of Salem Pipeline Backbone
☐	Study Area

0 6,000 12,000 24,000

Scale in Feet

NOTES

1. City of Salem pipelines provided by Black & Veatch.

Salem Seismic
Salem, Oregon

SITE MAP

May 2021 105679

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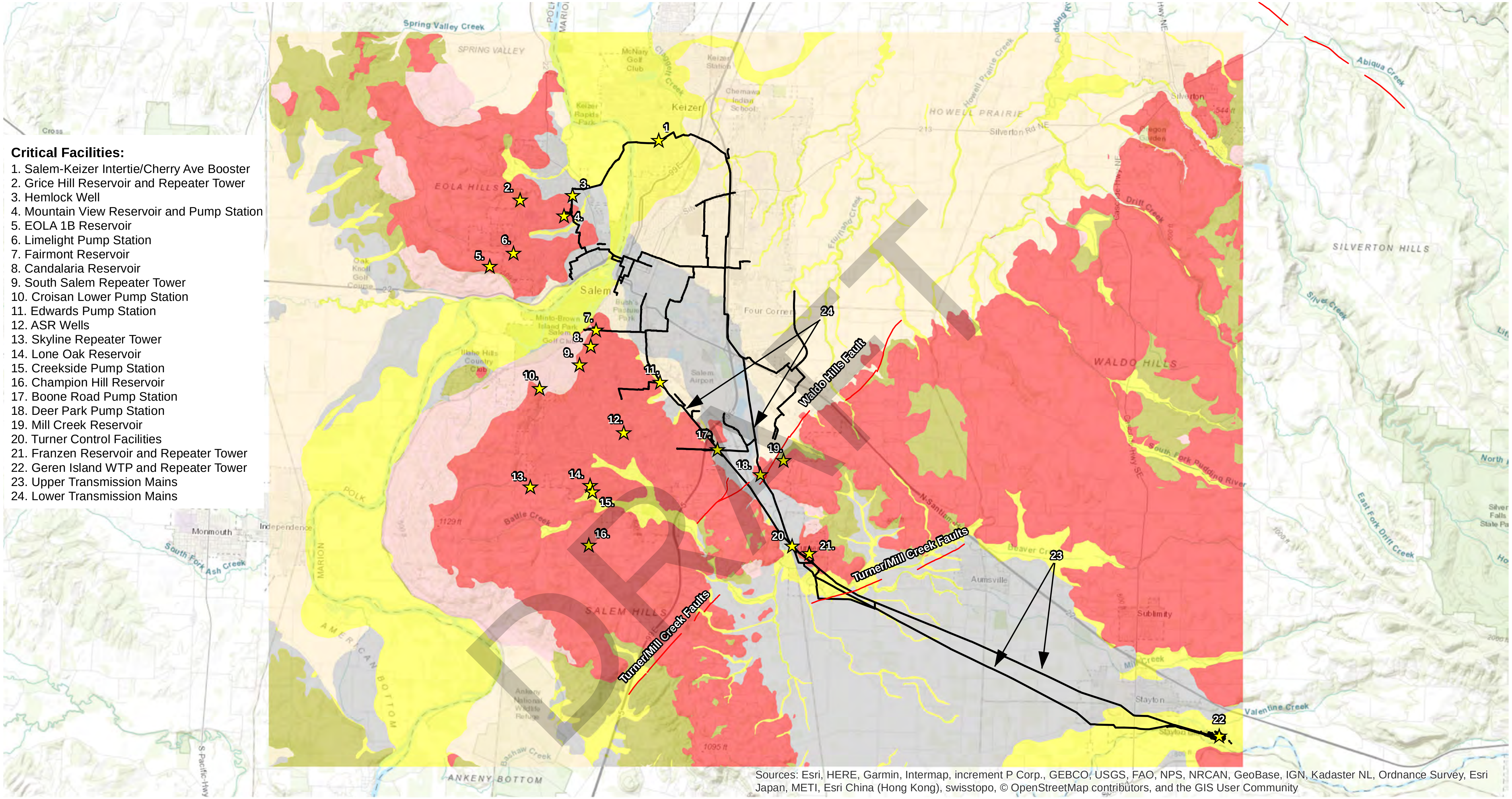
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 1

Filename: T:\Projects\PDX\105000s\105679 - Salem Seismic\Avmxd\11x17\New\Fig 1 - Site Map.mxd Date: 5/5/2021 Login: DSJ



Critical Facilities:

1. Salem-Keizer Intertie/Cherry Ave Booster
2. Grice Hill Reservoir and Repeater Tower
3. Hemlock Well
4. Mountain View Reservoir and Pump Station
5. EOLA 1B Reservoir
6. Limelight Pump Station
7. Fairmont Reservoir
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10. Croisan Lower Pump Station
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12. ASR Wells
13. Skyline Repeater Tower
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18. Deer Park Pump Station
19. Mill Creek Reservoir
20. Turner Control Facilities
21. Franzen Reservoir and Repeater Tower
22. Geren Island WTP and Repeater Tower
23. Upper Transmission Mains
24. Lower Transmission Mains

Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

LEGEND

☆ Critical Facilities

— City of Salem Pipeline Backbone

— Approximate Location of Quaternary Fault

Geologic Map Unit

🟨 Alluvial Deposits	▨ Landslide Deposits
▨ Missoula Flood deposits	🟩 Sedimentary Rock
▨ Terrace Deposits	🟥 Volcanic Rock

0 6,000 12,000 24,000

Scale in Feet

NOTES

1. Geologic map units from DOGAMI publications OGDC-7 and SLIDO-4.0. See text for details.
2. City of Salem pipelines provided by Black & Veatch.

Salem Seismic
Salem, Oregon

GEOLOGIC MAP

May 2021 105679

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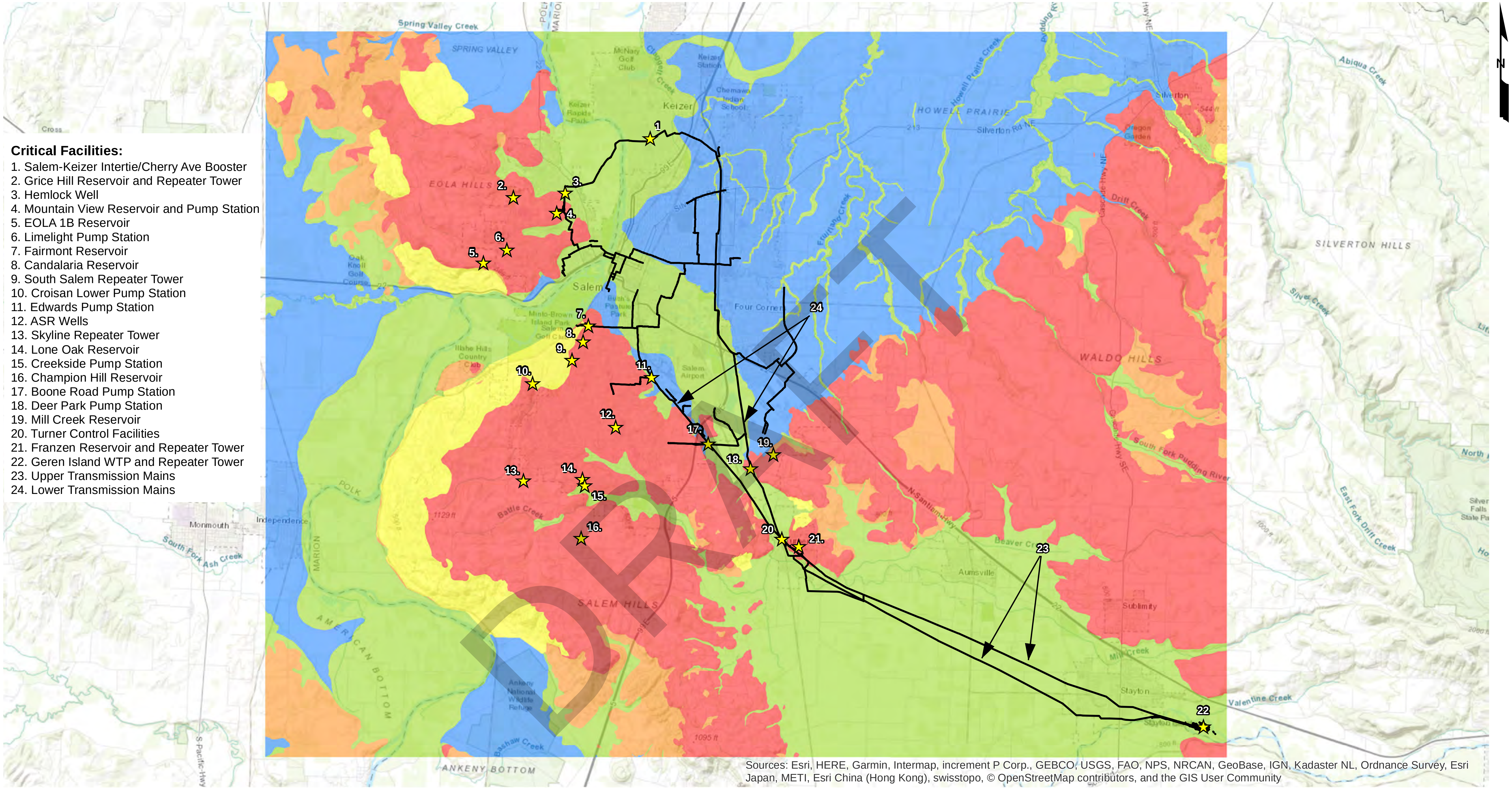
FIG. 2

Filename: T:\Projects\PDX\105000s\105679 - Salem Seismic\Avmxd\11x17\Fig 1 - Geologic Map.mxd Date: 5/6/2021 Login: DSJ



Critical Facilities:

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24. Lower Transmission Mains

Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

LEGEND

★ Critical Facilities

— City of Salem Pipeline Backbone

Shear Wave Velocity, Vs30 (m/s)

🟦	190
🟩	250
🟨	360
🟧	920
🟥	968

0 6,000 12,000 24,000

Scale in Feet

NOTES

1. Vs30 values based on Vs values from DOGAMI publication GMS-105. See text for details.
2. City of Salem pipelines provided by Black & Veatch.

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Salem, Oregon

SHEAR WAVE VELOCITY, Vs 30

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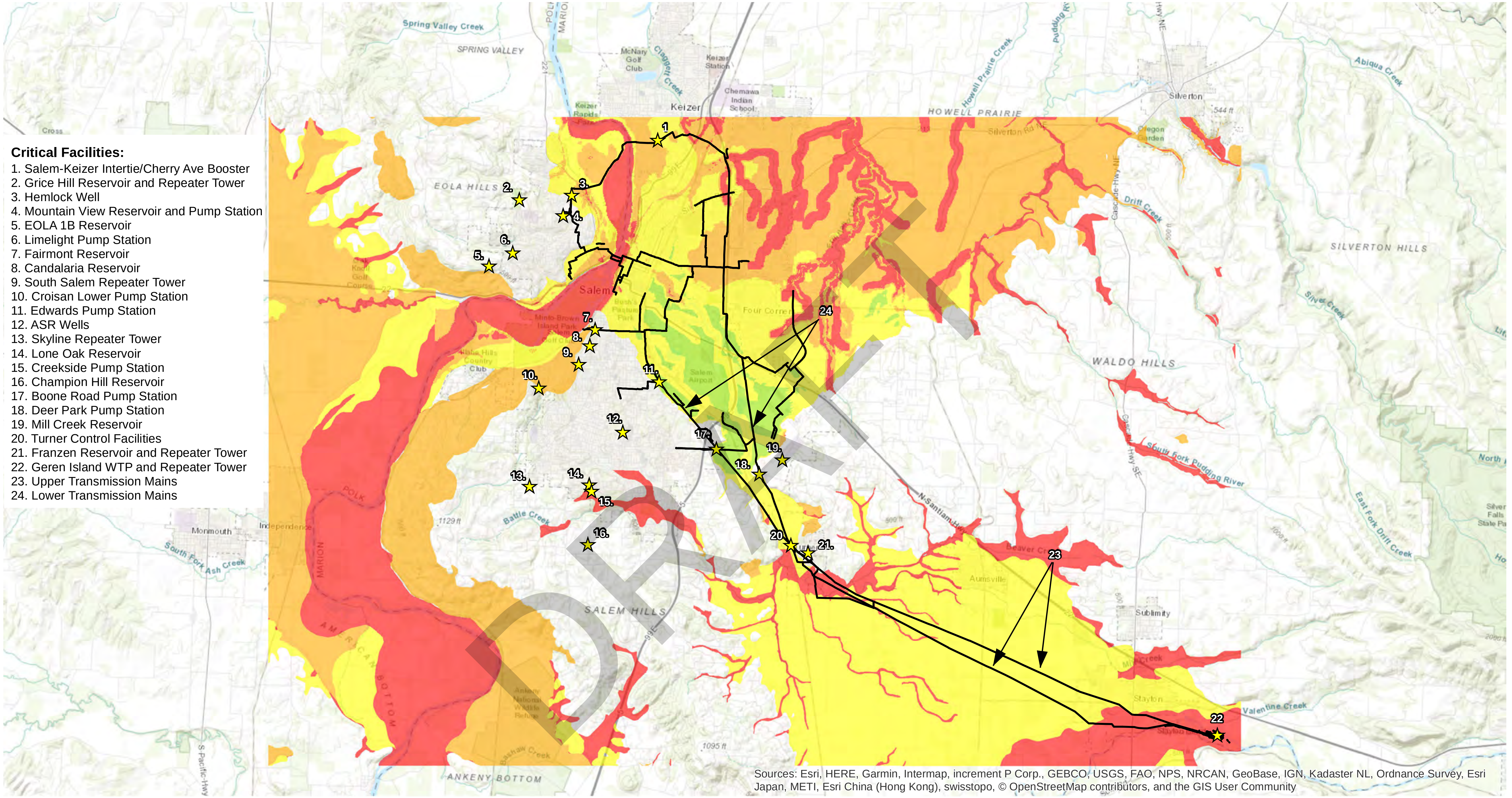
FIG. 3

Filename: T:\Projects\PDX\105000s\105679 - Salem Seismic\Avmxd\11x17\Fig 2 - Vs30.mxd Date: 4/22/2021 Login: DSJ



Critical Facilities:

1. Salem-Keizer Intertie/Cherry Ave Booster
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24. Lower Transmission Mains

Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

LEGEND

★ Critical Facilities

Liquefaction Hazard

🟩 Very Low
🟨 Low
🟧 Moderate
🟥 High

0 6,000 12,000 24,000

Scale in Feet

NOTES

1. Liquefaction hazard developed from data provided with DOGAMI publications GMS-105 and OGDC-7, the Youd and Perkins, 1978 methodology, and knowledge of regional liquefaction hazards.
2. City of Salem pipelines provided by Black & Veatch.

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LIQUEFACTION HAZARD

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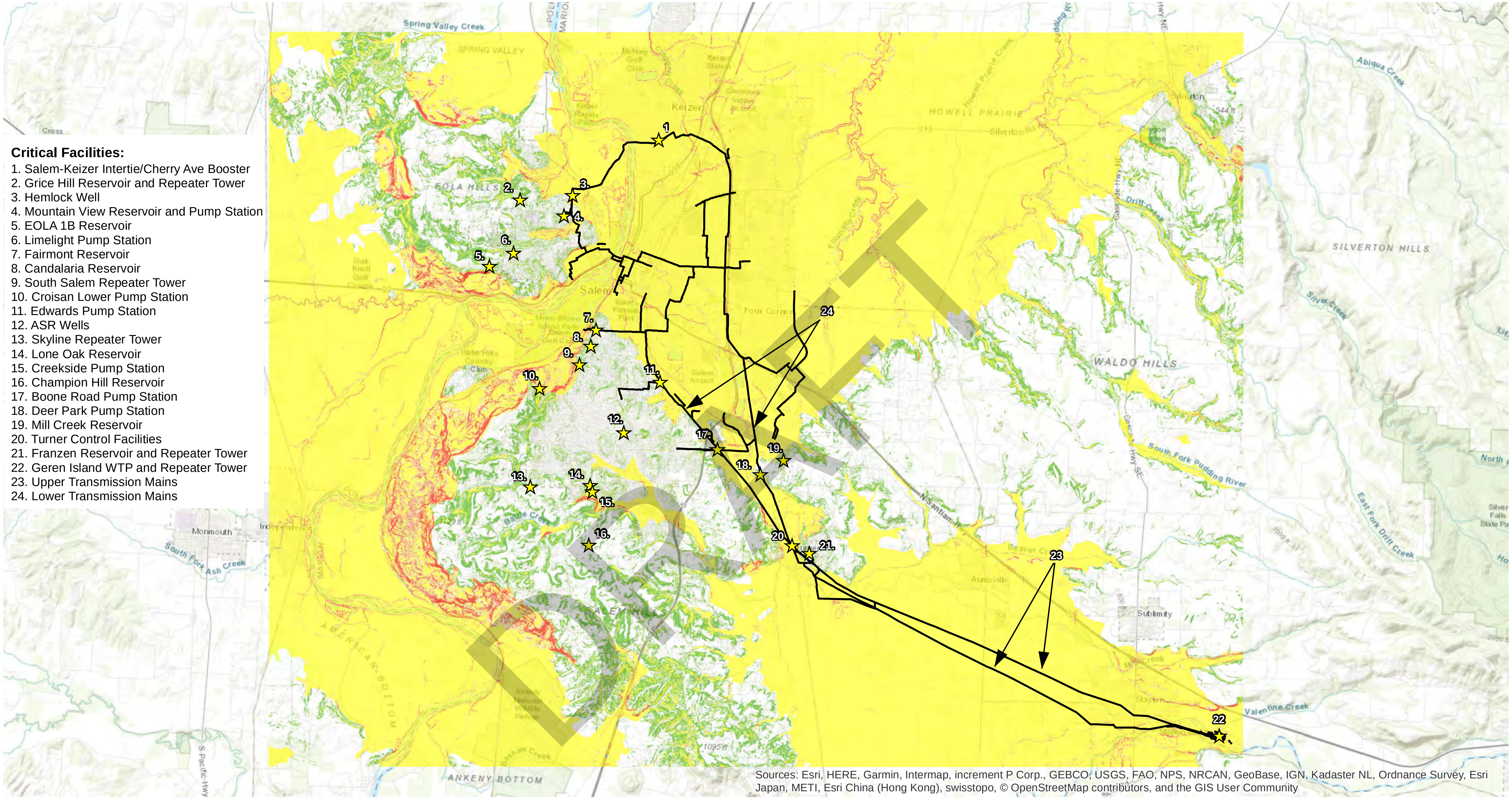
FIG. 4

Filename: T:\Projects\PDX\105000s\105679 - Salem Seismic\Avmxd\11x17\Fig 3 - Liquefaction Hazard.mxd Date: 4/22/2021 Login: DSJ



Critical Facilities:

1. Salem-Keizer Intertie/Cherry Ave Booster
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24. Lower Transmission Mains

Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

LEGEND

Landslide Susceptibility in Dry Conditions (Scale of 0-10)

- ⬜ 0
- 🟩 3
- 🟩 4
- 🟨 5
- ⬜ 6
- 🟧 7
- 🟥 9

★ Critical Facilities

— City of Salem Pipeline Backbone

0 6,000 12,000 24,000

Scale in Feet

NOTES

1. Landslide Susceptibility calculated from data provided with DOGAMI publications SLIDO-4.0, O-12-02, OGDC-6, and LiDAR. Methodology taken from HAZUS. See text for details.
2. City of Salem pipelines provided by Black & Veatch.

Salem Seismic
Salem, Oregon

LANDSLIDE SUSCEPTIBILITY (DRY CONDITIONS)

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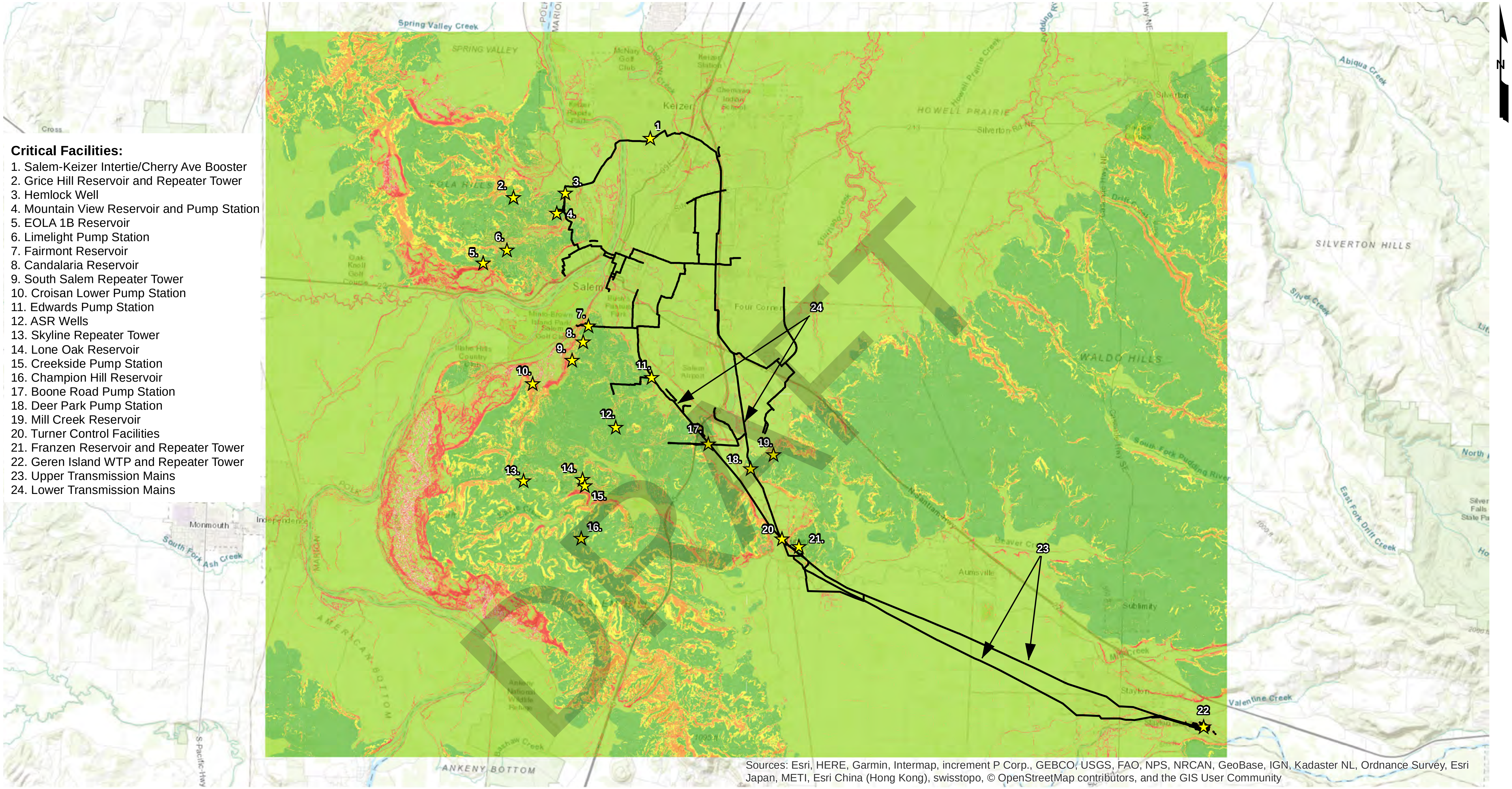
FIG. 5

Filename: T:\Projects\PDX\105000s\105679 - Salem Seismic\Avmxd\11x17\Fig 4 - Landslide Susceptibility (Dry).mxd Date: 4/22/2021 Login: DSJ



Critical Facilities:

1. Salem-Keizer Intertie/Cherry Ave Booster
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Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

LEGEND

Landslide Susceptibility in Wet Conditions

(Scale of 0-10)

🟩 5
🟩 7
🟨 8
🟧 9
🟥 10

★ Critical Facilities

— City of Salem Pipeline Backbone

0 6,000 12,000 24,000
Scale in Feet

NOTES
1. Landslide Susceptibility calculated from data provided with DOGAMI publications SLIDO-4.0, O-12-02, OGDC-6, and LiDAR. Methodology taken from HAZUS. See text for details.
2. City of Salem pipelines provided by Black & Veatch.

Salem Seismic
Salem, Oregon

LANDSLIDE SUSCEPTIBILITY (WET CONDITIONS)

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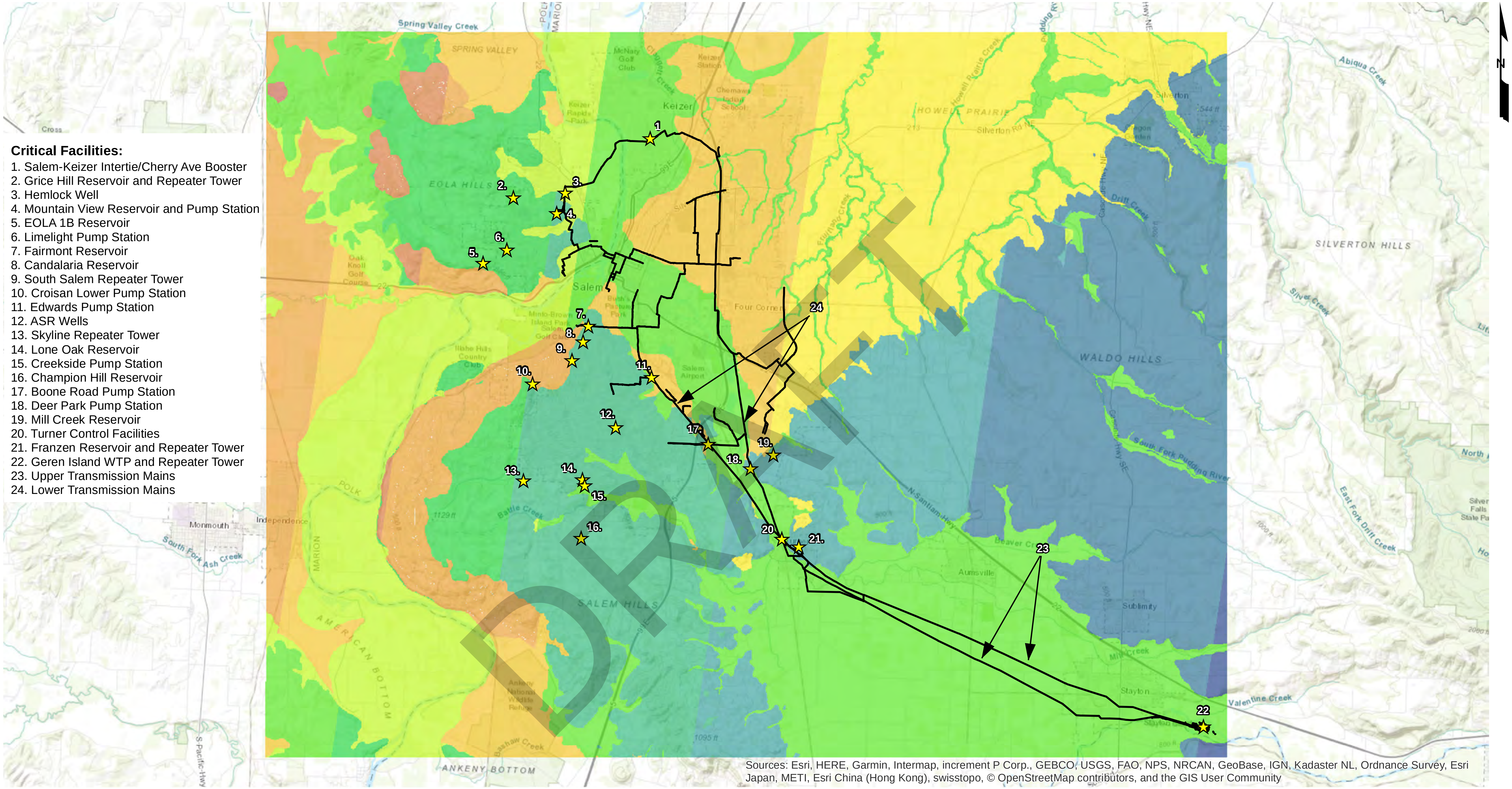
FIG. 6

Filename: T:\Projects\PDX\105000s\105679 - Salem Seismic\Avmxd\11x17\Fig 5 - Landslide Susceptibility (Wet).mxd Date: 4/22/2021 Login: DSJ



Critical Facilities:

1. Salem-Keizer Intertie/Cherry Ave Booster
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24. Lower Transmission Mains

Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

LEGEND

Site PGA (g)

🟦 0.11 - 0.12
🟦 0.12 - 0.13
🟦 0.13 - 0.14
🟩 0.14 - 0.15
🟩 0.15 - 0.16
🟩 0.16 - 0.17
🟩 0.17 - 0.18
🟨 0.18 - 0.19
🟨 0.19 - 0.2
🟧 0.2 - 0.21
🟧 0.21 - 0.22
🟥 0.22 - 0.23

☆ Critical Facilities
— City of Salem Pipeline Backbone

0 6,000 12,000 24,000
Scale in Feet

NOTES

1. PGA map for the magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided in DOGAMI publication O-13-06 and methodology in Boore and Atkinson, 2008. See text for details.
2. City of Salem pipelines provided by Black & Veatch.

Salem Seismic
Salem, Oregon

PEAK GROUND ACCELERATION, PGA

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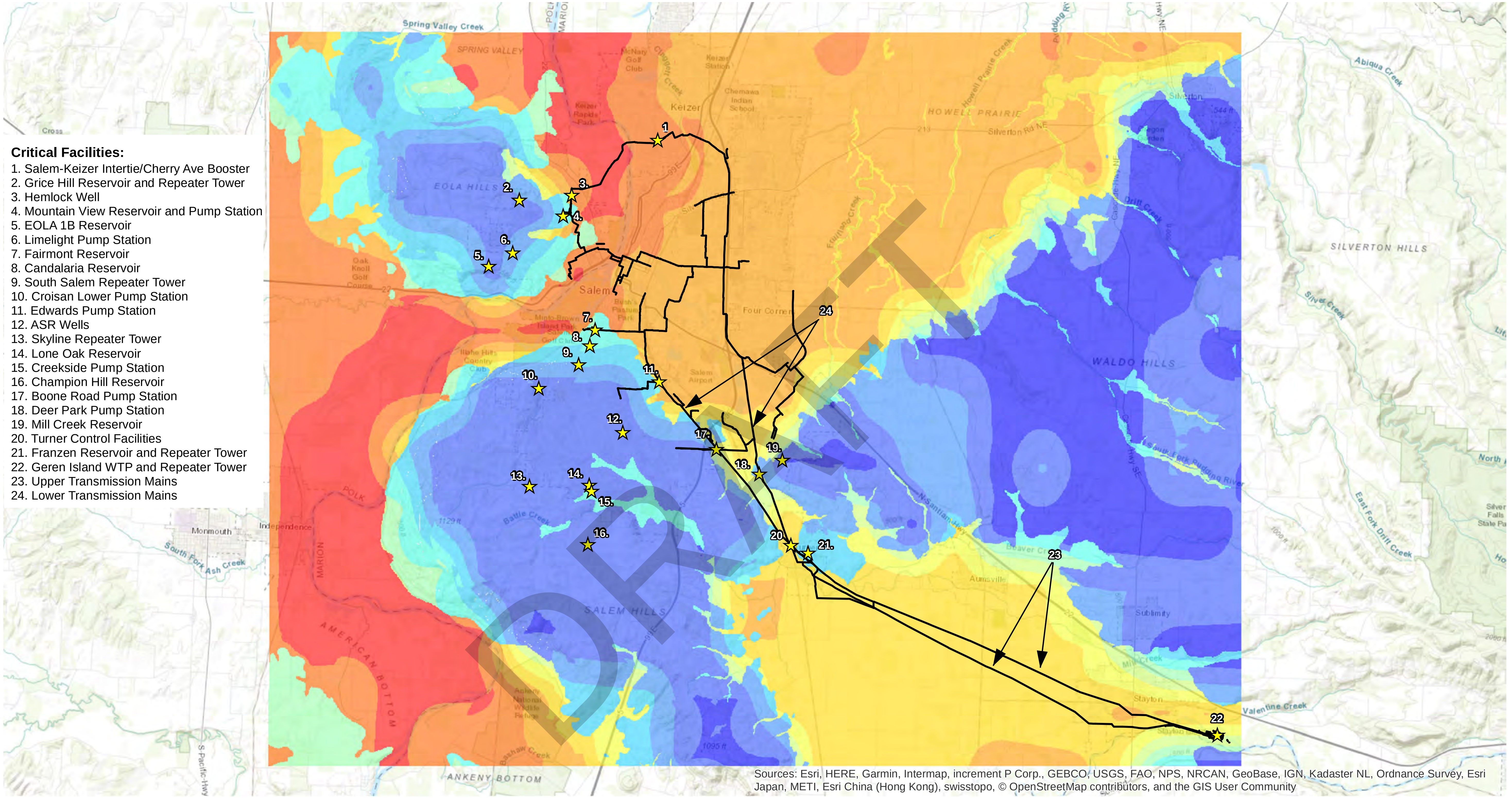
FIG. 7

Filename: T:\Projects\PDX\105000s\105679 - Salem Seismic\Avmxd\11x17\Fig 6 - PGA.mxd Date: 4/22/2021 Login: DSJ



Critical Facilities:
1. Salem-Keizer Intertie/Cherry Ave Booster
2. Grice Hill Reservoir and Repeater Tower
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4. Mountain View Reservoir and Pump Station
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Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

LEGEND

Site SA 0.3 (g)

🟦 0.21 - 0.26
🟦 0.26 - 0.31
🟦 0.31 - 0.36
🟦 0.36 - 0.41
🟩 0.41 - 0.46
🟩 0.46 - 0.51
🟩 0.51 - 0.56
🟨 0.56 - 0.61
🟨 0.61 - 0.66
🟧 0.66 - 0.71
🟧 0.71 - 0.76
🟥 0.76 - 0.81

☆ Critical Facilities
— City of Salem Pipeline Backbone

0 6,000 12,000 24,000
Scale in Feet

NOTES
1. SA0.3 map for the magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided with the USGS Scenario published September 20, 2011, and DOGAMI publications O-12-02 and OGDC-6. See text for details.
2. City of Salem pipelines provided by Black & Veatch.

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Salem, Oregon

0.3-SECOND SPECTRAL ACCELERATION, SA0.3

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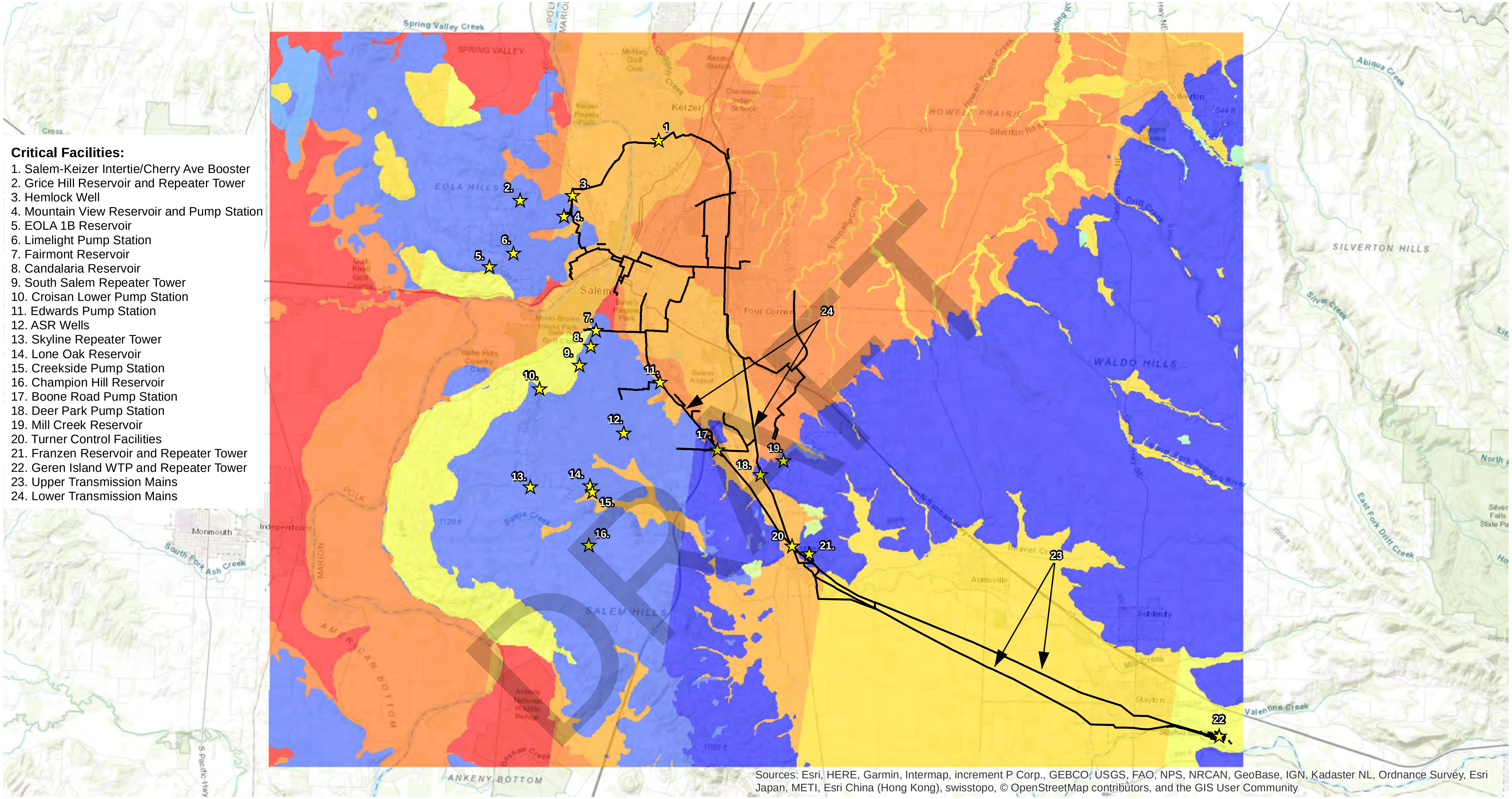
FIG. 8

Filename: T:\Projects\PDX\105000s\105679 - Salem Seismic\Avmxd\11x17\Fig 7 - SA03.mxd Date: 5/6/2021 Login: DSJ



Critical Facilities:
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Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

LEGEND

Site SA1 (g)

🟦	0.10 - 0.12	🟦	0.18 - 0.2	🟨	0.26 - 0.28
🟦	0.12 - 0.14	🟩	0.2 - 0.22	🟧	0.28 - 0.3
🟦	0.14 - 0.16	🟩	0.22 - 0.24	🟧	0.3 - 0.32
🟦	0.16 - 0.18	🟨	0.24 - 0.26	🟥	0.32 - 0.34

☆ Critical Facilities

— City of Salem Pipeline Backbone

0 6,000 12,000 24,000

Scale in Feet

NOTES

1. SA1 map for the magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided in DOGAMI publications O-13-06 and OGDC-6, and methodology in Boore and Atkinson, 2008. See text for details.
2. City of Salem pipelines provided by Black & Veatch.

Salem Seismic
Salem, Oregon

1-SECOND SPECTRAL ACCELERATION, SA1

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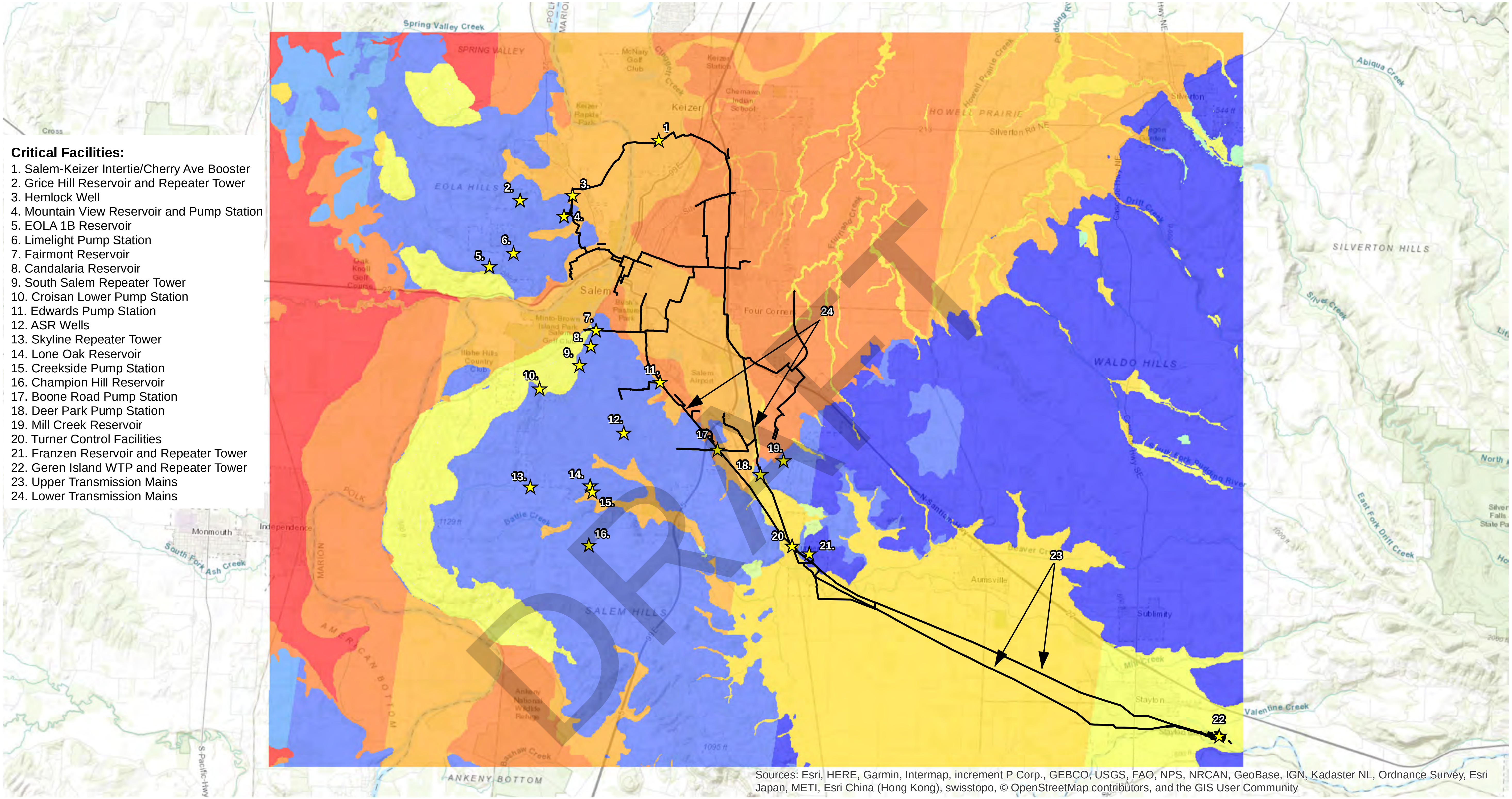
FIG. 9

Filename: T:\Projects\PDX\105000s\105679 - Salem Seismic\Avmxd\11x17\Fig 8 - SA1.mxd Date: 4/22/2021 Login: DSJ



Critical Facilities:
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LEGEND

Site PGV (cm/sec)

🟦	9 - 11	🟦	17 - 19	🟨	25 - 27
🟦	11 - 13	🟩	19 - 21	🟧	27 - 29
🟦	13 - 15	🟩	21 - 23	🟧	29 - 31
🟦	15 - 17	🟨	23 - 25	🟥	31 - 33

☆ Critical Facilities

— City of Salem Pipeline Backbone

0 6,000 12,000 24,000

Scale in Feet

NOTES
1. PGV map for the magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided in DOGAMI publications O-13-06 and OGDC-6, and methodology in Boore and Atkinson, 2008. See text for details.
2. City of Salem pipelines provided by Black & Veatch.

Salem Seismic
Salem, Oregon

PEAK GROUND VELOCITY, PGV

May 2021 105679

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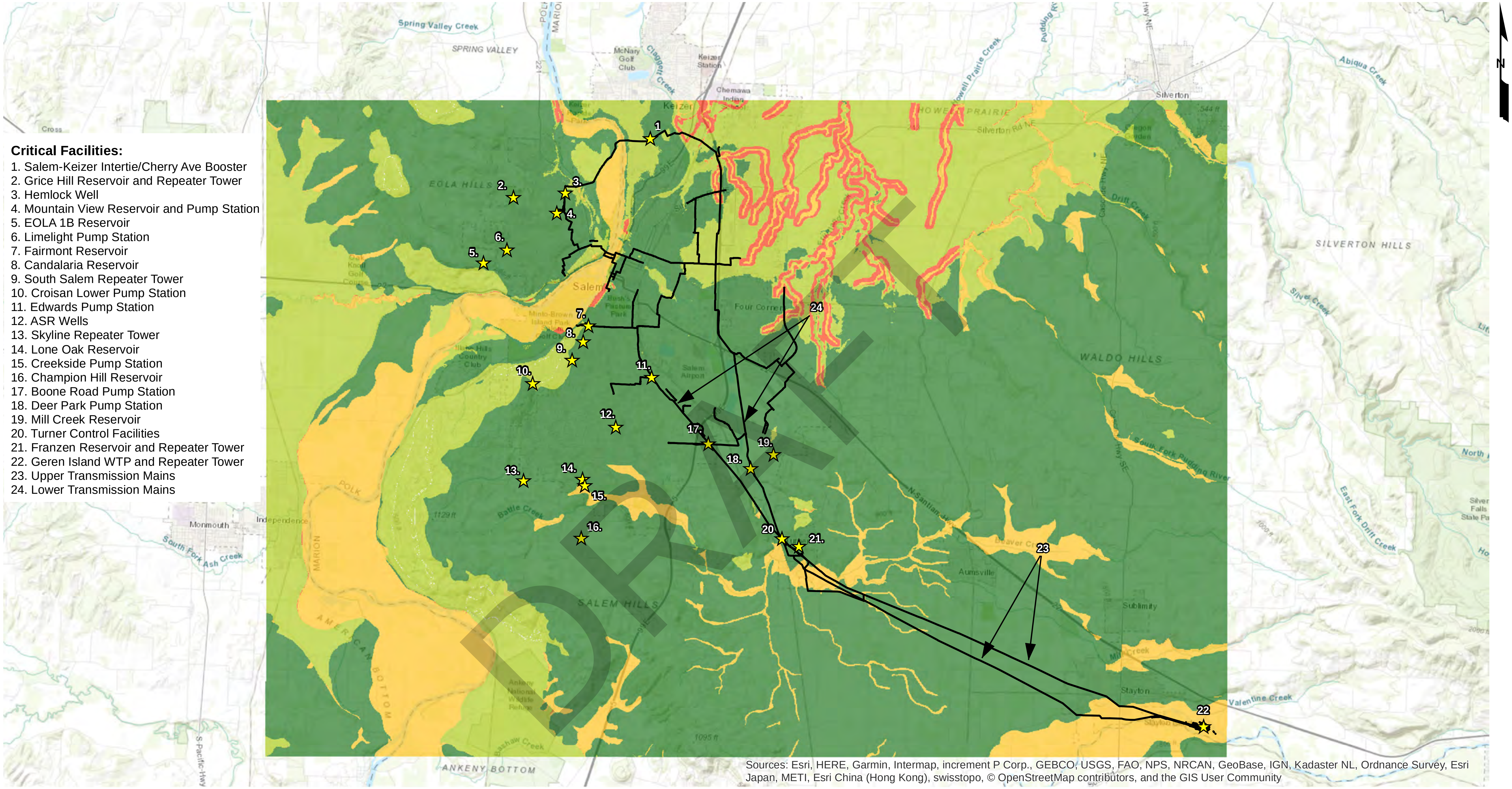
FIG. 10



Critical Facilities:

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LEGEND

Liquefaction Probability (%)

(Proportion of Area Expected to Liquefy)

🟩 0 - 1
🟩 1 - 5
🟨 5 - 10
🟥 10 - 16

☆ Critical Facilities

— City of Salem Pipeline Backbone

0 6,000 12,000 24,000

Scale in Feet

NOTES

1. Probability of liquefaction for magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided with DOGAMI publications O-12-02, GMS-105, and OGDC-6. See text for details.
2. City of Salem pipelines provided by Black & Veatch.

Salem Seismic
Salem, Oregon

PROBABILITY OF LIQUEFACTION

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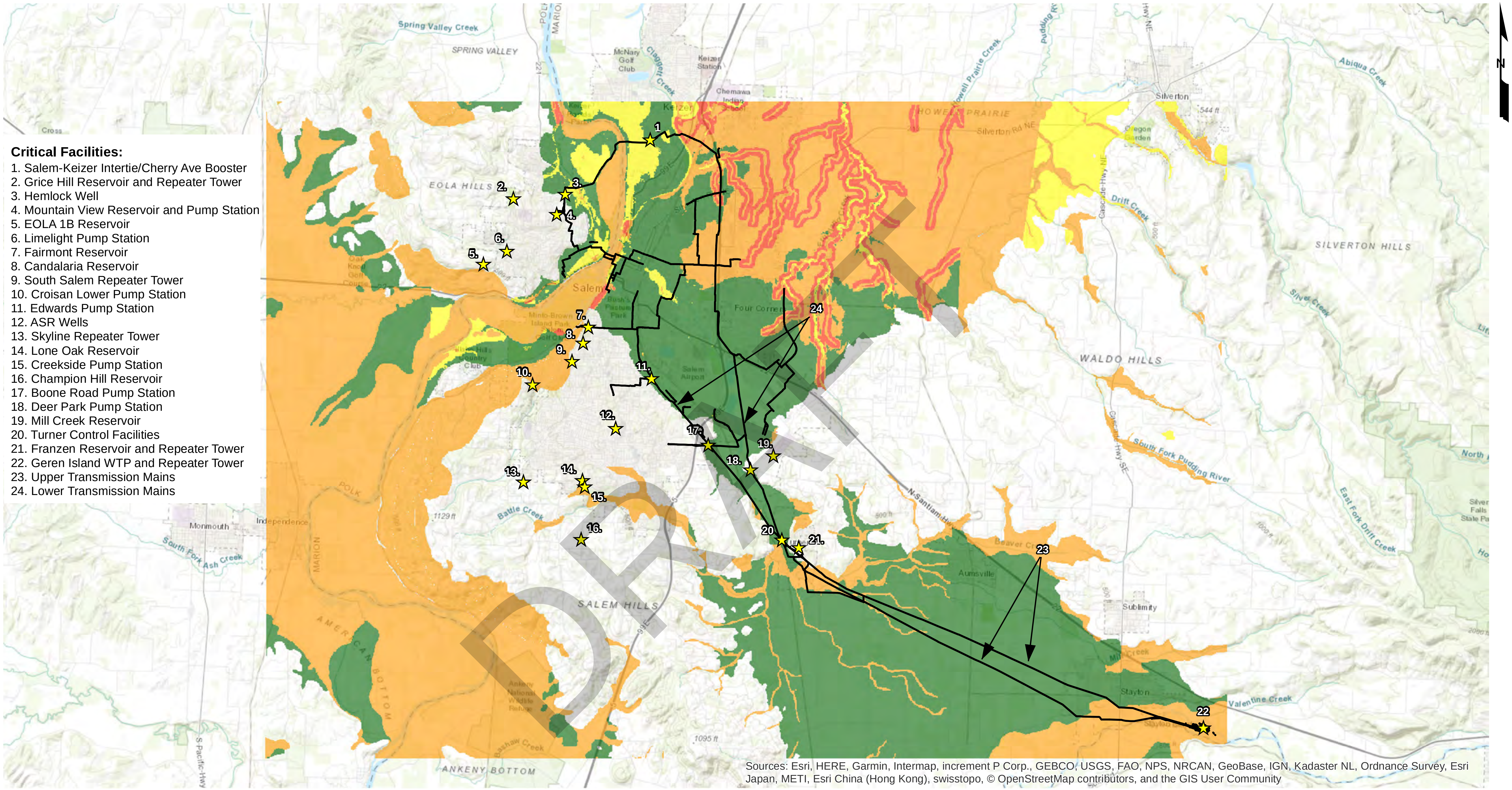
FIG. 11

Filename: T:\Projects\PDX\105000s\105679 - Salem Seismic\Avmxd\11x17\Fig 10 - Liquefaction Probability.mxd Date: 4/23/2021 Login: DSJ



Critical Facilities:

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23. Upper Transmission Mains
24. Lower Transmission Mains

Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

LEGEND

Liquefaction-Induced Lateral Spreading PGD (in)

🟩 0 - 0.1
🟩 0.1 - 2
🟨 2 - 6
🟧 6 - 12
🟥 12 - 24

★ Critical Facilities

— City of Salem Pipeline Backbone

0 6,000 12,000 24,000
Scale in Feet

NOTES

1. Liquefaction-induced lateral spreading PGD for the magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided with DOGAMI publications O-12-02, OGDC-6, GMS-105, and FEMA publication HAZUS 4.2 Technical Manual. See text for details.
2. City of Salem pipelines provided by Black & Veatch.

Salem Seismic
Salem, Oregon

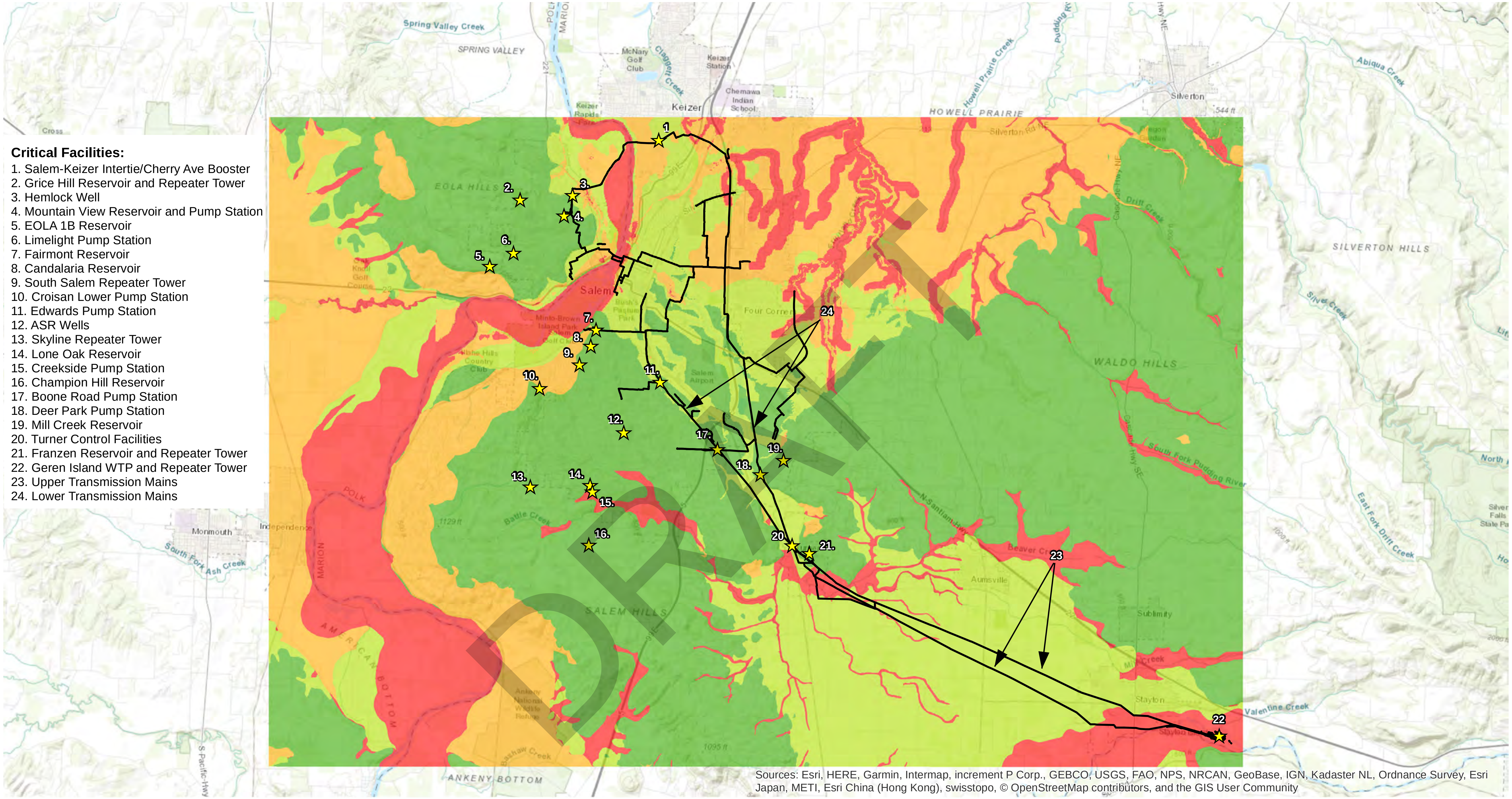
LIQUEFACTION-INDUCED LATERAL SPREADING PERMANENT GROUND DEFORMATION, PGD

May 2021 105679

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 12

Filename: T:\Projects\PDX\105000s\105679 - Salem Seismic\Avmxd\11x17\Fig 11 - Liquefaction Lateral Spreading.mxd Date: 4/23/2021 Login: DSJ



Critical Facilities:
1. Salem-Keizer Intertie/Cherry Ave Booster
2. Grice Hill Reservoir and Repeater Tower
3. Hemlock Well
4. Mountain View Reservoir and Pump Station
5. EOLA 1B Reservoir
6. Limelight Pump Station
7. Fairmont Reservoir
8. Candalaria Reservoir
9. South Salem Repeater Tower
10. Croisan Lower Pump Station
11. Edwards Pump Station
12. ASR Wells
13. Skyline Repeater Tower
14. Lone Oak Reservoir
15. Creekside Pump Station
16. Champion Hill Reservoir
17. Boone Road Pump Station
18. Deer Park Pump Station
19. Mill Creek Reservoir
20. Turner Control Facilities
21. Franzen Reservoir and Repeater Tower
22. Geren Island WTP and Repeater Tower
23. Upper Transmission Mains
24. Lower Transmission Mains

Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

LEGEND

☆ Critical Facilities

— City of Salem Pipeline Backbone

Liquefaction-Induced Settlement PGD (in)

🟩 0
🟨 1
🟧 2
🟥 6

0 6,000 12,000 24,000
Scale in Feet

NOTES
1. Liquefaction-induced lateral spreading PGD for the magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided with DOGAMI publications O-12-02, OGDC-6, GMS-105, and FEMA publication HAZUS 4.2 Technical Manual. See text for details.
2. City of Salem pipelines provided by Black & Veatch.

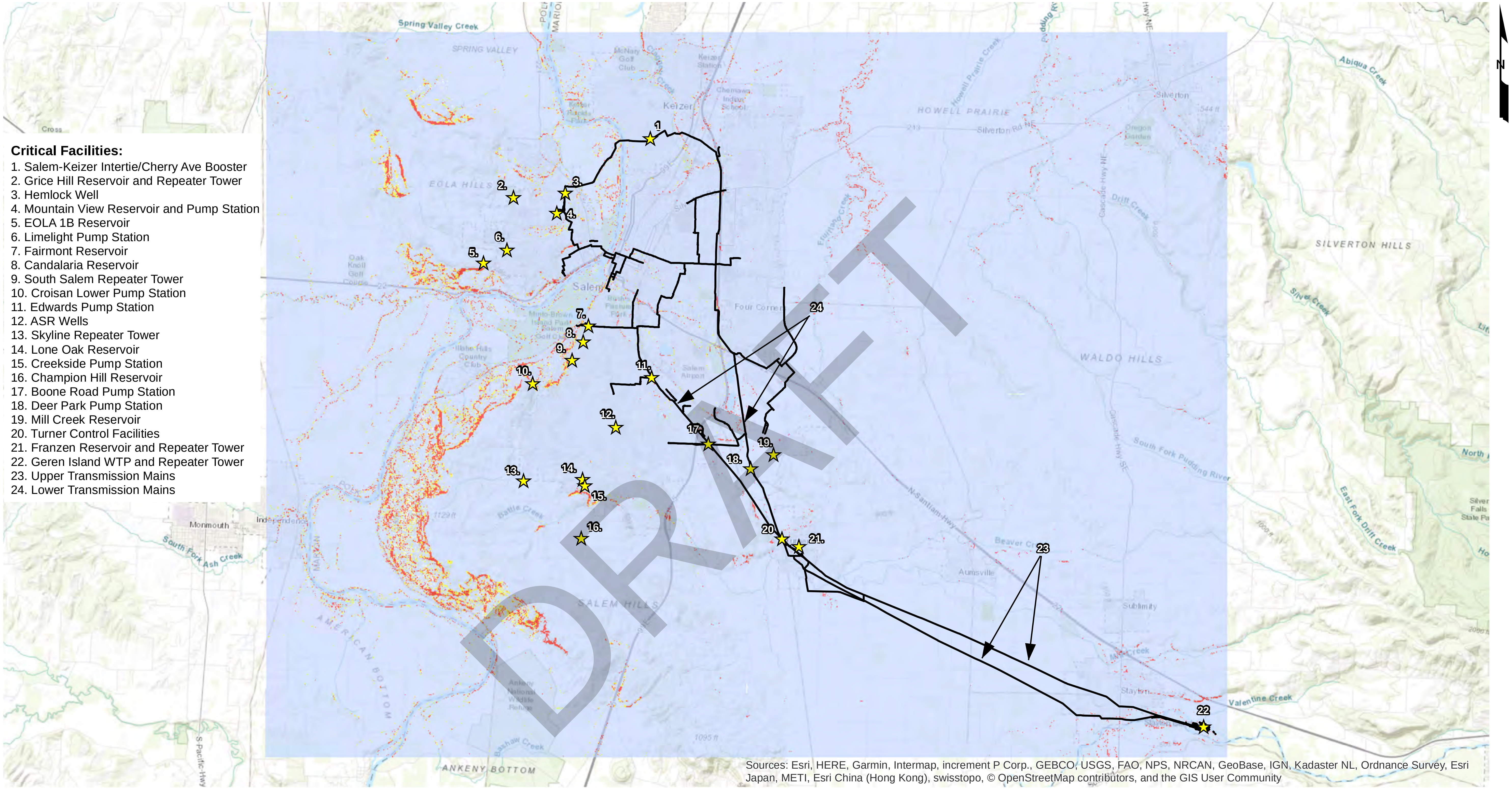
Salem Seismic
Salem, Oregon

LIQUEFACTION-INDUCED SETTLEMENT PERMANENT GROUND DEFORMATION, PGD

May 2021 105679

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 13



Critical Facilities:

1. Salem-Keizer Intertie/Cherry Ave Booster
2. Grice Hill Reservoir and Repeater Tower
3. Hemlock Well
4. Mountain View Reservoir and Pump Station
5. EOLA 1B Reservoir
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23. Upper Transmission Mains
24. Lower Transmission Mains

Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

LEGEND

Earthquake-Induced Landslide Probability in Dry Conditions (%)
(Proportion of Area Expected to Fail)

🟦 0
🟨 15
🟥 25

★ Critical Facilities

— City of Salem Pipeline Backbone

0 6,000 12,000 24,000
Scale in Feet

NOTES

1. Earthquake-induced landslide probability for the magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided with DOGAMI publications O-12-02, OGDC-6, SLIDO-4.0 and LiDAR. See text for details.
2. City of Salem pipelines provided by Black & Veatch.

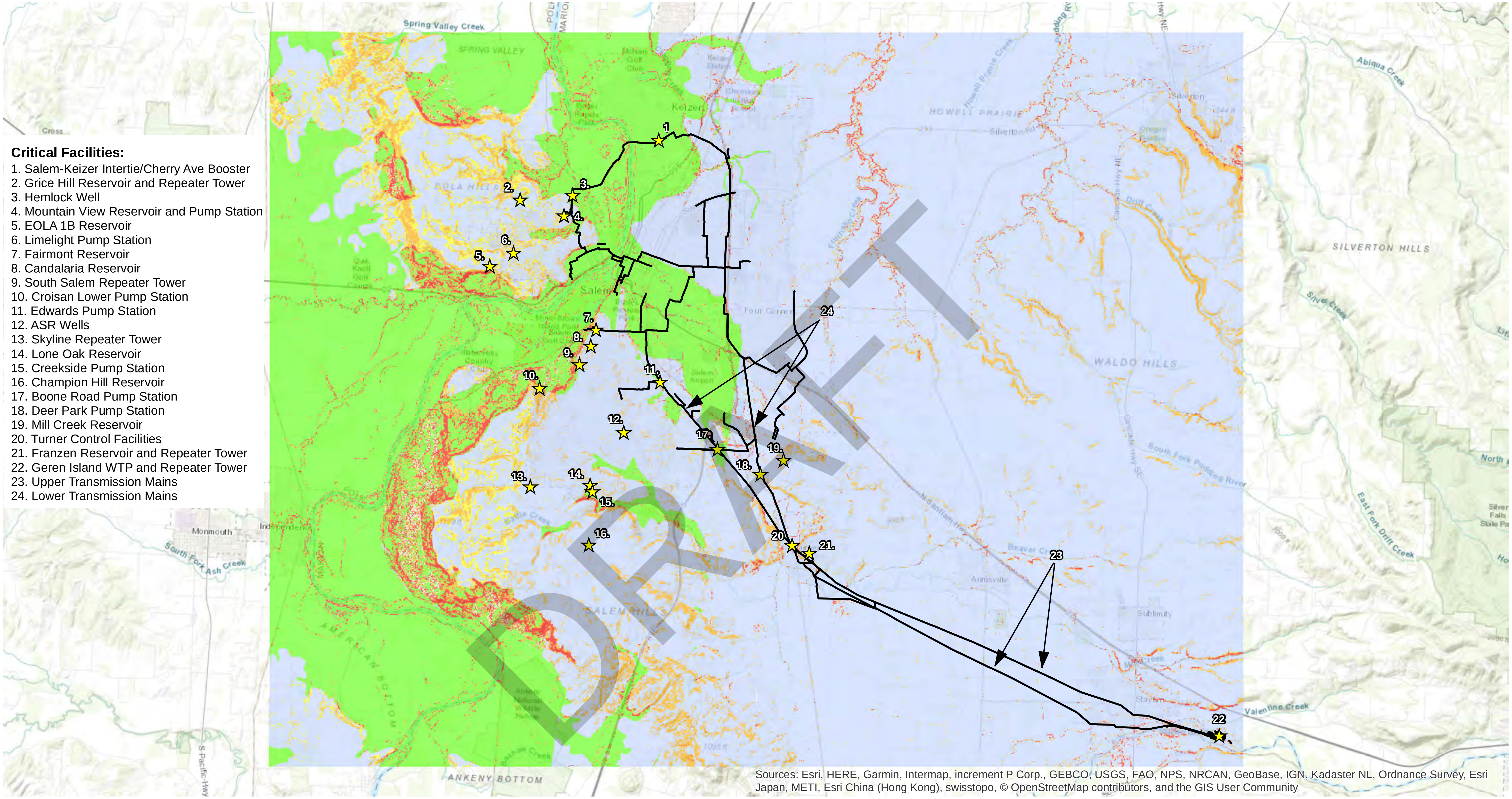
Salem Seismic
Salem, Oregon

PROBABILITY OF EARTHQUAKE-INDUCED LANDSLIDES (DRY)

May 2021
105679

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 14



Critical Facilities:

1. Salem-Keizer Intertie/Cherry Ave Booster
2. Grice Hill Reservoir and Repeater Tower
3. Hemlock Well
4. Mountain View Reservoir and Pump Station
5. EOLA 1B Reservoir
6. Limelight Pump Station
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23. Upper Transmission Mains
24. Lower Transmission Mains

Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

LEGEND

Earthquake-Induced Landslide Probability in Wet Conditions (%)
(Proportion of Area Expected to Fail)

🟦 0
🟩 15
🟨 20
🟧 25
🟥 30

★ Critical Facilities

— City of Salem Pipeline Backbone

0 6,000 12,000 24,000
Scale in Feet

NOTES
1. Earthquake-induced landslide probability for the magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided with DOGAMI publications O-12-02, OGDC-6, SLIDO-4.0 and LiDAR. See text for details.
2. City of Salem pipelines provided by Black & Veatch.

Salem Seismic
Salem, Oregon

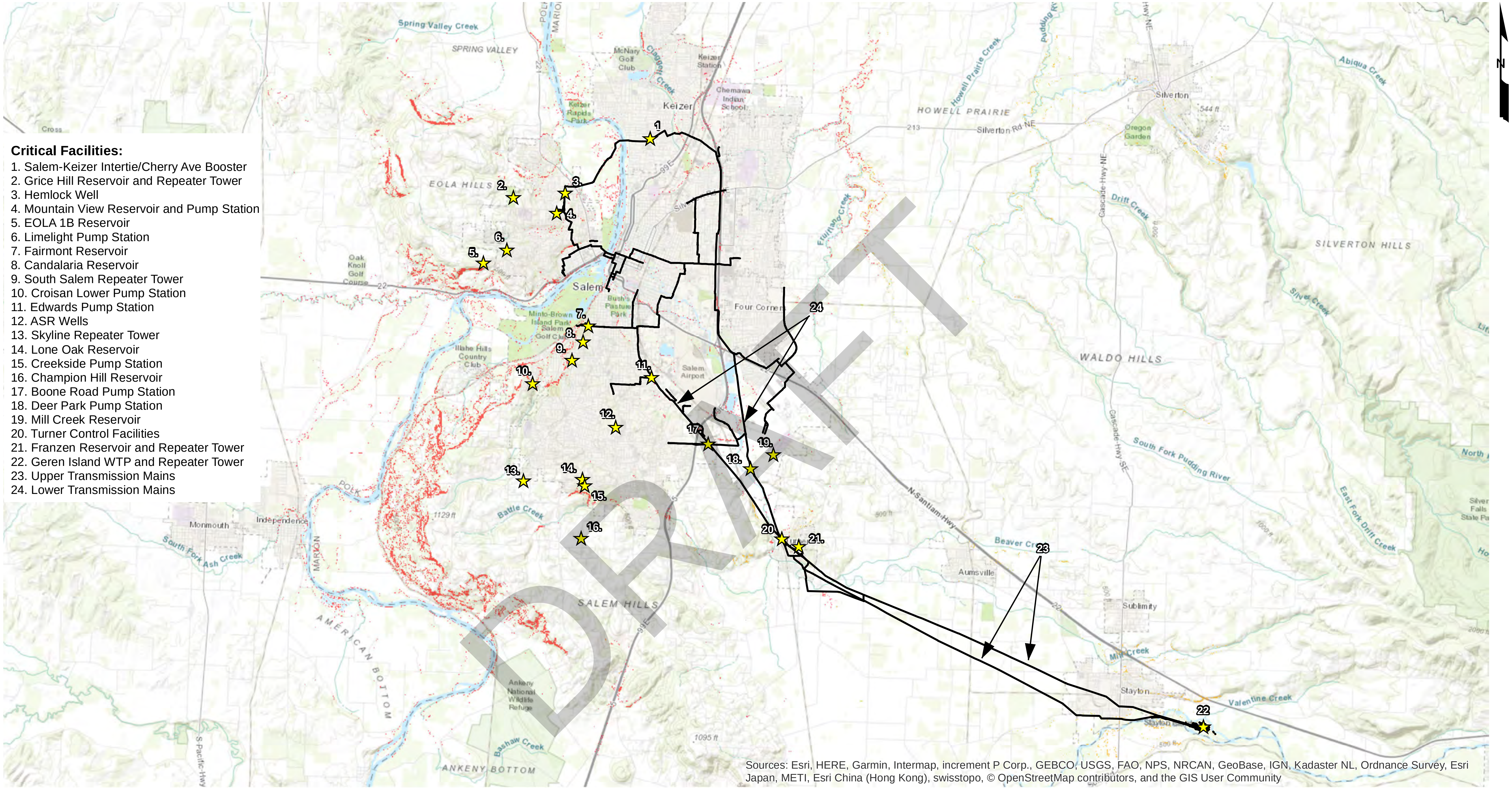
PROBABILITY OF EARTHQUAKE-INDUCED LANDSLIDES (WET)

May 2021 105679

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 15

Filename: T:\Projects\PDX\105000s\105679 - Salem Seismic\Avmxd\11x17\Fig 14 - Landslide Probability (Wet).mxd Date: 4/22/2021 Login: DSJ



Critical Facilities:
1. Salem-Keizer Intertie/Cherry Ave Booster
2. Grice Hill Reservoir and Repeater Tower
3. Hemlock Well
4. Mountain View Reservoir and Pump Station
5. EOLA 1B Reservoir
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24. Lower Transmission Mains

Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

LEGEND

Earthquake-Induced Landslide PGD (ft)

☐ Negligible
🟧 0 - 0.5
🟥 0.5 - 1

★ Critical Facilities

— City of Salem Pipeline Backbone

0 6,000 12,000 24,000
Scale in Feet

NOTES
1. Earthquake-induced landslide PGD for the magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided with DOGAMI publications O-12-02, OGDC-6, SLIDO-4.0 and LiDAR. See text for details.
2. City of Salem pipelines provided by Black & Veatch.

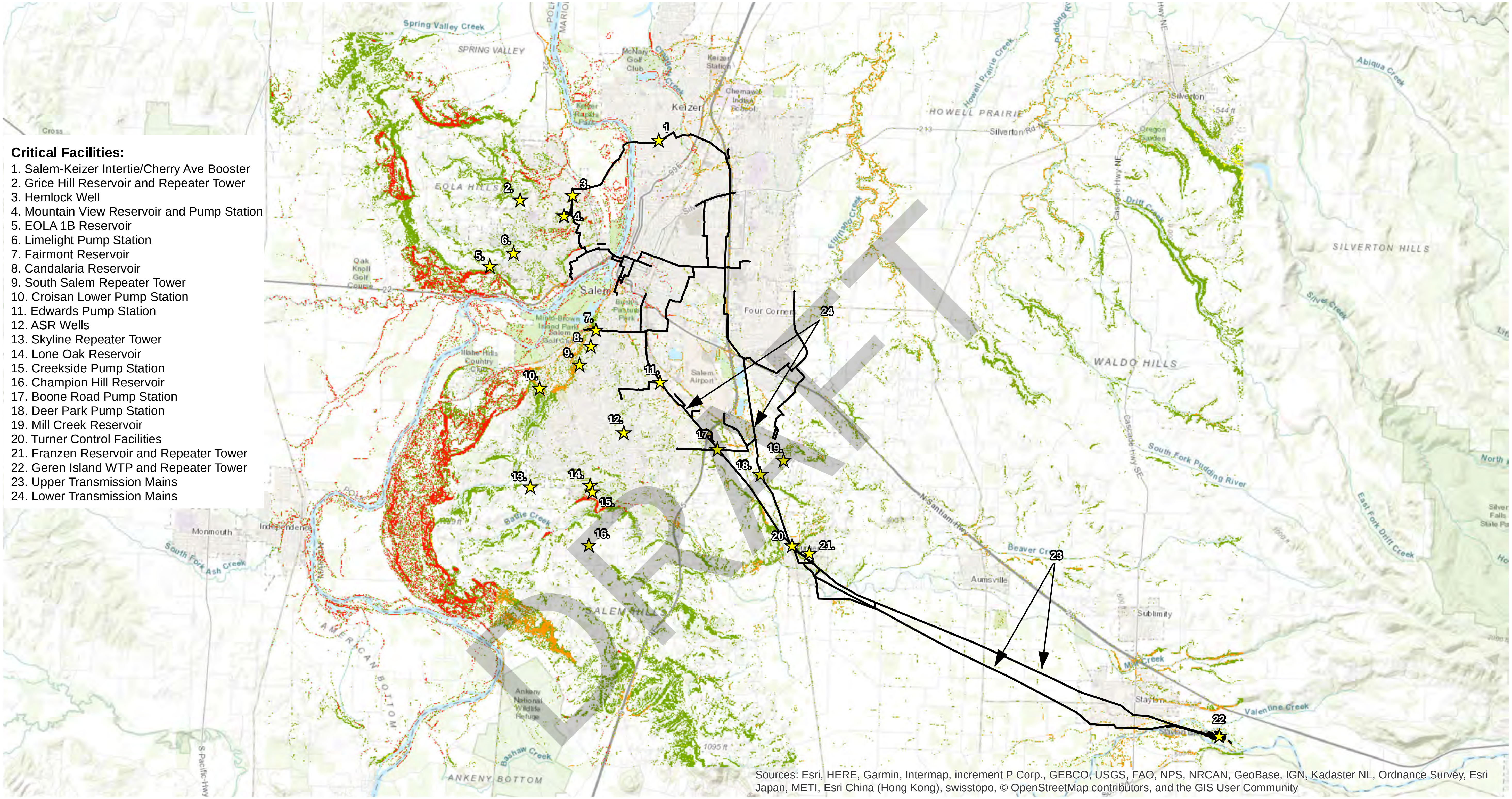
Salem Seismic
Salem, Oregon

EARTHQUAKE-INDUCED LANDSLIDE PERMANENT GROUND DEFORMATION, PGD (DRY)

May 2021   105679

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 16



Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, © OpenStreetMap contributors, and the GIS User Community

Critical Facilities:

1. Salem-Keizer Intertie/Cherry Ave Booster
2. Grice Hill Reservoir and Repeater Tower
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19. Mill Creek Reservoir
20. Turner Control Facilities
21. Franzen Reservoir and Repeater Tower
22. Geren Island WTP and Repeater Tower
23. Upper Transmission Mains
24. Lower Transmission Mains

LEGEND

Earthquake-Induced Landslide PGD (ft)

☐ Negligible
🟩 0-1
🟨 1-2
🟧 2-3
🟥 3-4

★ Critical Facilities

— City of Salem Pipeline Backbone

0 6,000 12,000 24,000

Scale in Feet

NOTES

1. Earthquake-induced landslide PGD for the magnitude 9.0 Cascadia Earthquake Scenario calculated from data provided with DOGAMI publications O-12-02, OGDC-6, SLIDO-4.0 and LiDAR. See text for details.
2. City of Salem pipelines provided by Black & Veatch.

Salem Seismic
Salem, Oregon

EARTHQUAKE-INDUCED LANDSLIDE PERMANENT GROUND DEFORMATION, PGD (WET)

May 2021 105679

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 17

Filename: T:\Projects\PDX\105000s\105679 - Salem Seismic\Avmxd\11x17\Fig 16 - Landslide PGD (Wet).mxd Date: 4/22/2021 Login: DSJ

LEGEND

🟥 Approximate Location of Shannon & Wilson Test Pit, 1987

⊕ Approximate Location of Applied Geotechnology Borehole, 1993

⬛ Approximate Location of Foundation Engineering Test Pit, 1996

⊕ Approximate Location of Foundation Engineering Borehole, 1996

⊕ Approximate Location of McMillen Jacobs Borehole, 2019

0 250 500 1,000

Scale in Feet

Salem Seismic,
Salem, Oregon

PREVIOUS EXPLORATIONS ON GEREN ISLAND

May 2021 105679

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 18

Filename: T:\Projects\PDX\105000s\105679 - Salem Seismic\Avmxd\11x17\Fig 18 - Previous Explorations.mxd Date: 4/23/2021 Login: KJW

APPENDIX A

EXISTING INFORMATION
SITE 1 - SALEIM-KEIZER INTERTIE
&
CHERRY AVE BOOSTER PUMP STATION

WELL REPORT
STATE OF OREGON

MAR1......
16771 MAR
16771

RECEIVED
JAN 29 1982
WATER RESOURCES DEPT
SALEM, OREGON

State Well No. 7s|3w-2
State Permit No.

ER:
Name Keizer Water Dist.
Address 64 Chemawa Rd NE
City Salem State Ore

(2) TYPE OF WORK (check):
New Well ☑ Deepening ☐ Reconditioning ☐ Abandon ☐
If abandonment, describe material and procedure in Item 12.

(3) TYPE OF WELL:
Rotary Air ☐ Driven ☐
Rotary Mud ☐ Dug ☐
C ☑ Bored ☐

(4) PROPOSED USE (check):
Domestic ☐ Industrial ☐ Municipal ☑
Irrigation ☐ Test Well ☐ Other ☐
Thermal: Withdrawal ☐ Reinjection ☐

(5) CASING INSTALLED: Steel ☑ Plastic ☐ Threaded ☐ Welded ☑
12" Diam. from +1 ft. to 120 ft. Gauge .250
" Diam. from ft. to ft. Gauge

LINER INSTALLED:
" Diam. from ft. to ft. Gauge

(6) PERFORATIONS: Perforated? ☐ Yes ☑ No
Type of perforator used
Size of perforations in. by in.
perforations from ft. to ft.
perforations from ft. to ft.
perforations from ft. to ft.

(7) SCREENS: Well screen installed? ☑ Yes ☐ No
Manufacturer's Name Johnson
Type 12" – 60 slot 120'-140' Model No. 304
Diam. 10" Slot Size 30 Set from 170 ft. to 188 ft.
Diam. 10" Slot Size 25 Set from 188 ft. to 205 ft.

(8) WELL TESTS: Drawdown is amount water level is lowered below static level
a pump test made? ☑ Yes ☐ No If yes, by whom?
d: 600 gal./min. with 90 ft. drawdown after 24 hrs.
" " " "
Air test gal./min. with drill stem at ft. hrs.
Bailer test gal./min. with ft. drawdown after hrs.
esian flow g.p.m.
perature of water Depth artesian flow encountered ft.

(9) CONSTRUCTION: Special standards: Yes ☐ No ☑
Well seal—Material used cement
Well sealed from land surface to 80 ft.
Diameter of well bore to bottom of seal 16 in.
Diameter of well bore below seal 12 in.
Number of sacks of cement used in well seal 104 sacks
How was cement grout placed? Pumped.
Was pump installed? no Type HP Depth ft.
Was a drive shoe used? ☑ Yes ☐ No Plugs Size: location ft.
Did any strata contain unusable water? ☐ Yes ☐ No
Type of Water? depth of strata
Method of sealing strata off
Was well gravel packed? ☐ Yes ☑ No Size of gravel:
Gravel placed from ft. to ft.

(10) LOCATION OF WELL:
County Marion Driller's well number
¼ ¼ Section 2 T. 7 R. 3w W.M.
Tax Lot # Lot Blk Subdivision
Address at well location: Cherry Ave

(11) WATER LEVEL: Completed well.
Depth at which water was first found 22 ft.
Static level 25 ft. below land surface. Date 1-22-82
Artesian pressure lbs. per square inch. Date

(12) WELL LOG: Diameter of well below casing 12
Depth drilled 232 ft. Depth of completed well 210 ft.
Formation: Describe color, texture, grain size and structure of materials; and show thickness and nature of each stratum and aquifer penetrated, with at least one entry for each change of formation. Report each change in position of Static Water Level and indicate principal water-bearing strata.

MATERIAL	From	To	SWL
Sandy clay	0	22	
Small to Large Gravel	22	35	
Sand Small to Large Gravel	35	43	
Small to Med Gravel	43	66	
Grey clay	66	75	
Cemented Gravel	75	95	
Small to Large Gravel	95	115	
Cemented Gravel	115	117	
Small to Large Gravel & Sand	117	140	
Cemented Gravel	140	165	
Sand & Gravel	165	167	
Clay & Gravel	167	168	
Small to Large Gravel	168	189	
Sand Some Small	189	208	
Gravel mostly Large	~~[illegible]~~		
Cemented Gravel	208	212	
Red Cinder	212	232	
Well Back filled from		232 –	
210 thru Red Cinder		Zone.	

Work started 10-16 1982 Completed 1-22 1982
Date well drilling machine moved off of well 1-22 1982

Drilling Machine Operator's Certification:

This well was constructed under my direct supervision. Materials used and information reported above are true to my best knowledge and belief.

[Signed] Norman H Ferrell Sr. (Drilling Machine Operator) Date 1-27, 1982
Drilling Machine Operator's License No. 455.

Water Well Contractor's Certification:

This well was drilled under my jurisdiction and this report is true to the best of my knowledge and belief.

Name EOLA WELL DRILLING (Person, firm or corporation) (Type or print)
4510 DALLAS RD. N.W.
Address SALEM, OR 97304
[Signed] Garet H Berndt (Water Well Contractor)
Contractor's License No. 619 Date 1-27, 1982

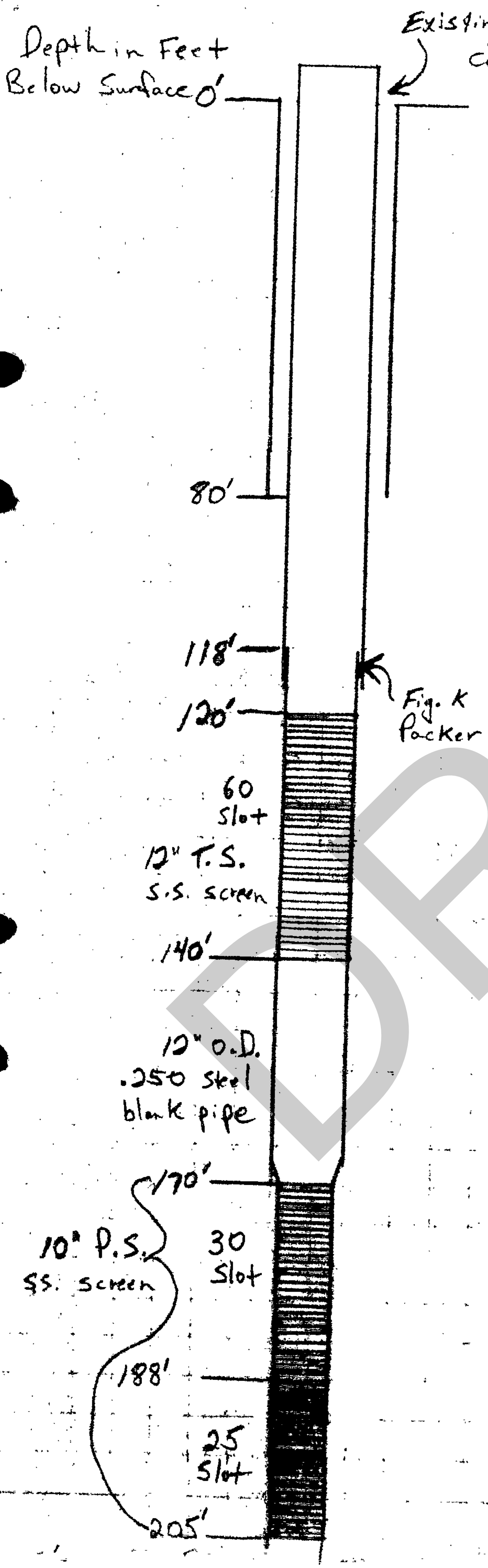
NOTICE TO WATER WELL CONTRACTOR
The original and first copy of this report
are to be filed with the

WATER RESOURCES DEPARTMENT,
SALEM, OREGON 97310
within 30 days from the date of well completion.

SP*12658-690

7S/3W-2
Marion

RECEIVED
JAN 29 1982
WATER RESOURCES DEPT
SALEM, OREGON



Keizer Water District
Cherry Ave. Production Wel

Vertical Well Profile
12-15-81 - N.T.S.
E. Butts

OREGON HEALTH DIVISION ONLY:

Received Date: 9/18/00

County Well Log ID # MARI 16771

WELL IDENTIFICATION LABEL ATTACHMENT FORM
(OREGON HEALTH DIVISION)

COMPANY /CURRENT WELL OWNER:

OWNER (S) WELL NO: #5

Name: City of Keizer

Mailing Address: P.O. Box 21000

City: Keizer State: OR Zip: 97307 Phone: (503) 390-3700

CONTACT PERSON:

NAME: Joe Edgell PHONE # (503) 390-3700

THIS FORM IS ONLY TO BE USED FOR WELLS WITH
POSITIVELY IDENTIFIED
WATER SUPPLY WELL REPORTS.

O.H.D. OFFICIAL USE ONLY

TOWNSHIP: 7 N [✓ S] RANGE: 3 E [✓ W] SECTION: 2 TAX-LOT: 9600

Well Identification Label : L-32102

LABEL ATTACHED BY: Tom Pattee DATE : 8/18/00
(O.H.D. OFFICIAL)

(WATER SUPPLY WELL REPORT MUST BE ATTACHED!)

Please Return Completed Form to:

Larry D. McQueen
Well Identification Program
Oregon Water Resources Department
158 12th Street NE
Salem OR 97310

LDM/WRD/OHD

APPENDIX B

EXISTING INFORMATION
SITE 2 - GRICE HILL RESERVOIR
&
REPEATER TOWER

RECEIVED
APR 5 1990
WATER RESOURCES DEPT.
SALEM, OREGON

POLK 002 7S/3W/17cb

STATE OF OREGON

WATER WELL REPORT

(as required by ORS 537.765)

(START CARD) # 19059

(1) OWNER:

Well Number: M8[illegible]

Name James Hellyer
Address 1900 27th Place N.W.
City Salem, Oregon 97304 State Zip

(2) TYPE OF WORK:

☒ New Well ☐ Deepen ☐ Recondition ☐ Abandon

(3) DRILL METHOD

☒ Rotary Air ☐ Rotary Mud ☐ Cable
☐ Other

(4) PROPOSED USE:

☒ Domestic ☐ Community ☐ Industrial ☐ Irrigation
☐ Thermal ☐ Injection ☐ Other

(5) BORE HOLE CONSTRUCTION:

Special Construction approval Yes ☐ No ☒ Depth of Completed Well 220 ft.
Explosives used Yes ☐ No ☒ Type Amount

HOLE			SEAL			Amount
Diameter	From	To	Material	From	To	sacks or pounds
10	0	120	Cement & 5%	0	120	35+bentonite
6	0	220				

How was seal placed: Method ☐ A ☐ B ☒ C ☐ D ☐ E
☐ Other
Backfill placed from ft. to ft. Material
Gravel placed from ft. to ft. Size of gravel

(6) CASING/LINER:

	Diameter	From	To	Gauge	Steel	Plastic	Welded	Threaded
Casing:	6	+1[illegible]	120	.25r0	☒	☐	☒	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
Liner:	4	0	220	160PSI	☐	☒	☐	☐
					☐	☐	☐	☐

Final location of shoe(s) 120

(7) PERFORATIONS/SCREENS:

☒ Perforations Method Skilsaw
☐ Screens Type Material

From	To	Slot size	Number	Diameter	Tele/pipe size	Casing	Liner
160	220	1/8" X 8"	135			☐	☒
						☐	☐
						☐	☐
						☐	☐
						☐	☐
						☐	☐

(8) WELL TESTS: Minimum testing time is 1 hour

☐ Pump ☐ Bailer ☒ Air ☐ Flowing Artesian

Yield gal/min	Drawdown	Drill stem at	Time
50		220	1 hr.

Temperature of water Depth Artesian Flow Found
Was a water analysis done? ☐ Yes By whom
Did any strata contain water not suitable for intended use? ☐ Too little
☐ Salty ☐ Muddy ☐ Odor ☐ Colored ☐ Other
Depth of strata:

(9) LOCATION OF WELL by legal description:

County Polk Latitude ' " Longitude ' "
Township 7S N or S, Range 3W E or W, WM.
Section 17 NW ¼ SW ¼
Tax Lot Lot Block Subdivision
Street Address of Well (or nearest address)
1900 27th Place N.W. Salem, OR

(10) STATIC WATER LEVEL:

55 ft. below land surface. Date 3/27/90
Artesian pressure — lb. per square inch. Date

(11) WATER BEARING ZONES:

Depth at which water was first found 84

From	To	Estimated Flow Rate	SWL
84	91	10	
121	220	50	55
107	112	8	

(12) WELL LOG:

Ground elevation

Material	From	To	SWL
Topsoil	0	2	
Brown Clay	2	41	
Brown Shale	41	46	
Brown Clay	46	55	
Broken Rock	55	73	
Black Basalt	73	77	
Broken Rock	77	84	
Broekn Rock W.B.	84	91	
Broken Rock	91	107	
Broken Rock W.B.	107	112	
Broken Rock	112	121	
Red Sandy Shale	121	129	
Brown Brown Broken Rock	129	149	
Light Gray Clay	149	156	
Broken Basalt W.B.	156	220	
Shale Traps placed on liner			
at 140' and 150'			

Date started 3/21/90 Completed 3/28/90

(unbonded) Water Well Constructor Certification:

I certify that the work I performed on the construction, alteration, or abandonment of this well is in compliance with Oregon well construction standards. Materials used and information reported above are true to my best knowledge and belief.

Signed Mark D Beir WWC Number 753
Date 3/28/90

(bonded) Water Well Constructor Certification:

I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. all work performed during this time is in compliance with Oregon well construction standards. This report is true to the best of my knowledge and belief. WILLAMETTE DRILLING CO. WWC Number =753

Signed Mark D Beir Date 3/28/90

RECEIVED
DEC 20 1999
WATER RESOURCES DEPT.
SALEM, OREGON

POLK
51079

STATE OF OREGON
WATER SUPPLY WELL REPORT
(as required by ORS 537.765)
Instructions for completing this report are on the last page of this form.

WELL I.D. # L 23074
START CARD # 115810

(1) OWNER: Well Number
Name MARK ROBINSON
Address 2246 27th place
City salem State ore Zip 97304

(2) TYPE OF WORK
☑ New Well ☐ Deepening ☐ Alteration (repair/recondition) ☐ Abandonment

(3) DRILL METHOD:
☑ Rotary Air ☐ Rotary Mud ☐ Cable ☐ Auger
☐ Other

(4) PROPOSED USE:
☑ Domestic ☐ Community ☐ Industrial ☐ Irrigation
☐ Thermal ☐ Injection ☐ Livestock ☑ Other

(5) BORE HOLE CONSTRUCTION:
Special Construction approval ☐ Yes ☑ No Depth of Completed Well 204 ft.
Explosives used ☐ Yes ☑ No Type Amount

HOLE			SEAL			
Diameter	From	To	Material	From	To	Sacks or pounds
10	0	95	cement	50	95	12 bags
6	95	204				
			bentonite	0	50	20 bags

How was seal placed: Method ☐ A ☐ B ☐ C ☑ D ☐ E
☑ Other filled to top w/ dry bentonite
Backfill placed from ft. to ft. Material
Gravel placed from 241 ft. to 260 ft. Size of gravel 3/4-

(6) CASING/LINER:

	Diameter	From	To	Gauge	Steel	Plastic	Welded	Threaded
Casing:	6	+1	95	250	☑	☐	☑	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
Liner:	4	7	204		☐	☑	☑	☐
				c40	☐	☐	☐	☐

Final location of shoe(s)

(7) PERFORATIONS/SCREENS:
☑ Perforations Method saw cut
☐ Screens Type Material

From	To	Slot size	Number	Diameter	Tele/pipe size	Casing	Liner
						☐	☐
120	203	1/8	55	6 long		☐	☐
						☐	☐
						☐	☐
						☐	☐

(8) WELL TESTS: Minimum testing time is 1 hour
☑ Pump ☐ Bailer ☑ Air ☐ Flowing Artesian

Yield gal/min	Drawdown	Drill stem at	Time
8gpm		260	1 hr.
20 gpm		195	pump

Temperature of water 56° Depth Artesian Flow Found
Was a water analysis done? ☐ Yes By whom
Did any strata contain water not suitable for intended use? ☐ Too little
☐ Salty ☐ Muddy ☐ Odor ☐ Colored ☐ Other
Depth of strata:

(9) LOCATION OF WELL by legal description:
County POLK Latitude Longitude
Township 7 S N or S Range 3 W E or W. WM.
Section 17 SW 1/4 ne 1/4
Tax Lot 1600 Lot Block Subdivision
Street Address of Well (or nearest address) same

(10) STATIC WATER LEVEL:
84' ft. below land surface. Date 11/26/99
Artesian pressure lb. per square inch. Date

(11) WATER BEARING ZONES:
Depth at which water was first found

From	To	Estimated Flow Rate	SWL
120	203	20 gpm	84

(12) WELL LOG:
Ground Elevation

Material	From	To	SWL
soil	0	1	
brown clay	1	12	
red clay	12	42	
tan clay	42	53	
brown clay	53	60	
rock brown broken Basalt	60	75	
rock black hard basalt	75	124	
rockblack brown broken	124	153	
tan brown clay w/rock	153	168	
rock black broken Basalt	168	185	
gray clay	185	187	
rock black broken Basalt	187	213	
rock black brown Basalt	213		
		260	
due to cavee broken			
rock,compleated depth			
of well is 204'			

Date started 11/14/99 Completed 11/26/99

(unbonded) Water Well Constructor Certification:
I certify that the work I performed on the construction, alteration, or abandonment of this well is in compliance with Oregon water supply well construction standards. Materials used and information reported above are true to the best of my knowledge and belief.

RECEIVED
FEB 03 2000
WATER RESOURCES DEPT.
SALEM, OREGON

WWC Number
Signed Date

(bonded) Water Well Constructor Certification:
I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. All work performed during this time is in compliance with Oregon water supply well construction standards. This report is true to the best of my knowledge and belief.
WWC Number 1585
Signed Date 11/26/99

ORIGINAL – WATER RESOURCES DEPARTMENT FIRST COPY – CONSTRUCTOR SECOND COPY – CUSTOMER

APPENDIX C

EXISTING INFORMATION
SITE 4 - MOUNTAIN VIEW RESERVOIR
&
PUMP STATION

RECEIVED

OCT 13 1999

WATER RESOURCES DEPT.
SALEM, OREGON

RECEIVED

NOV 19 1999

WATER RESOURCES DEPT.
SALEM, OREGON

STATE OF OREGON
WATER SUPPLY WELL REPORT
(as required by ORS 537.765)
Instructions for completing this report are on the last page of this form.

POLK
51034

WELL I.D. #L 34623
START CARD # 127263

(1) OWNER: Well Number 3423
Name Mike Kottek
Address 1657 Orchard Heights Rd NW
City Salem State OR Zip 97304

(2) TYPE OF WORK
☐ New Well ☐ Deepening ☒ Alteration (repair/recondition) ☐ Abandonment

(3) DRILL METHOD:
☐ Rotary Air ☐ Rotary Mud ☐ Cable ☐ Auger
☒ Other Crane Hoist

(4) PROPOSED USE:
☒ Domestic ☐ Community ☐ Industrial ☐ Irrigation
☐ Thermal ☐ Injection ☐ Livestock ☐ Other

(5) BORE HOLE CONSTRUCTION:
Special Construction approval ☐ Yes ☒ No Depth of Completed Well 180 ft.
Explosives used ☐ Yes ☒ No Type ______ Amount ______

HOLE			SEAL			
Diameter	From	To	Material	From	To	Sacks or pounds
6"	+1	180	Existing	Well		

How was seal placed: Method ☐ A ☐ B ☐ C ☐ D ☐ E
☒ Other Existing not disturbed
Backfill placed from ______ ft. to ______ ft. Material ______
Gravel placed from ______ ft. to ______ ft. Size of gravel ______

(6) CASING/LINER:

	Diameter	From	To	Gauge	Steel	Plastic	Welded	Threaded
Casing:	6"	Existing			☒	☐	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
Liner:	4"	0	180		☐	☒	☐	☐
Packer placed at			120		☐	☐	☐	☐

Final location of shoe(s) Unknown

(7) PERFORATIONS/SCREENS:
☒ Perforations Method Skilsaw
☐ Screens Type ______ Material ______

From	To	Slot size	Number	Diameter	Tele/pipe size	Casing	Liner
140	180	1/8x6	160			☐	☒
						☐	☐
						☐	☐
						☐	☐
						☐	☐

(8) WELL TESTS: Minimum testing time is 1 hour

☐ Pump ☐ Bailer ☐ Air ☐ Flowing Artesian

Yield gal/min	Drawdown	Drill stem at	Time
N/A			1 hr.

Temperature of water 53 Depth Artesian Flow Found ______
Was a water analysis done? No ☐ Yes By whom ______
Did any strata contain water not suitable for intended use? No ☐ Too little
☐ Salty ☐ Muddy ☐ Odor ☐ Colored ☐ Other ______
Depth of strata: ______

(9) LOCATION OF WELL by legal description:
County Polk Latitude ______ Longitude ______
Township 7S N or S Range 3W E or W. WM.
Section 16 NW 1/4 SW 1/4
Tax Lot 200 Lot ______ Block ______ Subdivision ______
Street Address of Well (or nearest address) Same as #1

(10) STATIC WATER LEVEL:
109 ft. below land surface. Date 10/5/99
Artesian pressure ______ lb. per square inch. Date ______

(11) WATER BEARING ZONES:

Depth at which water was first found Existing 109

From	To	Estimated Flow Rate	SWL
Existing			109

(12) WELL LOG:
Ground Elevation ______

Material	From	To	SWL
Because of exessive rust because			
of age of casing a PVC liner was			
placed to the bottom with			
a neoprene packer placed at			
120 ft.			
No log was found for			
original well			

Willamette Drilling Company

Date started 10/5/99 Completed 10/5/99

(unbonded) Water Well Constructor Certification:

I certify that the work I performed on the construction, alteration, or abandonment of this well is in compliance with Oregon water supply well construction standards. Materials used and information reported above are true to the best of my knowledge and belief.

WWC Number 1728
Signed Richard Massellis Date 10/11/99

(bonded) Water Well Constructor Certification:

I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. All work performed during this time is in compliance with Oregon water supply well construction standards. This report is true to the best of my knowledge and belief.

WWC Number 561
Signed Dallas L Davis Date 10/11/99

ORIGINAL – WATER RESOURCES DEPARTMENT FIRST COPY – CONSTRUCTOR SECOND COPY – CUSTOMER

POLK 53215

10-12-2011

STATE OF OREGON
GEOTECHNICAL HOLE REPORT
(as required by OAR 690-240-0035)

(1) OWNER/PROJECT

Hole Number B-2

PROJECT NAME/NBR: BRY 100611

First Name Last Name

Company LINDBECK FAMILY LLC

Address 2255 ELLIS AVE NE

City SALEM State OR Zip 97301

(2) TYPE OF WORK

☒ New ☐ Deepening ☒ Abandonment

☐ Alteration (repair/recondition)

(3) CONSTRUCTION

☐ Rotary Air ☐ Hand Auger ☒ Hollow stem auger

☐ Rotary Mud ☐ Cable ☐ Push Probe

☐ Other

(4) TYPE OF HOLE:

● Uncased Temporary ○ Cased Permanent

○ Uncased Permanent ○ Slope Stablity

○ Other

Other:

(5) USE OF HOLE

GEOTECHNICAL

(6) BORE HOLE CONSTRUCTION

Special Standard ☐ (Attach copy)

Depth of Completed Hole 20.00 ft.

BORE HOLE			SEAL				
Dia	From	To	Material	From	To	Amt	sacks/ lbs
8	0	20	Bentonite	0	20	10	S

Backfill placed from ft. to ft. Material

Filter pack from ft. to ft. Material Size

(7) CASING/SCREEN

Casing	Screen	Dia	+	From	To	Gauge	Stl	Plstc	Wld	Thrd
○	○		☐				○	○	☐	☐
○	○		☐				○	○	☐	☐
○	○		☐				○	○	☐	☐
○	○		☐				○	○	☐	☐
○	○		☐				○	○	☐	☐

(8) WELL TESTS

○ Pump ○ Bailer ○ Air ○ Flowing Artesian

Yield gal/min	Drawdown	Drill stem/Pump depth	Duration(hr)

Temperature °F Lab analysis ☐ Yes By

Supervising Geologist/Engineer

Water quality concerns? ☐ Yes (describe below)

From	To	Description	Amount	Units

(9) LOCATION OF HOLE (legal description)

County Polk Twp 7.00 S N/S Range 3.00 W E/W WM

Sec 16 SW 1/4 of the SW 1/4 Tax Lot 103

Tax Map Number Lot

Lat ° ' " or DMS or DD

Long ° ' " or DMS or DD

● Street address of hole ○ Nearest address

1500 ORCHARD HEIGHTS RD. NW SALEM, OREGON 97308

(10) STATIC WATER LEVEL

	Date	SWL(psi)	+	SWL(ft)
Existing Well / Predeepening			☐	
Completed Well			☐	

Flowing Artesian? ☐

WATER BEARING ZONES

Depth water was first found 16.00

SWL Date	From	To	Est Flow	SWL(psi)	+	SWL(ft)
					☐	
					☐	
					☐	
					☐	
					☐	

(11) SUBSURFACE LOG

Ground Elevation

Material	From	To
BROWNISH REDDISH CLAY	0	15
WEATHERED BASALT	15	20

Date Started 10-06-2011 Completed 10-06-2011

(12) ABANDONMENT LOG:

Material	From	To	Amt	sacks/ lbs
Bentonite	0	20	10	S

Date Started 10-06-2011 Completed 10-06-2011

Professional Certification (to be signed by an Oregon licensed water or monitoring well constructor,Oregon registered geologist or professional engineer).

I accept responsibility for the construction, deepening, alteration, or abandonment work performed during the construction dates reported above. All work performed during this time is in compliance with Oregon geotechnical hole construction standards. This report is true to the best of my knowledge and belief.

License/Registration Number 10626 Date

Electronically Submitted

First Name BRYAN Last Name MEAD

Affiliation SUBSURFACE TECHNOLOGIES

ORIGINAL - WATER RESOURCES DEPARTMENT

THIS REPORT MUST BE SUBMITTED TO THE WATER RESOURCES DEPARTMENT WITHIN 30 DAYS OF COMPLETION OF WORK

Form Version: 0.95

POLK 53216

10-12-2011

STATE OF OREGON
GEOTECHNICAL HOLE REPORT
(as required by OAR 690-240-0035)

(1) OWNER/PROJECT

Hole Number B-1

PROJECT NAME/NBR: BRY 100611

First Name Last Name

Company LINDBECK FAMILY LLC

Address 2255 ELLIS AVE NE

City SALEM State OR Zip 97301

(2) TYPE OF WORK

☒ New ☐ Deepening ☒ Abandonment

☐ Alteration (repair/recondition)

(3) CONSTRUCTION

☐ Rotary Air ☐ Hand Auger ☒ Hollow stem auger

☐ Rotary Mud ☐ Cable ☐ Push Probe

☐ Other

(4) TYPE OF HOLE:

● Uncased Temporary ○ Cased Permanent

○ Uncased Permanent ○ Slope Stablity

○ Other

Other:

(5) USE OF HOLE

GEOTECHNICAL

(6) BORE HOLE CONSTRUCTION

Special Standard ☐ (Attach copy)

Depth of Completed Hole 20.00 ft.

BORE HOLE			SEAL				
Dia	From	To	Material	From	To	Amt	sacks/ lbs
8	0	20	Bentonite	0	20	10	S

Backfill placed from ft. to ft. Material

Filter pack from ft. to ft. Material Size

(7) CASING/SCREEN

Casing	Screen	Dia	+	From	To	Gauge	Stl	Plstc	Wld	Thrd
○	○		☐				○	○	☐	☐
○	○		☐				○	○	☐	☐
○	○		☐				○	○	☐	☐
○	○		☐				○	○	☐	☐
○	○		☐				○	○	☐	☐

(8) WELL TESTS

○ Pump ○ Bailer ○ Air ○ Flowing Artesian

Yield gal/min	Drawdown	Drill stem/Pump depth	Duration(hr)

Temperature °F Lab analysis ☐ Yes By

Supervising Geologist/Engineer

Water quality concerns? ☐ Yes (describe below)

From	To	Description	Amount	Units

(9) LOCATION OF HOLE (legal description)

County Polk Twp 7.00 S N/S Range 3.00 W E/W WM

Sec 16 SW 1/4 of the SW 1/4 Tax Lot 103

Tax Map Number Lot

Lat ° ' " or DMS or DD

Long ° ' " or DMS or DD

● Street address of hole ○ Nearest address

1500 ORCHARD HEIGHTS RD. NW SALEM, OREGON 97308

(10) STATIC WATER LEVEL

	Date	SWL(psi)	+	SWL(ft)
Existing Well / Predeepening			☐	
Completed Well			☐	

Flowing Artesian? ☐

WATER BEARING ZONES

Depth water was first found 16.00

SWL Date	From	To	Est Flow	SWL(psi)	+	SWL(ft)
					☐	
					☐	
					☐	
					☐	
					☐	

(11) SUBSURFACE LOG

Ground Elevation

Material	From	To
BROWNISH REDDISH CLAY	0	15
WEATHERED BASALT	15	20

Date Started 10-06-2011 Completed 10-06-2011

(12) ABANDONMENT LOG:

Material	From	To	Amt	sacks/ lbs
Bentonite	0	20	10	S

Date Started 10-06-2011 Completed 10-06-2011

Professional Certification

(to be signed by an Oregon licensed water or monitoring well constructor,Oregon registered geologist or professional engineer).

I accept responsibility for the construction, deepening, alteration, or abandonment work performed during the construction dates reported above. All work performed during this time is in compliance with Oregon geotechnical hole construction standards. This report is true to the best of my knowledge and belief.

License/Registration Number 10626 Date

Electronically Submitted

First Name BRYAN Last Name MEAD

Affiliation SUBSURFACE TECHNOLOGIES

ORIGINAL - WATER RESOURCES DEPARTMENT

THIS REPORT MUST BE SUBMITTED TO THE WATER RESOURCES DEPARTMENT WITHIN 30 DAYS OF COMPLETION OF WORK

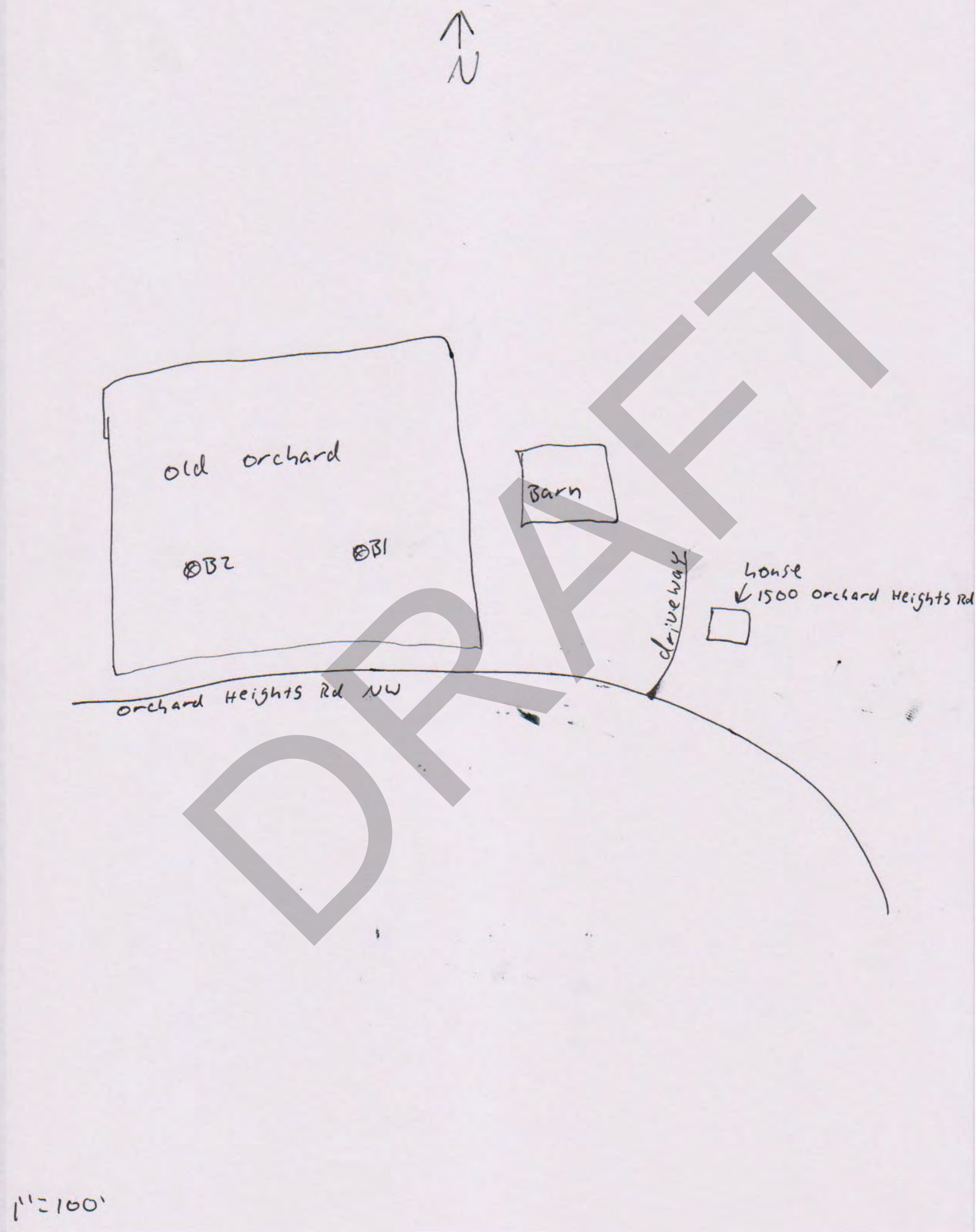
Form Version: 0.95

GEOTECHNICAL HOLE REPORT -
Map with location identified must be attached and shall include an approximate scale and north arrow

POLK 53216

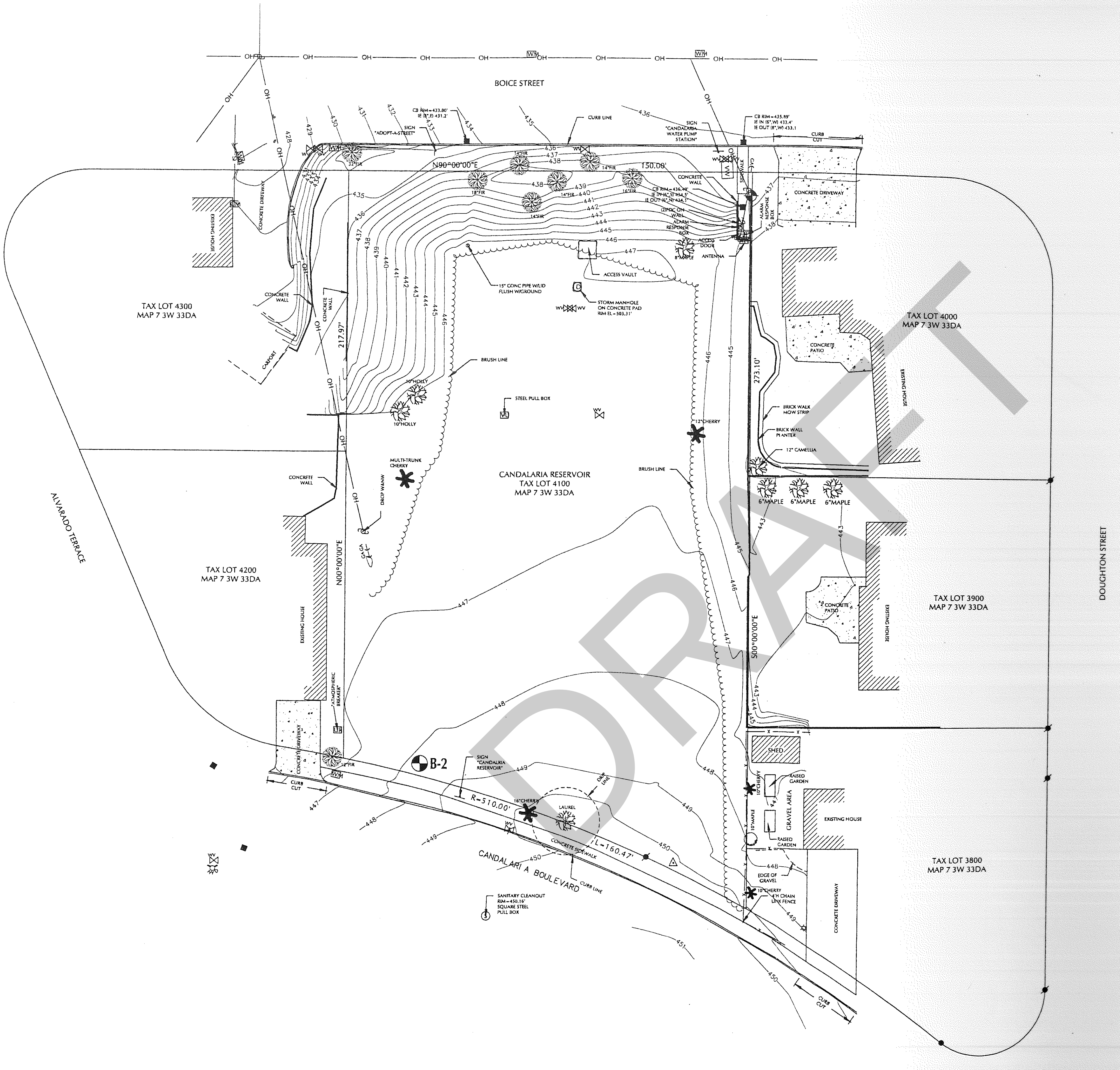
10-12-2011

Map of Hole



APPENDIX D

EXISTING INFORMATION
SITE 8 - CANDALARIA RESERVOIR



BORING MADE BY GRI
(JUNE 11, 2004)

SITE PLAN FROM FILE BY WESTLAKE CONSULTANTS, INC., DATED JUNE 17, 2004

North

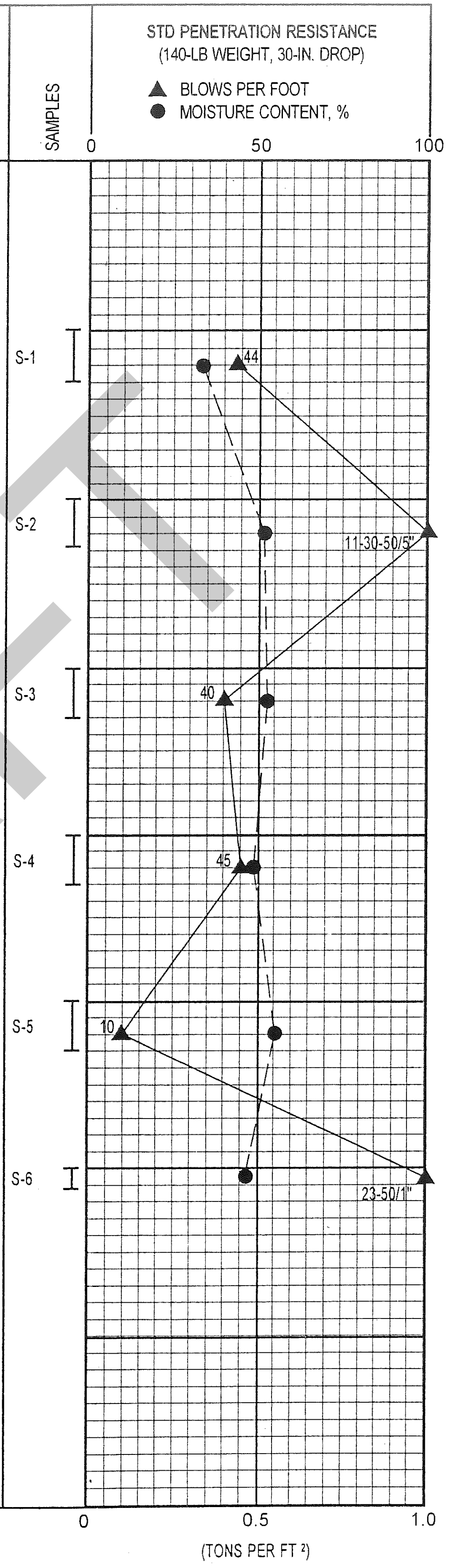
0 40 80 FT

GRI BLACK & VEATCH CORPORATION
CITY OF SALEM RESERVOIRS

SITE PLAN
(CANDALARIA RESERVOIR)

JULY 2004 JOB NO. 4091 FIG. 4

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION 448 ft (±)	DEPTH, FT
0		Hard, reddish-brown mottled black SILT; trace clay, scattered subangular fragments of severely weathered basalt, 2-in.-thick heavily rooted zone at the ground surface	
10		---------reddish yellow mottled red and black below 10 ft	
15		---------relict structure below 15 ft	
28.0	+ + + +	Soft (RH-1), gray BASALT; very close fractures, staining and mineralization on fracture surfaces, severely weathered	28.0
31.0		(6/11/2004)	31.0



I 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

■ UNDRAINED SHEAR STRENGTH, TSF

* NO RECOVERY

Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-2
(CANDALARIA RESERVOIR)

JULY 2004 JOB. NO. 4091 FIG. 2A

APPENDIX E

EXISTING INFORMATION
SITE 9 – SOUTH SALEM REPEATER TOWER

STATE OF OREGON
GEOTECHNICAL HOLE REPORT
(as required by OAR 690-240-035)

(1) OWNER/PROJECT: Hole Number **B-2**
Name **CITY OF SALEM**
Address **1580 - 20TH ST, SE #24**
City **SALEM** State **OR** Zip **97301**

(2) TYPE OF WORK
☑ New ☐ Deepening ☐ Alteration (repair/recondition) ☑ Abandonment

(3) CONSTRUCTION:
☐ Rotary Air ☐ Hand Auger ☑ Hollow Stem Auger
☐ Rotary Mud ☐ Cable Tool ☐ Push Probe ☐ Other ☐

(4) TYPE OF HOLE:
☑ Uncased Temporary ☐ Cased Permanent
☐ Uncased Permanent ☐ Slope Stability ☐ Other

(5) USE OF HOLE: **GEOTECHNICAL**

(6) BORE HOLE CONSTRUCTION:
Special Construction approval ☐ Yes ☑ No Depth of Completed Hole **45** ft.

HOLE			SEAL			
Diameter	From	To	Material	From	To	Sacks or pounds
8	**0**	**45**	**BENT CHIPS**	**45**	**0**	**23 SKS**

Backfill placed from ______ ft. to ______ ft. Material ______
Filter Pack placed from ______ ft. to ______ ft. Size of pack ______

(7) CASING/SCREEN:

	Diameter	From	To	Gauge	Steel	Plastic	Welded	Threaded
Casing:	**N/A**				☐	☐	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
Screen:					☐	☐	☐	☐
					☐	☐	☐	☐

Slot size ______

(8) WELL TEST:
☐ Pump ☐ Bailer ☐ Air ☐ Flowing Artesian
Permeability ______ Yield ______ GPM ______
Conductivity ______ PH ______
Temperature of water **N/A** °F Depth artesian flow found ______ ft.
Was water analysis done? ☐ Yes ☑ No
By whom? ______
Depth of strata analyzed. From ______ ft.
Remarks: ______

(9) LOCATION OF HOLE by legal description:
County **MARION** Latitude ______ Longitude ______
Township **8** **S** Range **3** **W** WM.
Section **4** **NW** 1/4 **NE** 1/4
Tax Lot **7900** Lot ______ Block ______ Subdivision ______
Street Address of Well (or nearest address) **955 DOWNS ST S**
SALEM, OR

Map with location indentified must be attached

(10) STATIC WATER LEVEL:
N/A ft. below land surface. Date **06/27/2006**
Artesian pressure ______ lb. per square inch. Date ______

(11) SUBSURFACE LOG:
Ground Elevation ______

Material Description	From	To	SWL
REDDISH WEATHERED BASALT	**0**	**45**	

Date Started **06/27/2006** Date Completed **06/27/2006**

(12) ABANDONMENT LOG:

Material Description	From	To	Sacks or Pounds
BENT CHIPS	**45**	**0**	**23 SKS**

Date started **06/27/2006** Date Completed **06/27/2006**

Professional Certification
(to be signed by a licensed water supply or monitoring well constructor, or registered geologist or civil engineer).

I accept responsibility for the construction, alteration, or abandonment work performed on during the construction dates reported above. All work performed during this time is in compliance with Oregon geotechnical hole construction standards. This report is true to the best of my knowledge and belief.

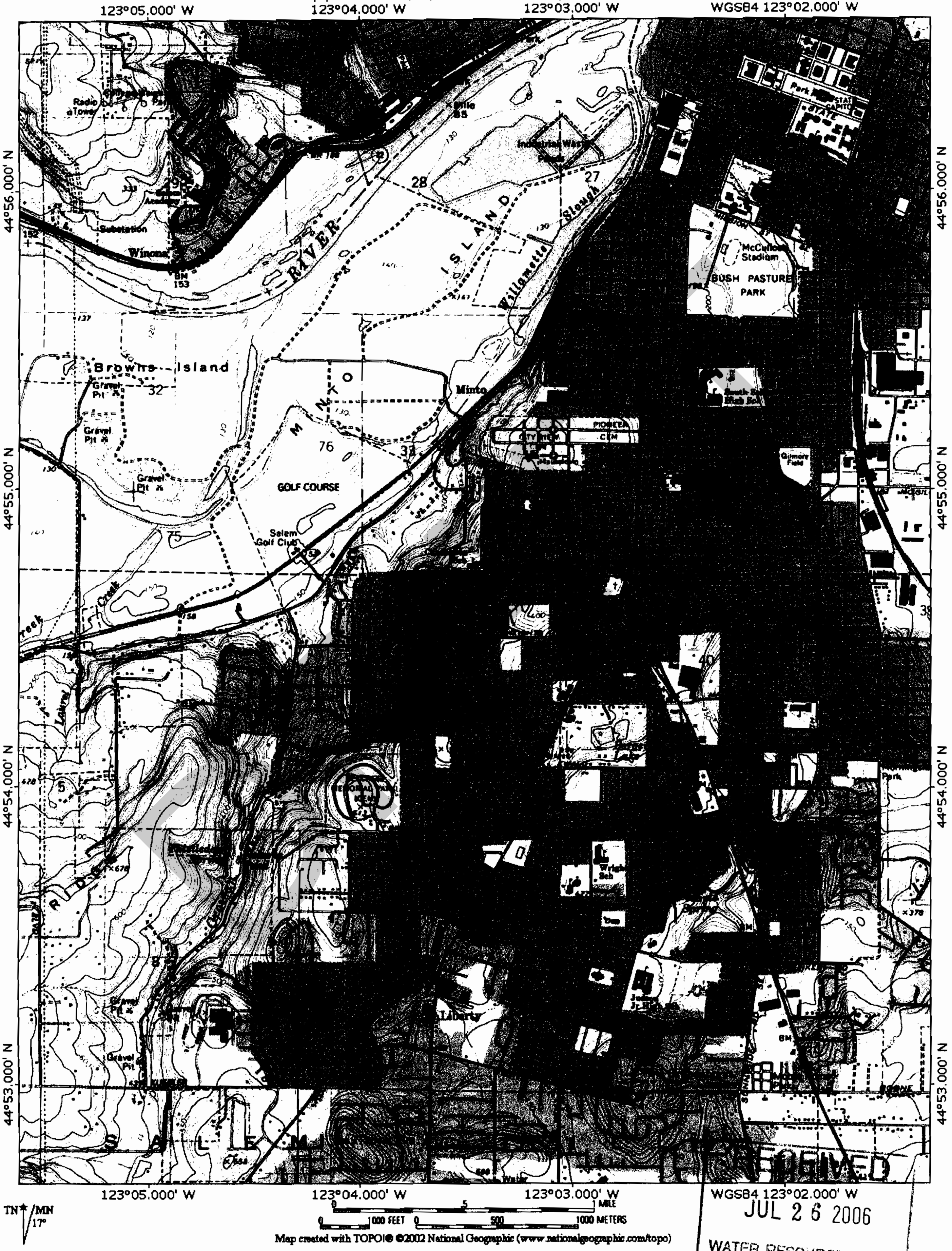
License or Registration Number **10536**
Signed Burt Marshall Date 7-21-06
BURTON MARSHALL
Affiliation **SUBSURFACE TECHNOLOGIES**

RECEIVED
JUL 26 2006
WATER RESOURCES DEPT
SALEM, OREGON

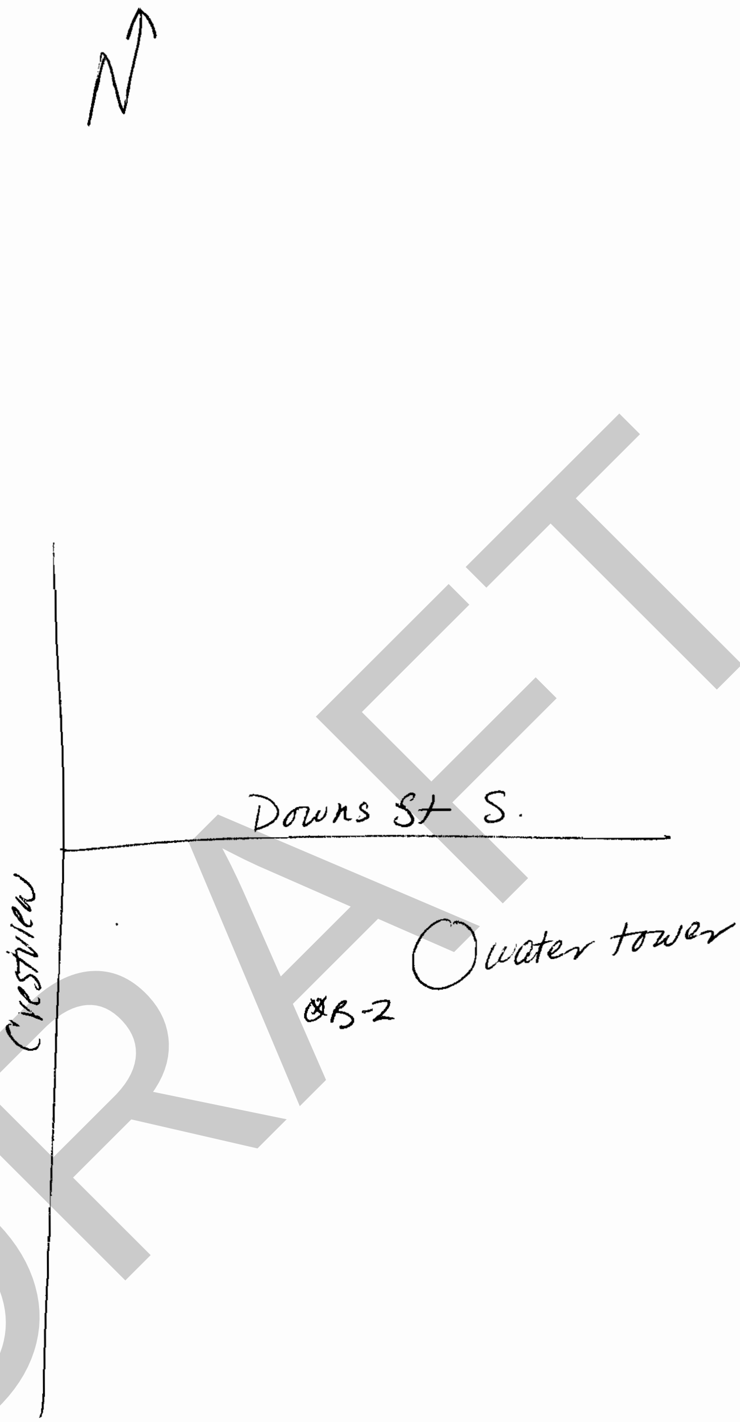
THIS REPORT MUST BE SUBMITTED TO THE WATER RESOURCES DEPARTMENT WITHIN 30 DAYS OF COMPLETION OF WORK

ORIGINAL & FIRST COPY-WATER RESOURCES DEPARTMENT SECOND COPY-CONSTRUCTOR THIRD COPY-CUSTOMER

TOPO! map printed on 06/28/06 from "OREGON.tpo" and "Untitled.tpg"



Map created with TOPO!® ©2002 National Geographic (www.nationalgeographic.com/topo)



1" = 50

APPENDIX F

EXISTING INFORMATION
SITE 10 - EDWARDS S1 PUMP STATION

MARI 50417

STATE OF OREGON
GEOTECHNICAL HOLE REPORT
(as required by OAR 690-240-035)

(1) OWNER/PROJECT: Hole Number B-3

Name City of Salem

Address 5355 Liberty ST. SE

City Salem State Or Zip 97301

(2) TYPE OF WORK

☒ New ☐ Deepening ☐ Alteration (repair/recondition) ☒ Abandonment

(3) CONSTRUCTION:

☐ Rotary Air ☐ Hand Auger ☐ Hollow Stem Auger

☒ Rotary Mud ☐ Cable Tool ☐ Push Probe ☐ Other

(4) TYPE OF HOLE:

☒ Uncased Temporary ☐ Cased Permanent

☐ Uncased Permanent ☐ Slope Stability ☐ Other

(5) USE OF HOLE: Foundation

(6) BORE HOLE CONSTRUCTION:

Special Construction approval ☐ Yes ☒ No Depth of Completed Hole 25 ft.

HOLE			SEAL			
Diameter	From	To	Material	From	To	Sacks or pounds
4 7/8	0	25	Holeplug	25	0	3

Backfill placed from ______ ft. to ______ ft. Material ______

Filter Pack placed from ______ ft. to ______ ft. Size of pack ______

(7) CASING/SCREEN:

	Diameter	From	To	Gauge	Steel	Plastic	Welded	Threaded
Casing:					☐	☐	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
Screen:					☐	☐	☐	☐
					☐	☐	☐	☐

Slot size ______

(8) WELL TEST:

☐ Pump ☐ Bailer ☐ Air ☐ Flowing Artesian

Permeability ______ Yield ______ GPM ______

Conductivity ______ PH ______

Temperature of water ______ °F/C Depth artesian flow found ______ ft.

Was water analysis done? ☐ Yes ☐ No

By whom? ______

Depth of strata analyzed. From ______ ft. to ______ ft.

Remarks: ______

(9) LOCATION OF HOLE by legal description:

County MARION Latitude ______ Longitude ______

Township 8S N or S Range 3W E or W. WM.

Section 2 S 1/4 E 1/4

Tax Lot ______ Lot ______ Block ______ Subdivision ______

Street Address of Well (or nearest address) ______

Map with location identified must be attached

(10) STATIC WATER LEVEL:

N/A ft. below land surface. Date 9-07-95

Artesian pressure ______ lb. per square inch. Date ______

(11) SUBSURFACE LOG:

Ground Elevation ______

Material Description	From	To	SWL
CL	0	13½	
Boulders + Gravel	13½	25	

Date Started ______ Date Completed ______

(12) ABANDONMENT LOG:

Material Description	From	To	Sacks or Pounds
Holeplug	25	0	3

Date started 9-07-95 Date Completed 9-07-95

Professional Certification

(to be signed by a licensed water supply or monitoring well constructor, or registered geologist or civil engineer).

I accept responsibility for the construction, alteration, or abandonment work performed on during the construction dates reported above. All work performed during this time is in compliance with Oregon geotechnical hole construction standards. This report is true to the best of my knowledge and belief.

License or Registration Number 10076

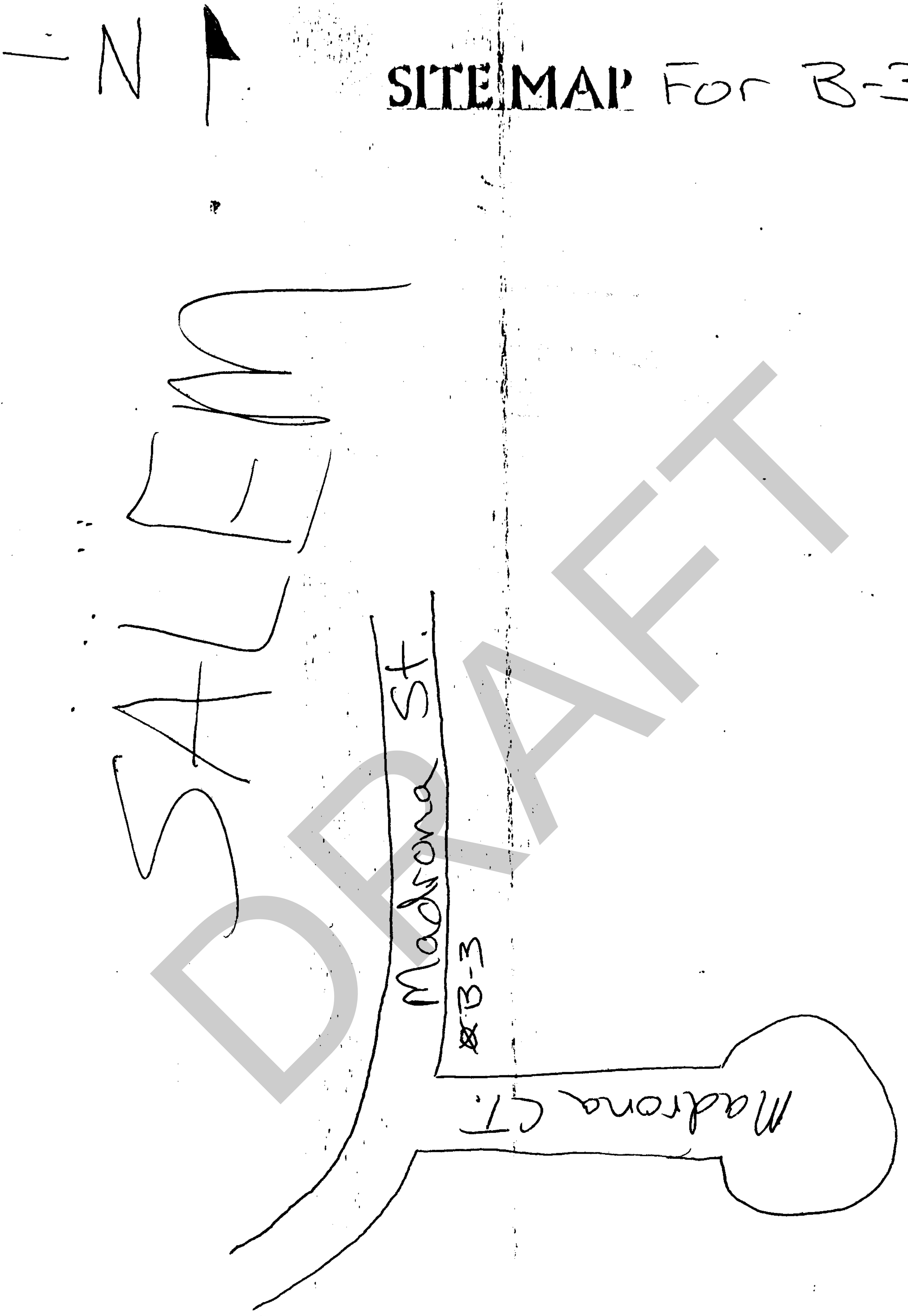
Signed Bradley Date 9-14-95

Affiliation ______

THIS REPORT MUST BE SUBMITTED TO THE WATER RESOURCES DEPARTMENT WITHIN 30 DAYS OF COMPLETION OF WORK

ORIGINAL & FIRST COPY-WATER RESOURCES DEPARTMENT SECOND COPY-CONSTRUCTOR THIRD COPY-CUSTOMER

SITE MAP For B-3



APPENDIX G

EXISTING INFORMATION
SITE 14 – LONE OAK RESERVOIR

MARI 56196

STATE OF OREGON
MONITORING WELL REPORT
(as required by ORS 537.765 & OAR 690-240-095)
Instructions for completing this report are on the last page of this form.

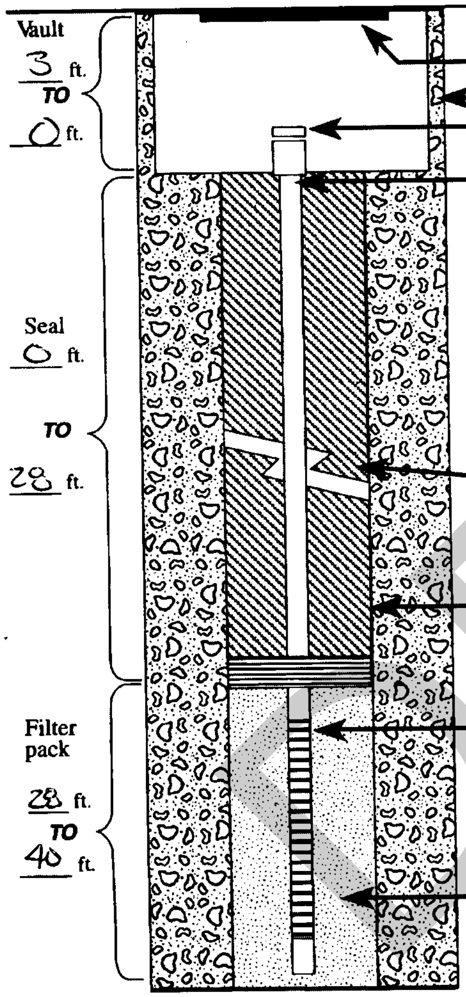
Well ID# L47219
Start Card # W 140867

(1) OWNER/PROJECT
WELL NO. MW 2
Name City of Salem Public Works Dept
Address 555 Liberty St SE Rm 325
City Salem State OR Zip 97301

(2) TYPE OF WORK
☒ New construction ☐ Alteration (Repair/Recondition)
☐ Conversion ☐ Deepening ☐ Abandonment

(3) DRILLING METHOD
☐ Rotary Air ☐ Rotary Mud ☐ Cable
☒ Hollow Stem Auger ☐ Other

(4) BORE HOLE CONSTRUCTION:
Special Standards Yes ☐ No ☒ Depth of Completed Well 40 ft.



Vault 3 ft. TO 0 ft.
Water-tight cover
~~Surface flush vault~~ ABOVE GROUND
Locking cap
Casing diameter 2" in.
material PVC
Welded ☐ Threaded ☒ Glued ☐
Liner diameter in.
material
Welded ☐ Threaded ☐ Glued ☐
Seal 0 ft. TO 28 ft.
Well seal:
Material BENT CHIPS
Amount 25 SK
Grout weight
Borehole diameter 10 in.
Bentonite plug at least 3 ft. thick
Screen material PVC
interval(s):
From 30 To 40
From To
Slot size .010 in.
Filter pack 28 ft. TO 40 ft.
Filter pack:
Material COL. SAND
Size 20 x 40 in.

(5) WELL TESTS:
☐ Pump ☐ Bailer ☐ Air ☐ Flowing Artesian
Permeability Yield GPM
Conductivity PH
Temperature of water °F/C Depth artesian flow found ft.
Was water analysis done? ☐ Yes ☐ No NONE
By whom?
Depth of strata to be analyzed. From ft. to ft.
Remarks:

Name of supervising Geologist/Engineer

(6) LOCATION OF WELL By legal description:
County MARION Latitude Longitude
Township 8 (N or [✓ S]) Range 3 (E or [✓ W]) Section 16
SE 1/4 of SE 1/4 of above section.
Street address of well location LONE OAK & MILDRED
Tax lot number of well location ROW
ATTACH MAP WITH LOCATION IDENTIFIED. Map shall include approximate scale and north arrow.

(7) STATIC WATER LEVEL:
Ft. below land surface. Date
Artesian Pressure lb/sq. in. Date
NONE

(8) WATER BEARING ZONES:
Depth at which water was first found

From	To	Est. Flow Rate	SWL

(9) WELL LOG:
Ground Elevation

Material	From	To	SWL
RES. DUEL SOIL	0	40	

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NOV - 8 2001
WATER RESOURCES DEPT.
SALEM, OREGON

Date started 10/10/01 Completed 10/10/01

(unbonded) Monitor Well Constructor Certification:

I certify that the work I performed on the construction, alteration, or abandonment of this well is in compliance with Oregon water supply well construction standards. Materials used and information reported above are true to the best of my knowledge and belief.

Signed J. Fort Caste MWC Number 10306 Date 10-13-01

(bonded) Monitor Well Constructor Certification:

I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. All work performed during this time is in compliance with Oregon water supply well construction standards. This report is true to the best of my knowledge and belief.

Signed MWC Number 10459 Date 10/12/01

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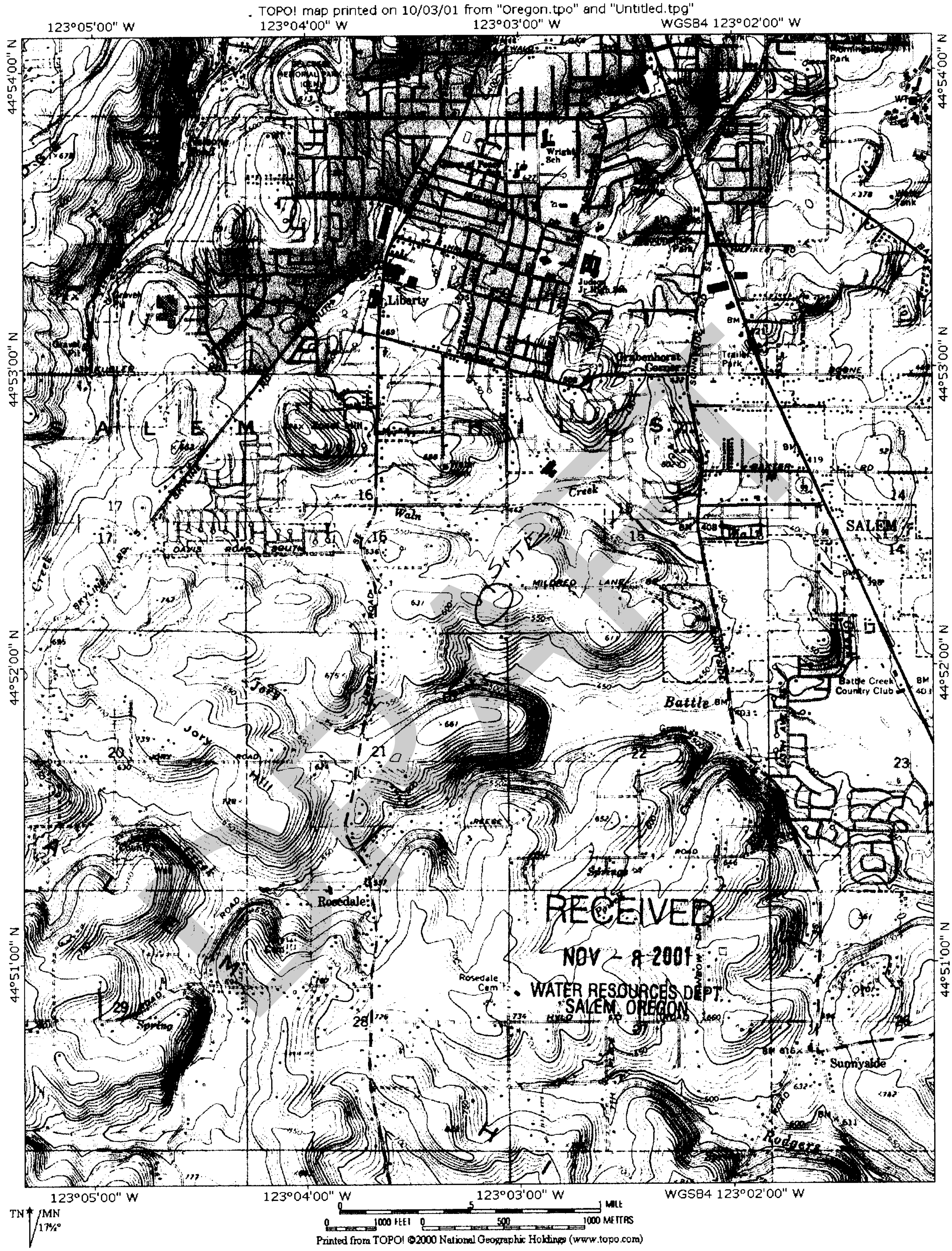
MARI 56196

TOPO! map printed on 10/03/01 from "Oregon.tpo" and "Untitled.tpg"



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STATE OF OREGON
MONITORING WELL REPORT
(as required by ORS 537.765 & OAR 690-240-095)
Instructions for completing this report are on the last page of this form.

MARI 56197

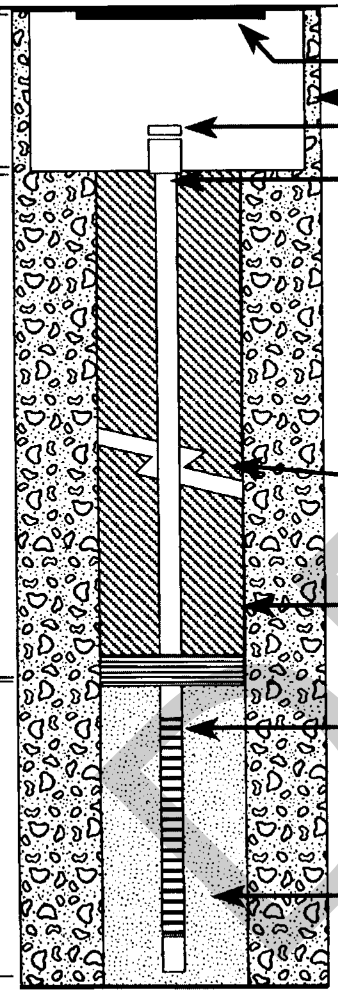
Well ID# L47218
Start Card # W140866

(1) OWNER/PROJECT
WELL NO. MW 1
Name CITY OF SALEM PUBLIC WORKS DEPT.
Address 555 LIBERTY ST SE RM 325
City SALEM State OR Zip 97301

(2) TYPE OF WORK
☒ New construction ☐ Alteration (Repair/Recondition)
☐ Conversion ☐ Deepening ☐ Abandonment

(3) DRILLING METHOD
☐ Rotary Air ☐ Rotary Mud ☐ Cable
☒ Hollow Stem Auger ☐ Other

(4) BORE HOLE CONSTRUCTION:
Special Standards Yes ☐ No ☒ Depth of Completed Well 40 ft.



Land surface

Vault 3 ft. TO 2 ft.

Water-tight cover
~~Surface flush vault~~ ABOVE GROUND
Locking cap

Casing
diameter 2" in.
material PVC
Welded ☐ Threaded ☒ Glued ☐

Liner
diameter in.
material
Welded ☐ Threaded ☐ Glued ☐

Seal 0 ft. TO 28 ft.

Well seal:
Material BENT CHIPS
Amount 25 SKS
Grout weight
Borehole diameter 10 in.
Bentonite plug at least 3 ft. thick

Filter pack 28 ft. TO 40 ft.

Screen
material PVC
interval(s):
From 30 To 40
From To
Slot size .010 in.

Filter pack:
Material COL. SAND
Size 20 X 40 in.

(5) WELL TESTS:
☐ Pump ☐ Bailer ☐ Air ☐ Flowing Artesian
Permeability Yield GPM
Conductivity PH
Temperature of water °F/C Depth artesian flow found ft.
Was water analysis done? ☐ Yes ☒ No
NONE
By whom?
Depth of strata to be analyzed. From ft. to ft.
Remarks:

Name of supervising Geologist/Engineer

(6) LOCATION OF WELL By legal description:
County MARION Latitude Longitude
Township 8 (N or [✓ S]) Range 3 (E or [✓ W]) Section 16
SE 1/4 of SE 1/4 of above section.
Street address of well location LONE OAK & MILDRED
Tax lot number of well location ROW
ATTACH MAP WITH LOCATION IDENTIFIED. Map shall include approximate scale and north arrow.

(7) STATIC WATER LEVEL:
Ft. below land surface. Date
Artesian Pressure lb/sq. in. Date
NONE

(8) WATER BEARING ZONES:
Depth at which water was first found

From	To	Est. Flow Rate	SWL

(9) WELL LOG:
Ground Elevation

Material	From	To	SWL
RESIDUAL SOIL	0	40	

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WATER RESOURCES DEPT.
SALEM, OREGON

Date started 10/9/01 Completed 10/9/01

(unbonded) Monitor Well Constructor Certification:

I certify that the work I performed on the construction, alteration, or abandonment of this well is in compliance with Oregon water supply well construction standards. Materials used and information reported above are true to the best of my knowledge and belief.

MWC Number
Signed Date

(bonded) Monitor Well Constructor Certification:

I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. All work performed during this time is in compliance with Oregon water supply well construction standards. This report is true to the best of my knowledge and belief.

MWC Number 10459
Signed Date 10/12/01

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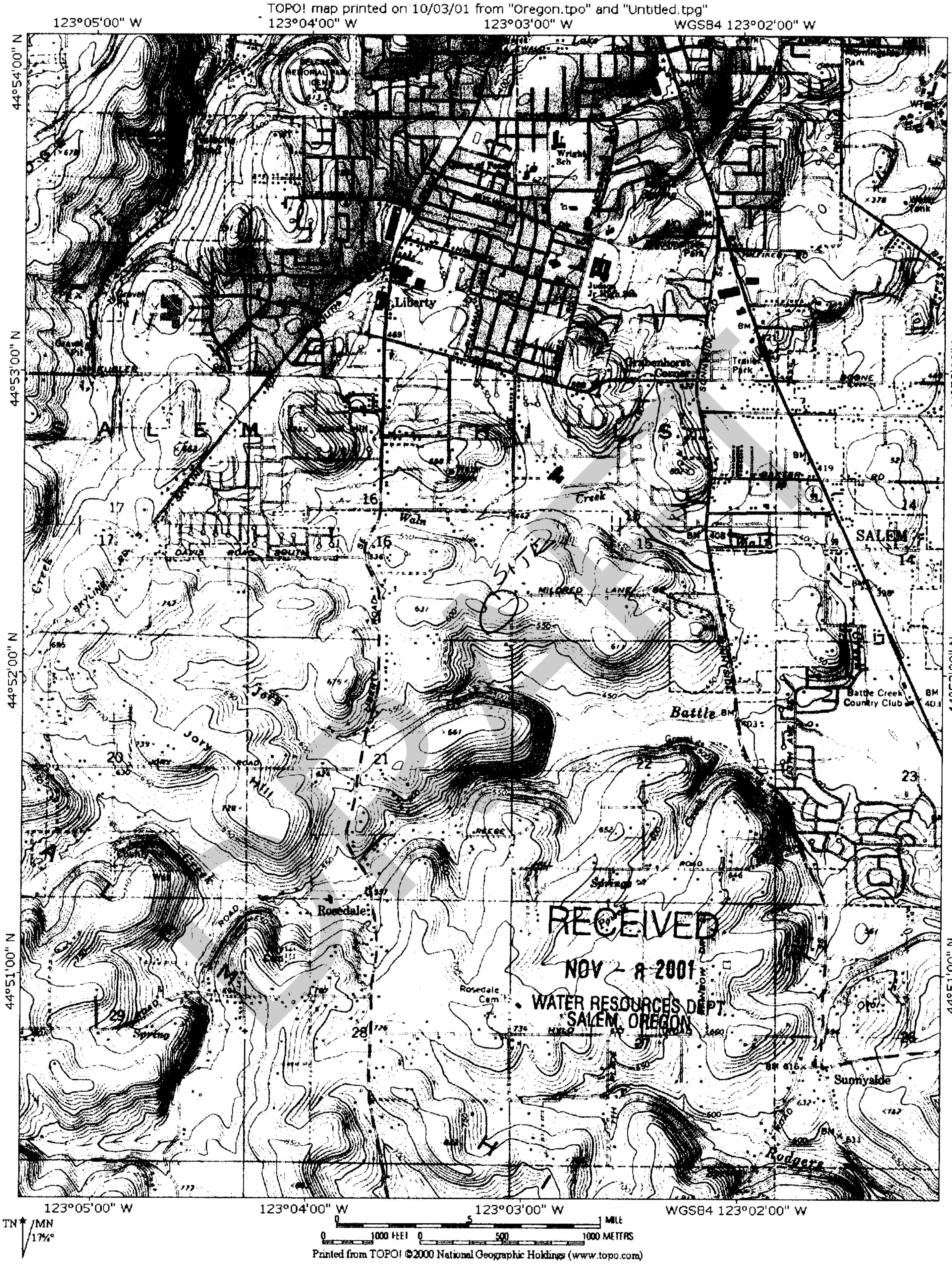
MARI 56197

TOPO! map printed on 10/03/01 from "Oregon.tpo" and "Untitled.tpg"



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STATE OF OREGON

MONITORING WELL REPORT

(as required by ORS 537.765 & OAR 690-240-095)

Instructions for completing this report are on the last page of this form.

MARI 56938

47218

Start Card # 140888

(1) OWNER/PROJECT:

WELL NO. MW 1

Name CITY OF SALEM PUBLIC WORK DEPART.

Address 555 LIBERTY ST SE RM 325

City SALEM State OR Zip 97501

(2) TYPE OF WORK:

☐ New construction ☐ Alteration (Repair/Recondition)

☐ Conversion ☐ Deepening ☒ Abandonment

(3) DRILLING METHOD:

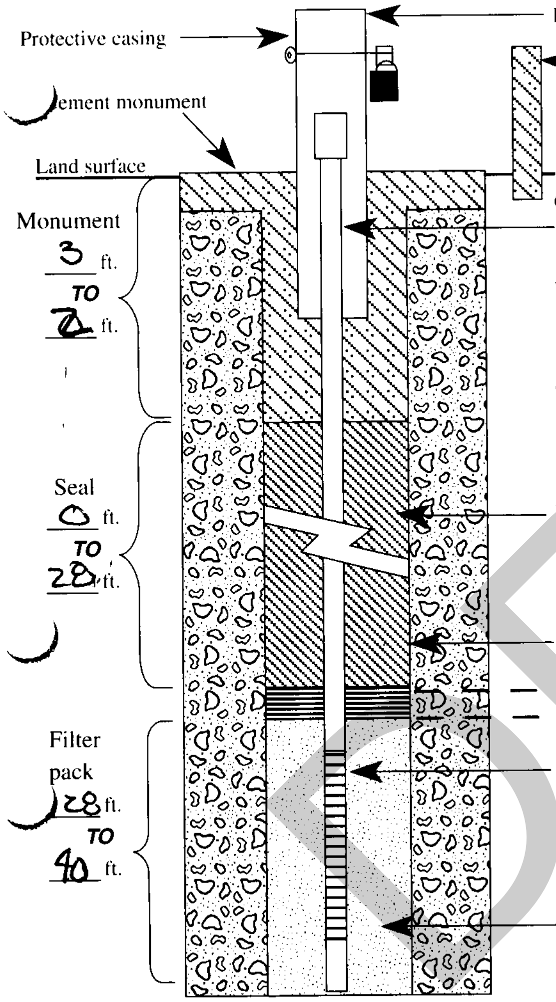
☐ Rotary Air ☐ Rotary Mud ☐ Cable

☒ Hollow Stem Auger ☐ Other

(4) BORE HOLE CONSTRUCTION

Special Standards Yes ☐ No ☒

Depth of completed well 40 ft.



Monument 3 ft. TO 2 ft.

Seal 0 ft. TO 28 ft.

Filter pack 28 ft. TO 40 ft.

Casing diameter 2" in.

material PVC

Welded ☐ Threaded ☒ Glued ☐

Liner diameter ______ in.

material ______

Welded ☐ Threaded ☐ Glued ☐

Well seal:

Material BENT CHIPS

Amount 25SKS

Grout weight ______

Borehole diameter 10 in.

Bentonite plug at least 3 ft. thick

Screen material PVC

interval(s):

From 30 To 40

From ______ To ______

Slot size .010 in.

Filter pack:

Material COL. SAND

Size 20 x 40 in.

(5) WELL TEST:

☐ Pump ☐ Bailer ☐ Air ☐ Flowing Artesian

Permeability ______ Yield ______ GPM

Conductivity ______ PH ______

Temperature of water ______ °F/C Depth artesian flow found ______ ft.

Was water analysis done? ☐ Yes ☐ No

NONE

By whom? ______

Depth of strata to be analyzed. From ______ ft. to ______ ft.

Remarks: ______

Name of supervising Geologist/Engineer ______

(6) LOCATION OF WELL By legal description

Well Location: County MARION

Township 8 (N or [✓ S]) Range 3 (E or [✓ W]) Section 16

1. SE 1/4 of SE 1/4 of above section.

2. **Either** Street address of well location LONE OAK & MILDRED

or Tax lot number of well location ROW

3. ATTACH MAP WITH LOCATION IDENTIFIED. Map shall include approximate scale and north arrow.

(7) STATIC WATER LEVEL:

______ Ft. below land surface. Date ______

Artesian Pressure ______ lb/sq. in. Date ______

NONE

(8) WATER BEARING ZONES:

Depth at which water was first found ______

From	To	Est. Flow Rate	SWL

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SALEM, OREGON

(9) WELL LOG:

Ground elevation ______

Material	From	To	SWL
RESIDUAL SOIL	0	40	
ABANDOMENT ON			
10/14/02			
FILLED WITH BENT			
GROUT FROM 40' TO 1'	40	1	
& BENT CHIPS FROM	BENT	GROUT 9.7#/ga	
1' TO 0'	1	0	
	BENT	CHIPS	1SK
MONUMENT & PROTECTIVE			
POST REMOVE			

Date started ~~10/9/02~~ 10/9/01 Completed ~~10/9/02~~ 10/9/01

(unbonded) Monitor Well Constructor Certification:

I certify that the work I performed on the construction, alteration, or abandonment of this well is in compliance with Oregon well construction standards. Materials used and information reported above are true to the best knowledge and belief.

MWC Number ______

Signed ______ Date ______

(bonded) Monitor Well Constructor Certification:

I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. All work performed during this time is in compliance with Oregon well construction standards. This report is true to the best of my knowledge and belief.

MWC Number 10459

Signed Date 10/14/02

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STATE OF OREGON

MARI 56939

MONITORING WELL REPORT

(as required by ORS 537.765 & OAR 690-240-095)

L91219

Start Card # 140889

Instructions for completing this report are on the last page of this form.

(1) OWNER/PROJECT:

WELL NO. MW 2

Name City of Salem Public Work Depart

Address 555 Liberty St SE Rm 325

City Salem State OR Zip 97301

(2) TYPE OF WORK:

☐ New construction ☐ Alteration (Repair/Recondition)

☐ Conversion ☐ Deepening ☒ Abandonment

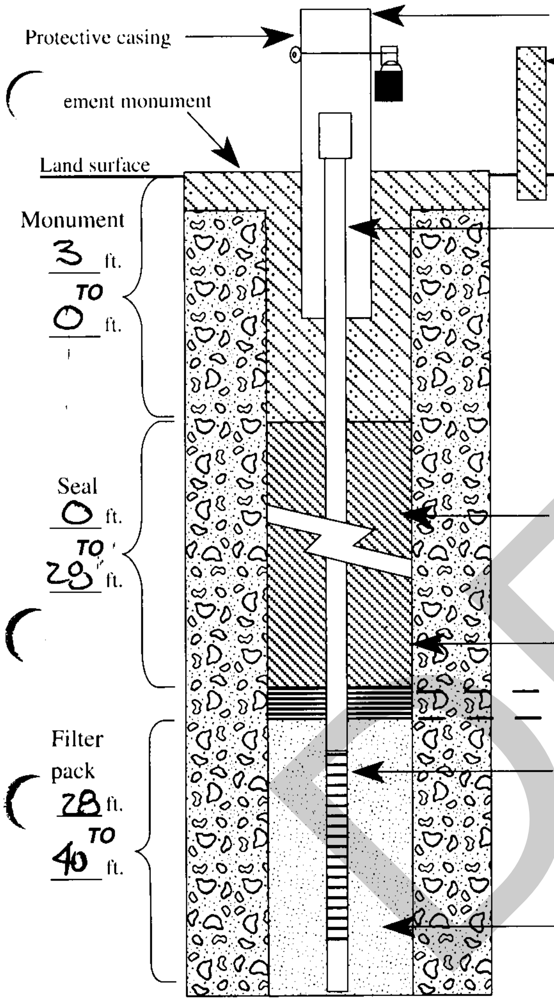
(3) DRILLING METHOD

☐ Rotary Air ☐ Rotary Mud ☐ Cable

☑ Hollow Stem Auger ☐ Other

(4) BORE HOLE CONSTRUCTION

Special Standards Yes ☐ No ☒ Depth of completed well 40 ft.



Monument 3 ft. TO 0 ft.

Seal 0 ft. TO 28 ft.

Filter pack 28 ft. TO 40 ft.

Casing diameter 2" in.

material PVC

Welded ☐ Threaded ☒ Glued ☐

Liner diameter ____ in.

material ____

Welded ☐ Threaded ☐ Glued ☐

Well seal:

Material Bent Chips

Amount 25 SKS

Grout weight ____

Borehole diameter 10 in.

Bentonite plug at least 3 ft. thick

Screen material PVC

interval(s):

From 30 To 40

From ____ To ____

Slot size .010 in.

Filter pack:

Material Col. Sand

Size 20X40 in.

(5) WELL TEST:

☐ Pump ☐ Bailer ☐ Air ☐ Flowing Artesian

Permeability ____ Yield ____ GPM

Conductivity ____ PH ____

Temperature of water ____ °F/C Depth artesian flow found ____ ft.

Was water analysis done? ☐ Yes ☐ No

By whom? ____

DONE

Depth of strata to be analyzed. From ____ ft. to ____ ft.

Remarks: ____

Name of supervising Geologist/Engineer ____

(6) LOCATION OF WELL By legal description

Well Location: County Marion

Township 8 (N or [✓ S]) Range 3 (E or [✓ W]) Section 16

1. SE 1/4 of SE 1/4 of above section.

2. **Either** Street address of well location Lone Oak & Mildred

or Tax lot number of well location ROW

3. ATTACH MAP WITH LOCATION IDENTIFIED. Map shall include approximate scale and north arrow.

(7) STATIC WATER LEVEL:

____ Ft. below land surface. Date ____

Artesian Pressure None lb/sq.in. Date ____

(8) WATER BEARING ZONES:

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WATER RESOURCES DEPT
SALEM, OREGON

Depth at which water was first found ____

From	To	Est. Flow Rate	SWL

(9) WELL LOG:

Ground elevation ____

Material	From	To	SWL
Residual Soil	0	40	
Monument &			
Protective Post			
Remove			
Abandonment on			
~~[illegible]~~ 10/14/02			
Filled with Bent	40	1	
Grout from 40' to	Bent	Grout	
1' & Bent chips	9.7#/Gal		
From 1' to 0'	1	0	
	Bent	Chips	
	1 SK		

Date started ~~[illegible]~~ 10/10/01 Completed ~~[illegible]~~ 10/10/01

(unbonded) Monitor Well Constructor Certification:

I certify that the work I performed on the construction, alteration, or abandonment of this well is in compliance with Oregon well construction standards. Materials used and information reported above are true to the best knowledge and belief.

MWC Number ____

Signed ____ Date ____

(bonded) Monitor Well Constructor Certification:

I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. All work performed during this time is in compliance with Oregon well construction standards. This report is true to the best of my knowledge and belief.

MWC Number 10459

Signed ____ Date 10/14/02

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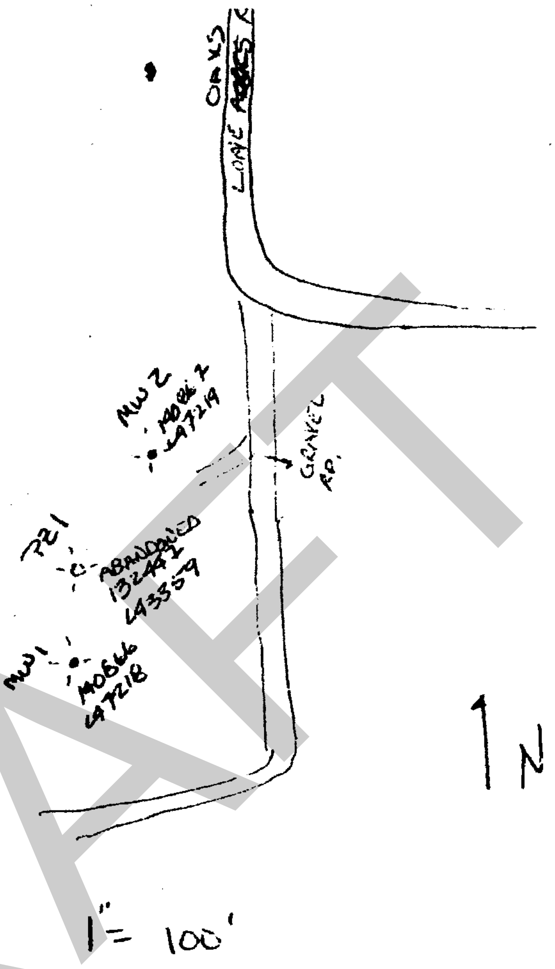
MARI 56939

56938
56939



MARI 57159

STATE OF OREGON
WATER SUPPLY WELL REPORT
(as required by ORS 537.765)

Instructions for completing this report are on the last page of this form.

WELL I.D. # L 62635
START CARD # 155923

(1) LAND OWNER Well Number ________
Name Don Company Inc.
Address 390 Holder Ln SE
City Salem State OR Zip 97306

(2) TYPE OF WORK
☒ New Well ☐ Deepening ☐ Alteration (repair/recondition) ☐ Abandonment

(3) DRILL METHOD:
☒ Rotary Air ☐ Rotary Mud ☐ Cable ☐ Auger
☐ Other ________

(4) PROPOSED USE:
☒ Domestic ☐ Community ☐ Industrial ☒ Irrigation
☐ Thermal ☐ Injection ☐ Livestock ☐ Other ________

(5) BORE HOLE CONSTRUCTION:
Special Construction approval ☐ Yes ☒ No Depth of Completed Well 314 ft.
Explosives used ☐ Yes ☒ No Type ________ Amount ________

HOLE			SEAL			
Diameter	From	To	Material	From	To	[✓ Sacks] or pounds
10	0	56	Cement	-7	237	30 + 5% bent.
8	56	237				
6.5	237	304				
6	304	314				

How was seal placed: Method ☐ A ☐ B ☒ C ☑ D ☐ E
☐ Other ________
Backfill placed from ____ ft. to ____ ft. Material ________
Gravel placed from ____ ft. to ____ ft. Size of gravel ________

(6) CASING/LINER:

	Diameter	From	To	Gauge	Steel	Plastic	Welded	Threaded
Casing:	6 in	+1	237	.25	☒	☐	☑	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
Liner:	4 in	0	314	#160	☐	☒	☐	☐
					☐	☐	☐	☐

Drive Shoe used ☐ Inside ☐ Outside ☒ None
Final location of shoe(s) ________

(7) PERFORATIONS/SCREENS:
☒ Perforations Method Saw
☐ Screens Type ________ Material ________

From	To	Slot size	Number	Diameter	Tele/pipe size	Casing	Liner
274	309	8×6	56			☐	☑
						☐	☐
						☐	☐
						☐	☐

(8) WELL TESTS: Minimum testing time is 1 hour
☐ Pump ☐ Bailer ☒ Air ☐ Flowing Artesian

Yield gal/min	Drawdown	Drill stem at	Time
20		312	1 hr.

Temperature of water 53 ± Depth Artesian Flow Found ________
Was a water analysis done? ☐ Yes By whom ________
Did any strata contain water not suitable for intended use? ☐ Too little
☐ Salty ☐ Muddy ☐ Odor ☐ Colored ☐ Other ________
Depth of strata: ________

(9) LOCATION OF WELL by legal description:
County Marion Latitude ________ Longitude ________
Township 8-S N or S Range 3-W E or W. WM.
Section 16 SE 1/4 SE 1/4
Tax Lot 400 Lot ____ Block ____ Subdivision ____
Street Address of Well (or nearest address) Same as #1

(10) STATIC WATER LEVEL:
249 ft. below land surface. Date 3-19-03
Artesian pressure ________ lb. per square inch Date ________

(11) WATER BEARING ZONES:
Depth at which water was first found 16

From	To	Estimated Flow Rate	SWL
16	48		
122	140	2	105
252	314	20	249

(12) WELL LOG:
Ground Elevation ________

Material	From	To	SWL
Red Soil	1	6	
Red Clay	6	16	
Brown Clay Soft	16	48	
Red Clay - hard	48	54	
Semi-Weathered basalt	54	76	
Hard gray basalt	76	106	
Med black basalt	106	122	
Very Weathered brown basalt	122	140	
Gray basalt - hard	140	240	
Semi-Weathered basalt	240	252	
Porous black basalt caving	252	314	

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SALEM, OREGON

Date started 3-12-03 Completed 3-19-03

(unbonded) Water Well Constructor Certification:

I certify that the work I performed on the construction, alteration, or abandonment of this well is in compliance with Oregon water supply well construction standards. Materials used and information reported above are true to the best of my knowledge and belief.

WWC Number 1629
Signed ________ Date 3-20-03

(bonded) Water Well Constructor Certification:

I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. All work performed during this time is in compliance with Oregon water supply well construction standards. This report is true to the best of my knowledge and belief.

WWC Number 1273
Signed Floyd N Sipple Date 3-20-03

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APPENDIX H

EXISTING INFORMATION
SITE 16 - CHAMPION HILL RESERVOIR

#14

Mar 17040

85/3W/28 ac

STATE OF OREGON
WATER WELL REPORT
(as required by ORS 537.765)

(START CARD) # 22888.

(1) OWNER:
Well Number:
Name William R. Long
Address 11401 Steinkamp
City Aumsville State Or Zip 97325

(2) TYPE OF WORK:
☒ New Well ☐ Deepen ☐ Recondition ☐ Abandon

(3) DRILL METHOD
☒ Rotary Air ☐ Rotary Mud ☐ Cable
☐ Other

(4) PROPOSED USE:
☒ Domestic ☐ Community ☐ Industrial ☐ Irrigation
☐ Thermal ☐ Injection ☐ Other

(5) BORE HOLE CONSTRUCTION:
Special Construction approval Yes ☐ No ☒ Depth of Completed Well 250 ft.
Explosives used Yes ☐ No ☒ Type Amount

HOLE			SEAL			Amount
Diameter	From	To	Material	From	To	sacks or pounds
10	0	20	Cement.	0	20	7
8	20	86	Cement.	20	84	8
6	86	250				

How was seal placed: Method ☐ A ☐ B ☒ C ☐ D ☐ E
☐ Other
Backfill placed from ft. to ft. Material
Gravel placed from ft. to ft. Size of gravel

(6) CASING/LINER:

	Diameter	From	To	Gauge	Steel	Plastic	Welded	Threaded
Casing:	6	+1	86	250	☒	☐	☒	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
Liner:					☐	☐	☐	☐
					☐	☐	☐	☐

Final location of shoe(s)

(7) PERFORATIONS/SCREENS:
☐ Perforations Method
☐ Screens Type Material

From	To	Slot size	Number	Diameter	Tele/pipe size	Casing	Liner
						☐	☐
						☐	☐
						☐	☐
						☐	☐
						☐	☐
						☐	☐

(8) WELL TESTS: Minimum testing time is 1 hour
☐ Pump ☐ Bailer ☒ Air ☐ Flowing Artesian

Yield gal/min	Drawdown	Drill stem at	Time
18		245'	1 hr.

Temperature of water Depth Artesian Flow Found
Was a water analysis done? ☐ Yes By whom
Did any strata contain water not suitable for intended use? ☐ Too little
☐ Salty ☐ Muddy ☐ Odor ☐ Colored ☐ Other
Depth of strata:

(9) LOCATION OF WELL by legal description:
County Marion Latitude Longitude
Township 8S N or S, Range 3W E or W, WM.
Section 28 SW ¼ NE ¼
Tax Lot Lot Block Subdivision
Street Address of Well (or nearest address) 151 Hylo Rd.
Salem, Or

(10) STATIC WATER LEVEL:
147 ft. below land surface. Date Oct 28, 1990
Artesian pressure lb. per square inch. Date

(11) WATER BEARING ZONES:
Depth at which water was first found 180'

From	To	Estimated Flow Rate	SWL
180	250	18 G.P.M.	147

(12) WELL LOG:
Ground elevation

Material	From	To	SWL
Soil	0	4	
Clay Red	4	30	
Clay Grey	30	60	
Clay Red	60	78	
Rock Very Hard Grey	78	180	
Rock Black Broken.	180	250	147

RECEIVED
NOV 13 1990
WATER RESOURCES DEPT.
SALEM, OREGON

Date started Oct. 23, 1990 Completed Oct 28, 1990

(unbonded) Water Well Constructor Certification:

I certify that the work I performed on the construction, alteration, or abandonment of this well is in compliance with Oregon well construction standards. Materials used and information reported above are true to my best knowledge and belief.

WWC Number
Signed Date

(bonded) Water Well Constructor Certification:

I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. all work performed during this time is in compliance with Oregon well construction standards. This report is true to the best of my knowledge and belief.

WWC Number 75
Signed William R Long Date Oct 28, 1990

ORIGINAL & FIRST COPY - WATER RESOURCES DEPARTMENT SECOND COPY - CONSTRUCTOR THIRD COPY - CUSTOMER 9809C 3/88

#16

Mari
17041

88/360/28 ac

STATE OF OREGON
WATER WELL REPORT
(as required by ORS 537.765)

(START CARD) # 22887.

(1) OWNER:
Well Number:
Name William R Long - Sloan.
Address 11401 Steinkamp Rd
City Aumsville, Or State Or. Zip 97325

(2) TYPE OF WORK:
☒ New Well ☐ Deepen ☐ Recondition ☐ Abandon

(3) DRILL METHOD
☒ Rotary Air ☐ Rotary Mud ☐ Cable
☐ Other

(4) PROPOSED USE:
☒ Domestic ☐ Community ☐ Industrial ☐ Irrigation
☐ Thermal ☐ Injection ☐ Other

(5) BORE HOLE CONSTRUCTION:
Special Construction approval Yes ☐ No ☒ Depth of Completed Well 250 ft.
Explosives used Yes ☐ No ☒ Type Amount

HOLE			SEAL			Amount
Diameter	From	To	Material	From	To	sacks ~~or pounds~~
10	0	20	Cement	0	20	7
8	20	85	Cement	20	85	7
5 7/8	85	250				

How was seal placed: Method ☐ A ☐ B ☒ C ☐ D ☐ E
☐ Other
Backfill placed from ft. to ft. Material
Gravel placed from ft. to ft. Size of gravel

(6) CASING/LINER:

	Diameter	From	To	Gauge	Steel	Plastic	Welded	Threaded
Casing:	6	+1	85	.250	☒	☐	☒	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
Liner:					☐	☐	☐	☐
					☐	☐	☐	☐

Final location of shoe(s)

(7) PERFORATIONS/SCREENS:
☐ Perforations Method
☐ Screens Type Material

From	To	Slot size	Number	Diameter	Tele/pipe size	Casing	Liner
						☐	☐
						☐	☐
						☐	☐
						☐	☐
						☐	☐
						☐	☐

(8) WELL TESTS: Minimum testing time is 1 hour
☐ Pump ☐ Bailer ☒ Air ☐ Flowing Artesian

Yield gal/min	Drawdown	Drill stem at	Time
20		245	1 hr.

Temperature of water Depth Artesian Flow Found
Was a water analysis done? ☐ Yes By whom
Did any strata contain water not suitable for intended use? ☐ Too little
☐ Salty ☐ Muddy ☐ Odor ☐ Colored ☐ Other
Depth of strata:

(9) LOCATION OF WELL by legal description:
County Marion Latitude Longitude
Township 8S. N or S. Range 3W E or W, WM.
Section 28 S.W ¼ N.E. ¼
Tax Lot Lot Block Subdivision
Street Address of Well (or nearest address) 163 Hylo Rd. Salem, Or

(10) STATIC WATER LEVEL:
145 ft. below land surface. Date Oct 22, 1990
Artesian pressure lb. per square inch. Date

(11) WATER BEARING ZONES:
Depth at which water was first found 170'

From	To	Estimated Flow Rate	SWL
170	250	20 G.P.M.	145

(12) WELL LOG:
Ground elevation

Material	From	To	SWL
Soil	0	4	
Clay Red	4	30	
Clay Gray	30	40	
Clay Red	40	73	
Rock Very Hard Gray	73	170	
Rock Black Broken	170	250	145

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NOV 13 1990
WATER RESOURCES DEPT.
SALEM, OREGON

Date started Oct. 17, 1990 Completed Oct 22, 1990

(unbonded) Water Well Constructor Certification:

I certify that the work I performed on the construction, alteration, or abandonment of this well is in compliance with Oregon well construction standards. Materials used and information reported above are true to my best knowledge and belief.

WWC Number
Signed Date

(bonded) Water Well Constructor Certification:

I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. all work performed during this time is in compliance with Oregon well construction standards. This report is true to the best of my knowledge and belief.

WWC Number 75
Signed William R Long Date Oct 22, 1990

16

MAR1 17405

8S/3W/28 ac

STATE OF OREGON
WATER WELL REPORT
(as required by ORS 537.765)

(START CARD) # 31621

(1) OWNER:
Well Number:

Name William R Long

Address 11101 Steinkamp Rd

City Brownsville. State Or. Zip 97327.

(2) TYPE OF WORK:
☒ New Well ☐ Deepen ☐ Recondition ☐ Abandon

(3) DRILL METHOD
☒ Rotary Air ☐ Rotary Mud ☐ Cable

☐ Other

(4) PROPOSED USE:
☒ Domestic ☐ Community ☐ Industrial ☐ Irrigation

☐ Thermal ☐ Injection ☐ Other

(5) BORE HOLE CONSTRUCTION:
Special Construction approval Yes ☐ No ☒ Depth of Completed Well 270 ft.

Explosives used Yes ☐ No ☒ Type Amount

HOLE			SEAL			Amount
Diameter	From	To	Material	From	To	sacks ~~or pounds~~
10	0	20	Cement	0	20	8
8	20	75	Cement	20	75'	10
6	75	270				

How was seal placed: Method ☐ A ☐ B ☒ C ☐ D ☐ E

☐ Other

Backfill placed from ft. to ft. Material

Gravel placed from ft. to ft. Size of gravel

(6) CASING/LINER:

	Diameter	From	To	Gauge	Steel	Plastic	Welded	Threaded
Casing:	6	+1	75	.250	☒	☐	☒	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
Liner:					☐	☐	☐	☐
					☐	☐	☐	☐

Final location of shoe(s)

(7) PERFORATIONS/SCREENS:
☐ Perforations Method

☐ Screens Type Material

From	To	Slot size	Number	Diameter	Tele/pipe size	Casing	Liner
						☐	☐
						☐	☐
						☐	☐
						☐	☐
						☐	☐
						☐	☐

(8) WELL TESTS: Minimum testing time is 1 hour
☐ Pump ☐ Bailer ☒ Air ☐ Flowing Artesian

Yield gal/min	Drawdown	Drill stem at	Time
30		265	1 hr.

Temperature of water 52 Depth Artesian Flow Found

Was a water analysis done? ☐ Yes By whom

Did any strata contain water not suitable for intended use? ☐ Too little

☐ Salty ☐ Muddy ☐ Odor ☐ Colored ☐ Other

Depth of strata:

(9) LOCATION OF WELL by legal description:
County Marion Latitude ' " Longitude ' "

Township 8S N or S. Range 3W E or W. WM.

Section 28 S.W ¼ N.E ¼

Tax Lot Lot Block Subdivision

Street Address of Well (or nearest address) 173 Hylo Rd S.E Salem, Or.

(10) STATIC WATER LEVEL:
132 ft. below land surface. Date July 24, 1991

Artesian pressure lb. per square inch. Date

(11) WATER BEARING ZONES:
Depth at which water was first found 180'

From	To	Estimated Flow Rate	SWL
180'	270	30 G.P.M.	132

(12) WELL LOG:
Ground elevation

Material	From	To	SWL
Soil	0	6	
Clay Tan	6	30	
Clay Brick Red	30	40	
Clay Tan	40	68	
Rock Black Hard	68	160	
Rock Brown	160	175	
Rock Black Broken Hard	175	270	132

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AUG 05 1991
WATER RESOURCES DEPT
SALEM, OREGON

Date started July 18, 1991 Completed July 24, 1991

(unbonded) Water Well Constructor Certification:

I certify that the work I performed on the construction, alteration, or abandonment of this well is in compliance with Oregon well construction standards. Materials used and information reported above are true to my best knowledge and belief.

WWC Number

Signed Date

(bonded) Water Well Constructor Certification:

I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. all work performed during this time is in compliance with Oregon well construction standards. This report is true to the best of my knowledge and belief.

WWC Number 75

Signed William R Long Date July 24, 1991

MARI
17739

MAR 30 1992

8s/3w/28aa

STATE OF OREGON
WATER WELL REPORT
(as required by ORS 537.765)

WATER RESOURCES DEPT (START CARD) # 25562

(1) OWNER:
Well Number: 1

Name: Raymond Holman & Helen Foley
Address: 7054 Liberty Rd. S.E.
City: Salem State: Ore. Zip: 97306

(2) TYPE OF WORK:
☑ New Well ☐ Deepen ☐ Recondition ☐ Abandon

(3) DRILL METHOD
☑ Rotary Air ☐ Rotary Mud ☐ Cable
☐ Other

(4) PROPOSED USE:
☑ Domestic ☐ Community ☐ Industrial ☐ Irrigation
☐ Thermal ☐ Injection ☐ Other

(5) BORE HOLE CONSTRUCTION:
Special Construction approval Yes ☐ No ☑ Depth of Completed Well 283 ft.
Explosives used Yes ☐ No ☑ Type Amount

HOLE			SEAL			Amount
Diameter	From	To	Material	From	To	sacks or pounds
10"	0	54	Cement	41	54	2 sak
			Bentonite	0	20+	7 sak
6"	54	283				

How was seal placed: Method ☐ A ☐ B ☑ C ☐ D ☐ E
☑ Other Poured Dry Bentonite To Fill
Backfill placed from ft. to ft. Material
Gravel placed from ft. to ft. Size of gravel

(6) CASING/LINER:

	Diameter	From	To	Gauge	Steel	Plastic	Welded	Threaded
Casing:	6"	+1	54'	250	☑	☐	☑	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
Liner:					☐	☐	☐	☐
					☐	☐	☐	☐

Final location of shoe(s)

(7) PERFORATIONS/SCREENS:
☐ Perforations Method
☐ Screens Type Material

From	To	Slot size	Number	Diameter	Tele/pipe size	Casing	Liner
						☐	☐
						☐	☐
						☐	☐
						☐	☐
						☐	☐
						☐	☐

(8) WELL TESTS: Minimum testing time is 1 hour
☐ Pump ☐ Bailer ☐ Air ☐ Flowing Artesian

Yield gal/min	Drawdown	Drill stem at	Time
3 GPM		283	1 hr.

Temperature of water 52° Depth Artesian Flow Found
Was a water analysis done? ☐ Yes By whom
Did any strata contain water not suitable for intended use? ☐ Too little
☐ Salty ☐ Muddy ☐ Odor ☐ Colored ☐ Other
Depth of strata:

(9) LOCATION OF WELL by legal description:
County Marion Latitude Longitude
Township 8S N or S. Range 3W E or W. WM.
Section 28 NE ¼ NE ¼
500 Tax Lot ~~400~~ Lot Block Subdivision
Street Address of Well (or nearest address) Same

per Driller 8/19/96

(10) STATIC WATER LEVEL:
112 ft. below land surface. Date 3-20-92
Artesian pressure lb. per square inch. Date

(11) WATER BEARING ZONES:
Depth at which water was first found

From	To	Estimated Flow Rate	SWL

(12) WELL LOG:
Ground elevation

Material	From	To	SWL
Soil	0	1	
Red Orange Clay	1	8	
Orange Clay	8	10	
Weatheredout Rock	10	27	
Red Clay	27	32	
Weathered Rock	32	48	
Basalt Rock	48	143	
Honey Cone Rock	143	164	
Basalt Rock	164	283	

ROBINSON DRILLING
WELLS & PUMPS
~~4520 Dallas-Salem Hwy.~~
~~Salem, Ore. 97304~~
~~371-1844~~

Date started 3-16-92 Completed 3-19-92

(unbonded) Water Well Constructor Certification:

I certify that the work I performed on the construction, alteration, or abandonment of this well is in compliance with Oregon well construction standards. Materials used and information reported above are true to my best knowledge and belief.

WWC Number
Signed Date

(bonded) Water Well Constructor Certification:

I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. all work performed during this time is in compliance with Oregon well construction standards. This report is true to the best of my knowledge and belief.

WWC Number OCL13
Signed George Robinson Date 3-24-92

MARI 58542
5854V

STATE OF OREGON
MONITORING WELL REPORT
(as required by ORS 537.765 & OAR 690-240-095)
Instructions for completing this report are on the last page of this form.

Well ID# L73355
Start Card # 160790

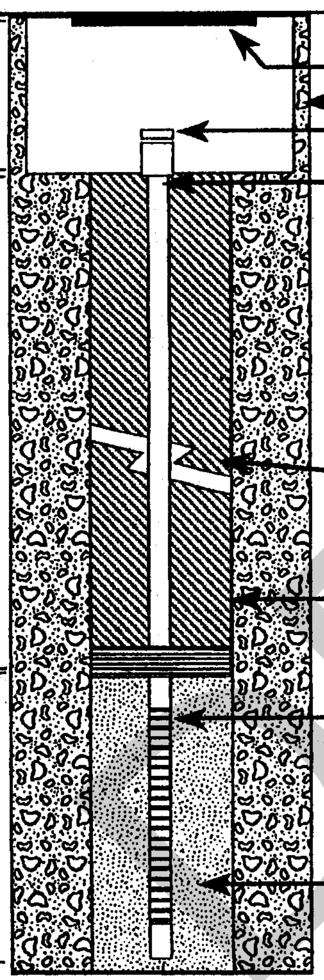
(1) OWNER/PROJECT
WELL NO. P-1
Name Edwin Cammack
Address 411 Hylo Road SE
City Salem State OR Zip 97306

(2) TYPE OF WORK
☒ New construction ☐ Alteration (Repair/Recondition)
☐ Conversion ☐ Deepening ☐ Abandonment

(3) DRILLING METHOD
☐ Rotary Air ☒ Rotary Mud ☐ Cable
☐ Hollow Stem Auger ☐ Other Piezometer

(4) BORE HOLE CONSTRUCTION:
Special Standards Yes ☐ No ☒
Depth of Completed Well 25 ft.

Vault 0 ft. TO 1 ft.
Seal 1 ft. TO 13 ft.
Filter pack 13 ft. TO 25 ft.



Casing diameter 1 in.
material PVC
Welded ☐ Threaded ☐ Glued ☒
Liner diameter ____ in.
material ____
Welded ☐ Threaded ☐ Glued ☐
Well seal:
Material Bentonite
Amount 4 bags
Grout weight ____
Borehole diameter 5 in.
Bentonite plug at least 3 ft. thick
Screen material PVC
interval(s):
From 15 To 25
From ____ To ____
Slot size Field in.
Filter pack:
Material Sand
Size 10x20 in.

(5) WELL TESTS:
☐ Pump ☐ Bailer ☐ Air ☐ Flowing Artesian
Permeability ____ Yield ____ GPM
Conductivity ____ PH ____
Temperature of water NOT °F/C Depth artesian flow found ____ ft.
Was water analysis done? ☐ Yes ☒ OBSERVED
By whom? ____
Depth of strata to be analyzed. From ____ ft. to ____ ft.
Remarks: ____
Name of supervising Geologist/Engineer ____

(6) LOCATION OF WELL By legal description:
County Marion Latitude ____ Longitude ____
Township 8 (N or [✓ S]) Range 3 (E or [✓ W]) Section 28
SE 1/4 of NE 1/4 of above section.
Street address of well location West of Champion Hill Rd SE
±800' North of Intersection of Hylo Rd SE
Tax lot number of well location 230
ATTACH MAP WITH LOCATION IDENTIFIED. Map shall include approximate scale and north arrow.

(7) STATIC WATER LEVEL:
NOT OBSERVED
____ Ft. below land surface Date ____
Artesian Pressure ____ lb/sq. in. Date ____

(8) WATER BEARING ZONES:
Depth at which water was first found ____

From	To	Est. Flow Rate	SWL
NOT OBSERVED			

(9) WELL LOG:
Ground Elevation ____

Material	From	To	SWL
Silts	0	25	

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SALEM, OREGON

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MAR 11 2005
WATER RESOURCES DEPT
SALEM, OREGON

Date started 11/10/04 Completed 11/10/04

(unbonded) Monitor Well Constructor Certification:

I certify that the work I performed on the construction, alteration, or abandonment of this well is in compliance with Oregon water supply well construction standards. Materials used and information reported above are true to the best of my knowledge and belief.

Signed Carlos Anguiano MWC Number 10500 Date 12/1/04

(bonded) Monitor Well Constructor Certification:

I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. All work performed during this time is in compliance with Oregon water supply well construction standards. This report is true to the best of my knowledge and belief.

Signed MWC Number 10442 Date 12/1/04

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MARI 58543

MARI 58543

STATE OF OREGON

GEOTECHNICAL HOLE REPORT

(as required by OAR 690-240-035)

(1) OWNER/PROJECT: Hole Number B-3

Name Edwin Cammack

Address 411 Hylo Road SE

City Salem, State OR Zip 97306

(2) TYPE OF WORK

☒ New ☐ Deepening ☐ Alteration (repair/recondition) ☒ Abandonment

(3) CONSTRUCTION:

☐ Rotary Air ☐ Hand Auger ☐ Hollow Stem Auger

☒ Rotary Mud ☐ Cable Tool ☐ Push Probe ☐ Other

(4) TYPE OF HOLE:

☒ Uncased Temporary ☐ Cased Permanent

☐ Uncased Permanent ☐ Slope Stability ☐ Other

(5) USE OF HOLE:

Geotechnical Study

(6) BORE HOLE CONSTRUCTION:

Special Construction approval ☐ Yes ☒ No Depth of Completed Hole 65 ft.

HOLE			SEAL			
Diameter	From	To	Material	From	To	Sacks or pounds
5	0	65		X		

Backfill placed from ____ ft. to ____ ft. Material ____

Filter Pack placed from ____ ft. to ____ ft. Size of pack ____

(7) CASING/SCREEN:

	Diameter	From	To	Gauge	Steel	Plastic	Welded	Threaded
Casing:					☐	☐	☐	☐
					☐	☐	☐	☐
				X	☐	☐	☐	☐
					☐	☐	☐	☐
Screen:					☐	☐	☐	☐
					☐	☐	☐	☐

Slot size

(8) WELL TEST:

☐ Pump ☐ Bailer ☐ Air ☐ Flowing Artesian

Permeability ____ Yield ____ GPM ____

Conductivity ____ PH ____

Temperature of water ____ °F/C Depth artesian flow found ____ ft.

Was water analysis done? ☐ Yes ☒ No

By whom?

Depth of strata analyzed. From

Remarks:

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WATER RESOURCES DEPT
SALEM, OREGON

(9) LOCATION OF HOLE by legal description:

County Marion Latitude ____ Longitude ____

Township 8 N or [✓ S] Range 3 E or [✓ W]. WM.

Section 28 SE 1/4 NE 1/4

Tax Lot 230 Lot ____ Block ____ Subdivision ____

Street Address of Well (or nearest address) West of Champion Hill Rd SE ± 800' North of Intersection with Hylo Rd SE

Map with location identified must be attached

(10) STATIC WATER LEVEL:

X ft. below land surface. Date ____

Artesian pressure ____ lb. per square inch. Date ____

(11) SUBSURFACE LOG:

Ground Elevation ____

Material Description	From	To	SWL
Silts	0	63	
Decomposed Basalts	63	65	

Date Started 11/10/04 Date Completed 11/10/04

(12) ABANDONMENT LOG:

Material Description	From	To	Sacks or Pounds
Bentonite	0	65	23 bags

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WATER RESOURCES DEPT
SALEM, OREGON

Date started 11/10/04 Date Completed 11/10/04

Professional Certification

(to be signed by a licensed water supply or monitoring well constructor, or Oregon registered geologist or civil engineer).

I accept responsibility for the construction, alteration, or abandonment work performed during the construction dates reported above. All work performed during this time is in compliance with Oregon's geotechnical hole construction standards. This report is true to the best of my knowledge and belief.

License or Registration Number 10500

Signed Carlos Anguiano Date 12/1/04

Affiliation

THIS REPORT MUST BE SUBMITTED TO THE WATER RESOURCES DEPARTMENT WITHIN 30 DAYS OF COMPLETION OF WORK

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MARI 58544

MARI 58544

STATE OF OREGON
GEOTECHNICAL HOLE REPORT
(as required by OAR 690-240-035)

(1) OWNER/PROJECT: Hole Number B-2

Name Edwin Cammack

Address 411 Hylo Road SE

City Salem State OR Zip 97306

(2) TYPE OF WORK

☒ New ☐ Deepening ☐ Alteration (repair/recondition) ☒ Abandonment

(3) CONSTRUCTION:

☐ Rotary Air ☐ Hand Auger ☐ Hollow Stem Auger

☒ Rotary Mud ☐ Cable Tool ☐ Push Probe ☐ Other

(4) TYPE OF HOLE:

☒ Uncased Temporary ☐ Cased Permanent

☐ Uncased Permanent ☐ Slope Stability ☐ Other

(5) USE OF HOLE:

Geotechnical Study

(6) BORE HOLE CONSTRUCTION:

Special Construction approval ☐ Yes ☒ No Depth of Completed Hole 50 ft.

HOLE			SEAL			
Diameter	From	To	Material	From	To	Sacks or pounds
5	0	50				

Backfill placed from ____ ft. to ____ ft. Material ____

Filter Pack placed from ____ ft. to ____ ft. Size of pack ____

(7) CASING/SCREEN:

	Diameter	From	To	Gauge	Steel	Plastic	Welded	Threaded
Casing:					☐	☐	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
Screen:					☐	☐	☐	☐
					☐	☐	☐	☐

Slot size ____

(8) WELL TEST:

☐ Pump ☐ Bailer ☐ Air ☐ Flowing Artesian

Permeability ____ Yield ____ GPM ____

Conductivity ____ PH ____

Temperature of water ____ °F/C Depth artesian flow found ____ ft.

Was water analysis done? ☐ Yes ☒ No

By whom? ____

Depth of strata analyzed. From ____ To ____

Remarks: ____

RECEIVED

MAR 11 2005

WATER RESOURCES DEPT

SALEM, OREGON

(9) LOCATION OF HOLE by legal description:

County Marion Latitude ____ Longitude ____

Township 8 N or [✓ S] Range 3 E or [✓ W]. WM.

Section 28 SE 1/4 NE 1/4

Tax Lot 230 Lot ____ Block ____ Subdivision ____

Street Address of Well (or nearest address) West of Champion Hill Rd SE I 800' North of Intersection Hylo Rd SE

Map with location identified must be attached

(10) STATIC WATER LEVEL:

____ ft. below land surface. Date ____

Artesian pressure ____ lb. per square inch. Date ____

(11) SUBSURFACE LOG:

Ground Elevation ____

Material Description	From	To	SWL
Silts	0	40	
Decomposed Basalts	40	50	

Date Started 11/9/04 Date Completed 11/9/04

(12) ABANDONMENT LOG:

Material Description	From	To	Sacks or Pounds
Bentonite	0	50	20 bags

RECEIVED

DEC 02 2004

WATER RESOURCES DEPT

SALEM, OREGON

Date started [illegible] Date Completed 11/1/04

Professional Certification

(to be signed by a licensed water supply or monitoring well constructor, or Oregon registered geologist or civil engineer).

I accept responsibility for the construction, alteration, or abandonment work performed during the construction dates reported above. All work performed during this time is in compliance with Oregon's geotechnical hole construction standards. This report is true to the best of my knowledge and belief.

License or Registration Number 10500

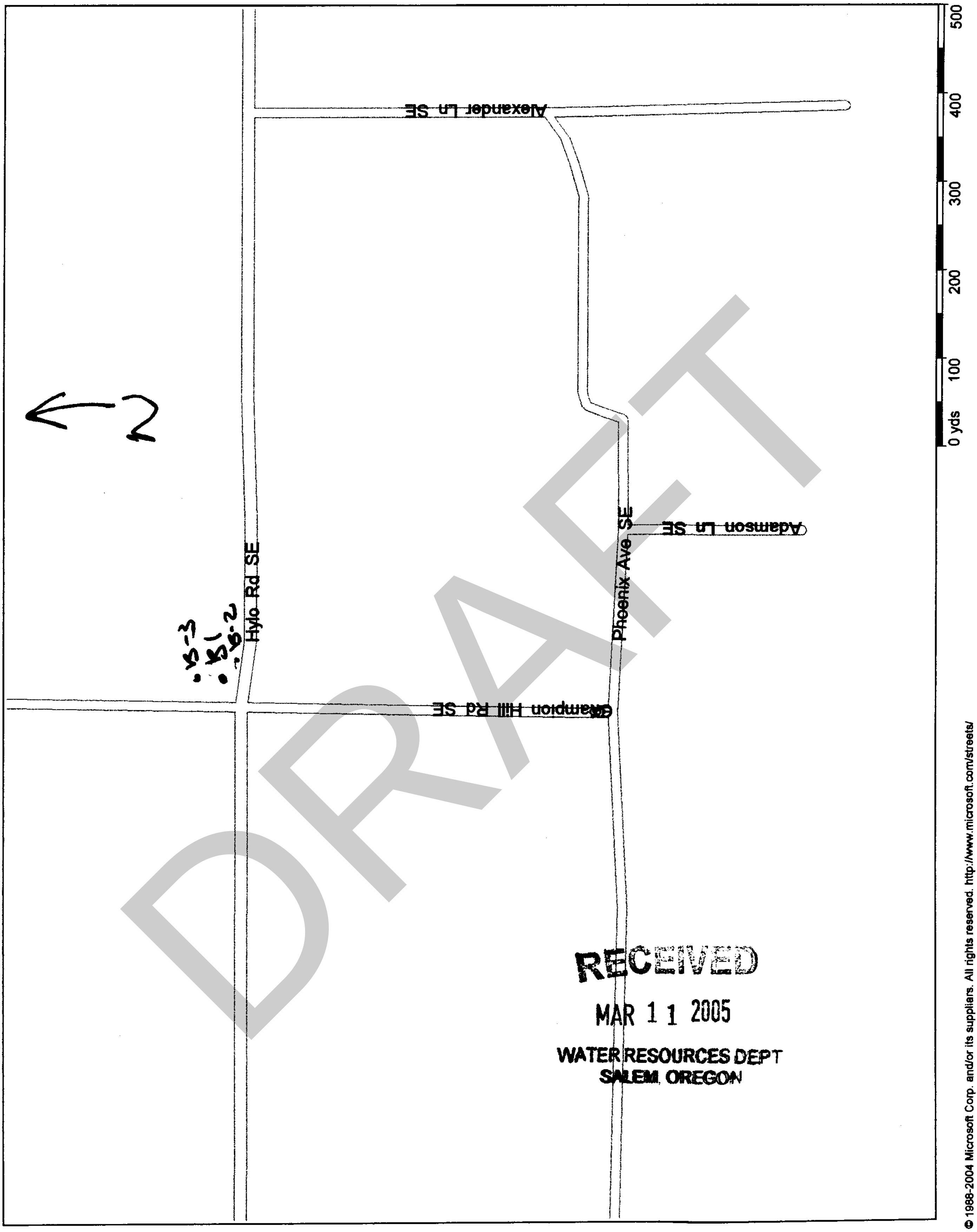
Signed Carlos Anguiano Date 12/1/04

Affiliation ____

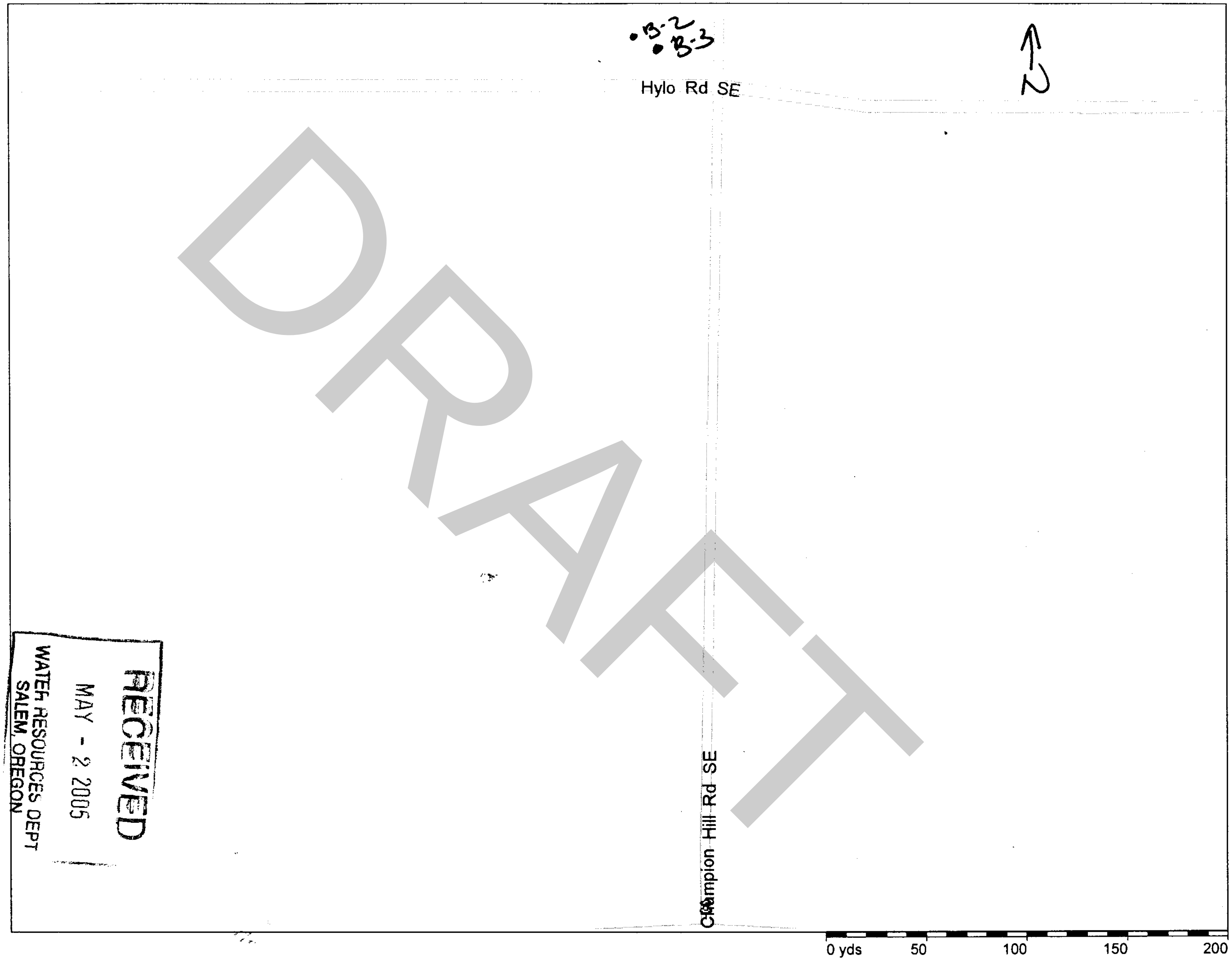
THIS REPORT MUST BE SUBMITTED TO THE WATER RESOURCES DEPARTMENT WITHIN 30 DAYS OF COMPLETION OF WORK

ORIGINAL – WATER RESOURCES DEPARTMENT FIRST COPY – CONSTRUCTOR SECOND COPY – CUSTOMER

Oregon, United States, North America



Oregon, United States, North America



APPENDIX I

EXISTING INFORMATION
SITE 18 - DEER PARK
PUMP STATION

STATE OF OREGON

MONITORING WELL REPORT

(as required by ORS 537.765 & OAR 690-240-095)

MARI 53734

Instructions for completing this report are on the last page of this form.

Received Date **01/04/1999**
Well ID Tag# **L 29739**
Start Card # **117263**

(1) OWNER/PROJECT

Well No. **29739**
Co Job No. **2422**

Name **OREGON DEPARTMENT OF CORRECTIONS**
Street **2575 CENTER ST NE**
City **SALEM** State **OR** Zip **97310**

(2) TYPE OF WORK

☒ New Construction ☐ Alter (Recondition) ☐ Alter (Repair)
☐ Conversion ☐ Deepening ☐ Abandonment

(3) DRILLING METHOD

☐ Rotary Air ☐ Rotary Mud ☐ Cable
☒ Hollow Stem Auger Other *****

(4) BORE HOLE CONSTRUCTION

Special Standards ☐ Depth of completed well **20** ft.

Diameter	From	To
10.00	0.00	20

Material	Begin Depth	End Depth	Material Amount	Units
Concrete	0.00	1.00	2.00	S
Bentonite	1.00	8.00	6.00	S

Vault
0 ft.
1 TO
ft.

Casing Diameter Liner ☐

Monument
ft.
TO
ft.

Casing or Liner	Diameter	Begin Depth	End Depth	Gauge	Material	Construction Weld	Construction Threaded	Location Of Shoe
C	2.00				Plastic	☐	☒	

Seal
ft.
TO
ft.

From	To	Material	Amount	Seal Grout Weight	Units
0.00	1.00	Concrete	2.00		S
1.00	8.00	Bentonite	6.00		S

Filter Pack
8 ft.
TO
20 ft.

Screen ☐

Diameter	From	To	Gauge	Material	Type	Slot Size
	10	20		PL		.010

Filter Pack
Material SA
Size 20.00 in.

(5) WELL TEST

Permeability Yield
Conductivity PH
Temperature of water **53** °F/C Depth artesian flow found ft.
Was water analysis done? ☒
By Whom? **PBS ENVIRONNENTAL**
Depth of strata to be analyzed. From ft. to ft.
Remarks

Name of supervising Geologist/Engineer

(6) LOCATION OF WELL By legal description

County
Township **8.00 S** Range **2.00 W** Section **18**
1. **SE** 1/4 of **NE** 1/4 of above section.
Legal Desc:

2. Either Street address of well location
5485 TURNER RD SE
or Tax lot number of well location **100**

3. ATTACH MAP WITH LOCATION IDENTIFIED. Map shall include approximate scale and north arrow.

(7) STATIC WATER LEVEL

15.0 Ft. below land surface. Date **/15/1998**
Artesian Pressure lb/sq. in. Date

(8) WATER BEARING ZONES

Depth at which water was first found **15** ft.

From	To	Est. Flow Rate	SWL
15	20		15

(9) WELL LOG

Ground elevation ft.

Material	From	To	SWL
SILTY CLAY	0	15	15
SANDY CLAY	15	20	

Date started **12/10/1998** Completed **12/10/1998**

(unbonded) Monitor Well Constructor Certification:

I certify that the work I performed on the construction, alteration, or abandonment of this well is in compliance with Oregon well construction standards. Materials used and information reported above are true to the best knowledge and belief.

MWC Number **10440**
Signed By **PABLO ARMANDO** Date

(bonded) Monitor Well Constructor Certification:

I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. All work performed during this time is in compliance with Oregon well construction standards. This report is true to the best of my knowledge and belief.

MWC Number **10011**
Signed By **GREG MCINNIS** Date

STATE OF OREGON

MONITORING WELL REPORT

(as required by ORS 537.765 & OAR 690-240-095)

MARI 53735

Instructions for completing this report are on the last page of this form.

Received Date 01/04/1999
Well ID Tag# L 29740
Start Card # 117264

(1) OWNER/PROJECT

Well No. 29740
Co Job No. 2422

Name OREGON DEPARTMENT OF CORRECTIONS
Street 2575 CENTER ST NE
City SALEM State OR Zip 97310

(2) TYPE OF WORK

☒ New Construction ☐ Alter (Recondition) ☐ Alter (Repair)
☐ Conversion ☐ Deepening ☐ Abandonment

(3) DRILLING METHOD

☐ Rotary Air ☐ Rotary Mud ☐ Cable
☒ Hollow Stem Auger Other *****

(4) BORE HOLE CONSTRUCTION

Special Standards ☐ Depth of completed well 23 ft.

Diameter	From	To
10.00	0.00	23

Material	Begin Depth	End Depth	Material Amount	Units
Concrete	0.00	1.00	2.00	S
Bentonite	1.00	11.00	7.00	S

Vault
0 ft.
1 TO
ft.

Casing Diameter Liner ☐

Monument
ft.
TO
ft.

Casing or Liner	Diameter	Begin Depth	End Depth	Gauge	Material	Construction Weld	Threaded	Location Of Shoe
C	2.00				Plastic		☒	

Seal
ft.
TO
ft.

From	To	Material	Amount	Seal Grout Weight	Units
0.00	1.00	Concrete	2.00		S
1.00	11.00	Bentonite	7.00		S

Filter Pack Screen ☐
11 ft.
TO
23 ft.

Diameter	From	To	Gauge	Material	Type	Slot Size
	13	23		PL		.010

Filter Pack
Material SA
Size 20.00 in.

(5) WELL TEST

Permeability Yield
Conductivity PH
Temperature of water 53 °F/C Depth artesian flow found ft.
Was water analysis done? ☒
By Whom? PBS ENVIRONMENTAL
Depth of strata to be analyzed. From ft. to ft.
Remarks

Name of supervising Geologist/Engineer

(6) LOCATION OF WELL By legal description

County
Township 8.00 S Range 2.00 W Section 18
1. SE 1/4 of NE 1/4 of above section.
Legal Desc:

2. Either Street address of well location
5485 TURNER RD SE
or Tax lot number of well location 100
3. ATTACH MAP WITH LOCATION IDENTIFIED. Map shall include approximate scale and north arrow.

(7) STATIC WATER LEVEL

18.0 Ft. below land surface. Date /10/1998
Artesian Pressure lb/sq. in. Date

(8) WATER BEARING ZONES

Depth at which water was first found 18 ft.

From	To	Est. Flow Rate	SWL
18	23		18

(9) WELL LOG

Ground elevation ft.

Material	From	To	SWL
SILTY CLOAY	0	16	
GRAVELY CLAY	16	23	18

Date started 12/10/1998 Completed 12/10/1998

(unbonded) Monitor Well Constructor Certification:

I certify that the work I performed on the construction, alteration, or abandonment of this well is in compliance with Oregon well construction standards. Materials used and information reported above are true to the best knowledge and belief.

MWC Number 10440
Signed By PABLO ARMANDO Date

(bonded) Monitor Well Constructor Certification:

I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. All work performed during this time is in compliance with Oregon well construction standards. This report is true to the best of my knowledge and belief.

MWC Number 10011
Signed By GREG MCINNIS Date

STATE OF OREGON
MONITORING WELL REPORT
(as required by ORS 537.765 & OAR 690-240-095)

MARI 53736

Instructions for completing this report are on the last page of this form.

Received Date **01/04/1999**
Well ID Tag# **L 29741**
Start Card # **117265**

(1) OWNER/PROJECT

Well No. **29741**
Co Job No. **2422**

Name **OREGON DEPARTMENT OF CORRECTIONS**
Street **2575 CENTER ST NE**
City **SALEM** State **OR** Zip **97310**

(2) TYPE OF WORK

☒ New Construction ☐ Alter (Recondition) ☐ Alter (Repair)
☐ Conversion ☐ Deepening ☐ Abandonment

(3) DRILLING METHOD

☐ Rotary Air ☐ Rotary Mud ☐ Cable
☒ Hollow Stem Auger Other *****

(4) BORE HOLE CONSTRUCTION

Special Standards ☐ Depth of completed well **16** ft.

Diameter	From	To
10.00	0.00	16

Material	Begin Depth	End Depth	Material Amount	Units
Concrete	0.00	1.00	2.00	S
Bentonite	1.00	4.00	3.00	S

Vault
0 ft.
1 TO
ft.

Casing Diameter Liner ☐

Monument
ft.
TO
ft.

Casing or Liner	Diameter	Begin Depth	End Depth	Gauge	Material	Construction Weld	Construction Threaded	Location Of Shoe
C	2.00				Plastic	☐	☒	

Seal
ft.
TO
ft.

From	To	Material	Amount	Seal Grout Weight	Units
0.00	1.00	Concrete	2.00		S
1.00	4.00	Bentonite	3.00		S

Filter Pack
4 ft.
TO
16 ft.

Screen ☐

Diameter	From	To	Gauge	Material	Type	Slot Size
	6	16		PL		.010

Filter Pack
Material **SA**
Size **20.00** in.

(5) WELL TEST

Permeability Yield
Conductivity PH
Temperature of water **53** °F/C Depth artesian flow found ft.
Was water analysis done? ☒
By Whom? **PBS ENVIRONMENTAL**
Depth of strata to be analyzed. From ft. to ft.
Remarks

Name of supervising Geologist/Engineer

(6) LOCATION OF WELL By legal description

County
Township **8.00 S** Range **2.00 W** Section **18**
1. **SE** 1/4 of **NE** 1/4 of above section.
Legal Desc:

2. Either Street address of well location
5485 TURNER RD SE
or Tax lot number of well location **100**

3. ATTACH MAP WITH LOCATION IDENTIFIED. Map shall include approximate scale and north arrow.

(7) STATIC WATER LEVEL

15.0 Ft. below land surface. Date **/10/1998**
Artesian Pressure lb/sq. in. Date

(8) WATER BEARING ZONES

Depth at which water was first found **15** ft.

From	To	Est. Flow Rate	SWL
15	16		15

(9) WELL LOG

Ground elevation ft.

Material	From	To	SWL
SILTY CLAY	0	12	
CLAY	12	16	15

Date started **12/10/1998** Completed **12/10/1998**

(unbonded) Monitor Well Constructor Certification:

I certify that the work I performed on the construction, alteration, or abandonment of this well is in compliance with Oregon well construction standards. Materials used and information reported above are true to the best knowledge and belief.

MWC Number **10440**
Signed By **PABLO ARMANDO** Date

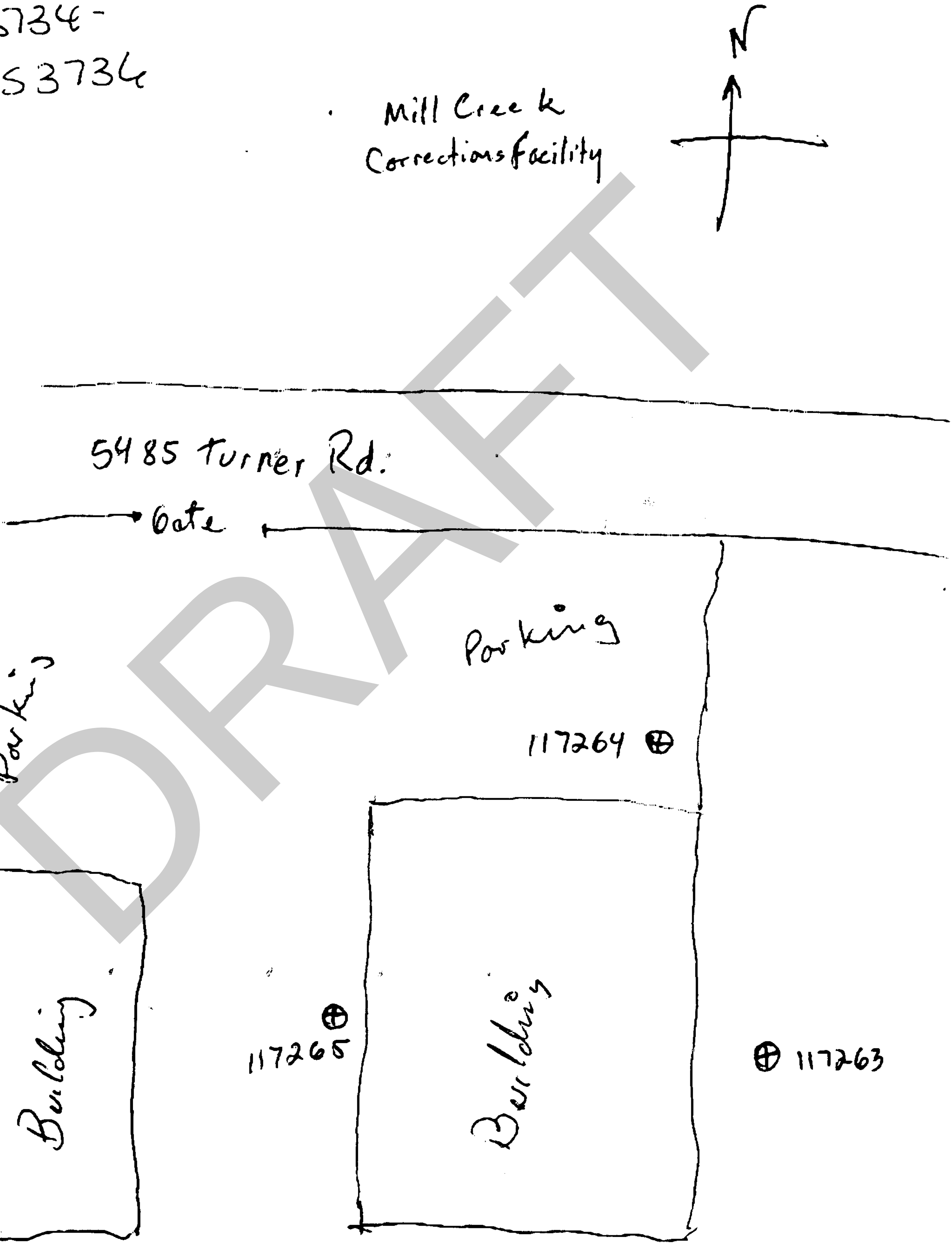
(bonded) Monitor Well Constructor Certification:

I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. All work performed during this time is in compliance with Oregon well construction standards. This report is true to the best of my knowledge and belief.

MWC Number **10011**
Signed By **GREG MCINNIS** Date

Marion
S3734-
S3734

SITE MAP



STATE OF OREGON

GEOTECHNICAL HOLE REPORT

MARI 53757

Received date 01/15/1999

(as required by OAR 690-240-035)

(1) OWNER/PROJECT

Hole No.
Co.Job No. **B-1**

Name **OREGON DEPARTMENT OF CORRECTION**
Street **2575 CENTER ST NE**
City **SALEM** State OR Zip 97310

(2) TYPE OF WORK

☒ New ☐ Alter (Recondition) ☐ Alter (Repair)
☐ Deepening ☒ Abandonment

(3) CONSTRUCTION

☐ Rotary Air ☐ Hand Auger ☐ Hollow Stem Auger
☐ Rotary Mud ☐ Cable Tool ☒ Push Probe Other

(4) TYPE OF HOLE

☒ Uncased Temporary ☐ Cased Permanent
☐ Uncased Permanent ☐ Slope Stability Other

(5) USE OF HOLE

SOIL COLLECTION

(6) BORE HOLE CONSTRUCTION

Special Standards ☐ Depth of completed well **16** ft.

HOLE

Diameter	From	To
2.00	0.00	16

SEAL

From	To	Material	Amount	Seal Grout Weight	Units
0.00	16.00	Bentonite	22.00		P

Backfill placed from ft. TO ft. Material
Filter pack placed from ft. TO ft. Size in.

(7) CASING/SCREEN

Screen ☐

(8) WELL TEST

Permeability Yield GPM
Conductivity PH
Temperature of water °F/C Depth artesian flow found ft.
Was water analysis done? ☐
By Whom?
Depth of strata to be analyzed. From ft. to ft.
Remarks

Name of supervising Geologist/Engineer

(9) LOCATION OF HOLE By legal description

County **Marion** Latitude Longitude
Township **8.00 S** Range **2.00 W**
Section **18** **SE 1/4** **NE 1/4**
Tax lot **100** Lot Block Subdivision
Legal desc:
Street Address of Well (or nearest address)
5485 TURNER RD SE

MAP with location indentified must be attached

(10) STATIC WATER LEVEL

Ft. below land surface. Date
Artesian Pressure lb/sq. in. Date

(11) SUBSURFACE LOG

Ground Elevation ft.

Material	From	To	SWL
SILTY CLAY	0	10	
BASALT	10	16	

Date started **12/15/1998** Completed **12/15/1998**

(12) ABANDONMENT LOG

Date started Completed

Professional Certification

(to be signed by a licensed water supply or monitoring well constructor, or registered geologist or civil engineer).

I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. All work performed during this time is in compliance with Oregon geotechnical hole construction standards. This report is true to the best of my knowledge and belief.

License or Registration Number **10402**

Signed By **KEITH VIDOS** Date

Affiliation **GEO TECH EXPLORATIONS**

THIS REPORT MUST BE SUBMITTED TO THE WATER RESOURCES DEPARTMENT WITHIN 30 DAYS OF COMPLETION OF WORK

STATE OF OREGON

GEOTECHNICAL HOLE REPORT

(as required by OAR 690-240-035)

MARI 53758

Received date **01/15/1999**

(1) OWNER/PROJECT

Hole No.
Co.Job No. **B-2**

Name **OREGON DEPARTMENT OF CORRECTIONS**
Street **2575 CENTER ST NE**
City **SALEM** State OR Zip 97310

(2) TYPE OF WORK

☒ New ☐ Alter (Recondition) ☐ Alter (Repair)
☐ Deepening ☒ Abandonment

(3) CONSTRUCTION

☐ Rotary Air ☐ Hand Auger ☐ Hollow Stem Auger
☐ Rotary Mud ☐ Cable Tool ☒ Push Probe Other

(4) TYPE OF HOLE

☒ Uncased Temporary ☐ Cased Permanent
☐ Uncased Permanent ☐ Slope Stability Other

(5) USE OF HOLE

SOIL COLLECTION

(6) BORE HOLE CONSTRUCTION

Special Standards ☐ Depth of completed well **16** ft.

HOLE

Diameter	From	To
2.00	0.00	16

SEAL

From	To	Material	Amount	Seal Grout Weight	Units
0.00	16.00	Bentonite	22.00		P

Backfill placed from ft. TO ft. Material
Filter pack placed from ft. TO ft. Size in.

(7) CASING/SCREEN

Screen ☐

(8) WELL TEST

Permeability Yield GPM
Conductivity PH
Temperature of water °F/C Depth artesian flow found ft.
Was water analysis done? ☐
By Whom?
Depth of strata to be analyzed. From ft. to ft.
Remarks

Name of supervising Geologist/Engineer

(9) LOCATION OF HOLE By legal description

County **Marion** Latitude Longitude
Township **8.00 S** Range **2.00 W**
Section **18** **SE 1/4 NE 1/4**
Tax lot **100** Lot Block Subdivision
Legal desc:
Street Address of Well (or nearest address)
5485 TURNER RD SE

MAP with location indentified must be attached

(10) STATIC WATER LEVEL

Ft. below land surface. Date
Artesian Pressure lb/sq. in. Date

(11) SUBSURFACE LOG

Ground Elevation ft.

Material	From	To	SWL
SILTY CLAY	0	6	
WEATHERED BASALT	6	16	

Date started **12/15/1998** Completed **12/15/1998**

(12) ABANDONMENT LOG

Date started Completed

Professional Certification

(to be signed by a licensed water supply or monitoring well constructor, or registered geologist or civil engineer).

I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. All work performed during this time is in compliance with Oregon geotechnical hole construction standards. This report is true to the best of my knowledge and belief.

License or Registration Number **10402**

Signed By **KEITH VIDOS** Date

Affiliation **GEO TECH EXPLORATIONS**

THIS REPORT MUST BE SUBMITTED TO THE WATER RESOURCES DEPARTMENT WITHIN 30 DAYS OF COMPLETION OF WORK

STATE OF OREGON

GEOTECHNICAL HOLE REPORT

(as required by OAR 690-240-035)

MARI 53759

Received date **01/15/1999**

(1) OWNER/PROJECT

Hole No.

Co.Job No. **B-3**

Name **OREGON DEPARTMENT OF CORRECTIONS**

Street **2575 CENTER ST NE**

City **SALEM** State **OR** Zip **97310**

(2) TYPE OF WORK

☒ **New** ☐ **Alter (Recondition)** ☐ **Alter (Repair)**

☐ **Deepening** ☒ **Abandonment**

(3) CONSTRUCTION

☐ **Rotary Air** ☐ **Hand Auger** ☐ **Hollow Stem Auger**

☐ **Rotary Mud** ☐ **Cable Tool** ☒ **Push Probe** Other

(4) TYPE OF HOLE

☒ **Uncased Temporary** ☐ **Cased Permanent**

☐ **Uncased Permanent** ☐ **Slope Stability** Other

(5) USE OF HOLE

SOIL COLLECTION

(6) BORE HOLE CONSTRUCTION

Special Standards ☐ Depth of completed well **14** ft.

HOLE

Diameter	From	To
2.00	0.00	16

SEAL

From	To	Material	Amount	Seal Grout Weight	Units
0.00	16.00	Bentonite	22.00		P

Backfill placed from ft. TO ft. Material

Filter pack placed from ft. TO ft. Size in.

(7) CASING/SCREEN

Screen ☐

(8) WELL TEST

Permeability Yield GPM

Conductivity PH

Temperature of water °F/C Depth artesian flow found ft.

Was water analysis done? ☐

By Whom?

Depth of strata to be analyzed. From ft. to ft.

Remarks

Name of supervising Geologist/Engineer

(9) LOCATION OF HOLE By legal description

County **Marion** Latitude Longitude

Township **8.00 S** Range **2.00 W**

Section **18** **SE 1/4 NE 1/4**

Tax lot **100** Lot Block Subdivision

Legal desc:

Street Address of Well (or nearest address)

5485 TURNER RD SE

MAP with location indentified must be attached

(10) STATIC WATER LEVEL

Ft. below land surface. Date

Artesian Pressure **lb/sq. in.** Date

(11) SUBSURFACE LOG

Ground Elevation ft.

Material	From	To	SWL
SILTY CLAY	0	10	
WEATHERED BASALT	10	16	

Date started **12/15/1998** Completed **12/15/1998**

(12) ABANDONMENT LOG

Date started Completed

Professional Certification

(to be signed by a licensed water supply or monitoring well constructor, or registered geologist or civil engineer).

I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. All work performed during this time is in compliance with Oregon geotechnical hole construction standards. This report is true to the best of my knowledge and belief.

License or Registration Number **10402**

Signed By **KEITH VIDOS** Date

Affiliation **GEO TECH EXPLORATIONS**

THIS REPORT MUST BE SUBMITTED TO THE WATER RESOURCES DEPARTMENT WITHIN 30 DAYS OF COMPLETION OF WORK

STATE OF OREGON

GEOTECHNICAL HOLE REPORT

(as required by OAR 690-240-035)

MARI 53760

Received date **01/15/1999**

(1) OWNER/PROJECT

Hole No.

Co.Job No. **B-4**

Name **OREGON DEPARTMENT OF CORRECTIONS**

Street **2575 CENTER ST NE**

City **SALEM** State **OR** Zip **97310**

(2) TYPE OF WORK

☒ New ☐ Alter (Recondition) ☐ Alter (Repair)

☐ Deepening ☒ Abandonment

(3) CONSTRUCTION

☐ Rotary Air ☐ Hand Auger ☐ Hollow Stem Auger

☐ Rotary Mud ☐ Cable Tool ☒ Push Probe Other

(4) TYPE OF HOLE

☒ Uncased Temporary ☐ Cased Permanent

☐ Uncased Permanent ☐ Slope Stability Other

(5) USE OF HOLE

SOIL COLLECTION

(6) BORE HOLE CONSTRUCTION

Special Standards ☐ Depth of completed well **16** ft.

HOLE

Diameter	From	To
2.00	0.00	16

SEAL

From	To	Material	Amount	Seal Grout Weight	Units
0.00	16.00	Bentonite	22.00		P

Backfill placed from ft. TO ft. Material

Filter pack placed from ft. TO ft. Size in.

(7) CASING/SCREEN

Screen ☐

(8) WELL TEST

Permeability Yield GPM

Conductivity PH

Temperature of water °F/C Depth artesian flow found ft.

Was water analysis done? ☐

By Whom?

Depth of strata to be analyzed. From ft. to ft.

Remarks

Name of supervising Geologist/Engineer

(9) LOCATION OF HOLE By legal description

County **Marion** Latitude Longitude

Township **8.00 S** Range **2.00 W**

Section **18** **SE 1/4 NE 1/4**

Tax lot Lot Block Subdivision

Legal desc:

Street Address of Well (or nearest address)

5485 TURNER RD SE

MAP with location indentified must be attached

(10) STATIC WATER LEVEL

Ft. below land surface. Date

Artesian Pressure lb/sq. in. Date

(11) SUBSURFACE LOG

Ground Elevation ft.

Material	From	To	SWL
SILTY CLAY	0	6	
WEATHERED BASALT	6	16	

Date started **12/15/1998** Completed **12/16/1998**

(12) ABANDONMENT LOG

Date started Completed

Professional Certification

(to be signed by a licensed water supply or monitoring well constructor, or registered geologist or civil engineer).

I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. All work performed during this time is in compliance with Oregon geotechnical hole construction standards. This report is true to the best of my knowledge and belief.

License or Registration Number **10402**

Signed By **KEITH VIDOS** Date

Affiliation **GEO TECH EXPLORATIONS**

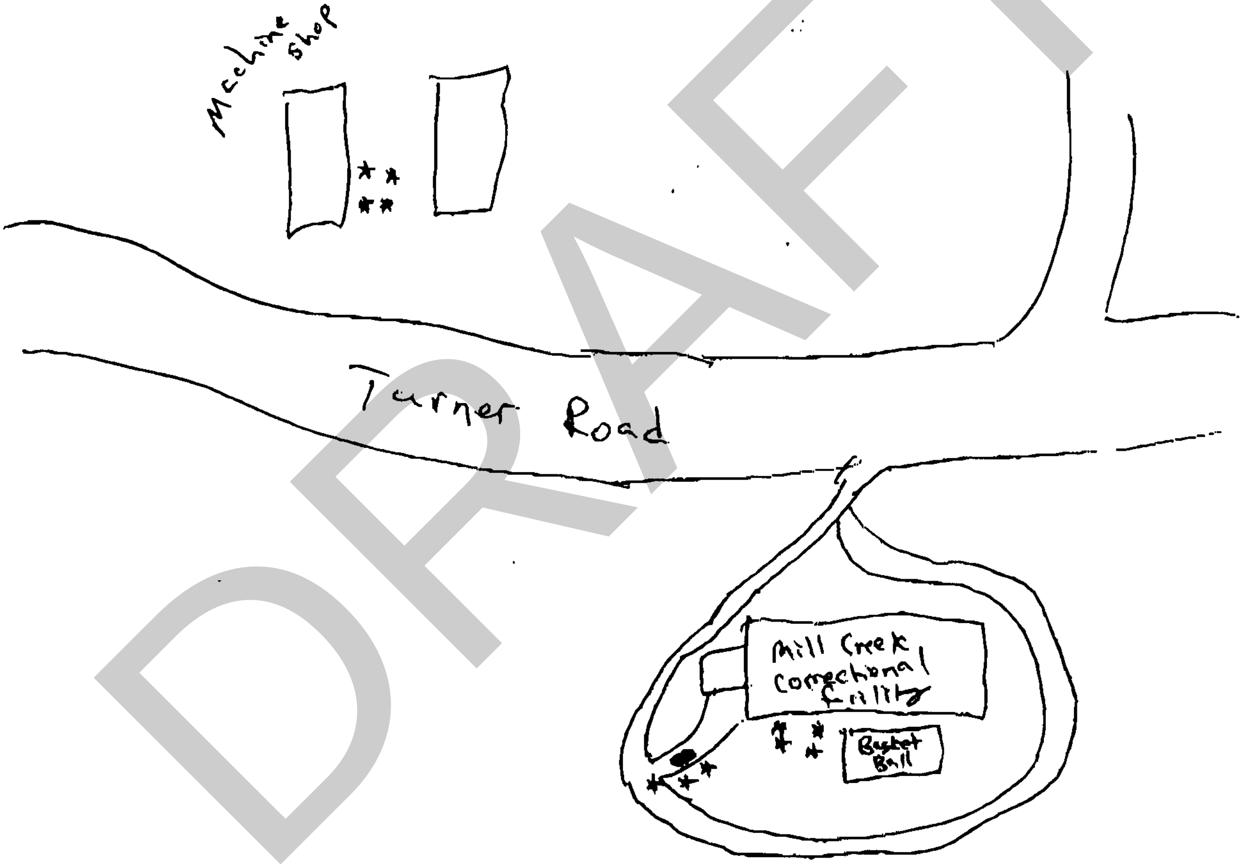
THIS REPORT MUST BE SUBMITTED TO THE WATER RESOURCES DEPARTMENT WITHIN 30 DAYS OF COMPLETION OF WORK

SITE MAP

Marian
53757-
53760

N

not to scale



APPENDIX J

EXISTING INFORMATION
SITE 19 - MILL CREEK RESERVOIR

MARI 59203

STATE OF OREGON
WATER SUPPLY WELL REPORT
(as required by ORS 537.765)
Instructions for completing this report are on the last page of this form.

OWRD RECEIVED OVER THE COUNTER

(WELL I.D.) # L 78556
(START CARD) # 175952

(1) OWNER: Well Number 1
Name Eugene Arnautov
Address 3280 Cooke St S
City Salem State OR Zip 97302

(2) TYPE OF WORK
☑ New Well ☐ Deepening ☐ Alteration (repair/recondition) ☐ Abandonment

(3) DRILL METHOD:
☑ Rotary Air ☐ Rotary Mud ☐ Cable ☐ Auger
☐ Other

(4) PROPOSED USE:
☑ Domestic ☐ Community ☐ Industrial ☐ Irrigation
☐ Thermal ☐ Injection ☐ Livestock ☐ Other

(5) BORE HOLE CONSTRUCTION:
Special Construction approval ☐ Yes ☑ No Depth of Completed Well 305' ft.
Explosives used ☐ Yes ☑ No Type Amount

HOLE			SEAL			
Diameter	From	To	Material	From	To	Sacks or pounds
10"	0'	25'	Bentonite	0'	21'	8 Bags
8"	25'	99'	Cement	21'	99'	15 Bags
6"	99'	305'				

How was seal placed: Method ☐ A ☐ B ☐ C ☑ D ☐ E
☐ Other Filled and tamped to top with dry bentonite.
Backfill placed from ft. to ft. Material
Gravel placed from ft. to ft. Size of gravel

(6) CASING/LINER:

	Diameter	From	To	Gauge	Steel	Plastic	Welded	Threaded
Casing:	6"	+1'	99'	250	☑	☐	☑	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
Liner:	4"	0'	260'	160	☐	☑	☑	☐
	4 1/2"	260'	305'	200	☐	☑	☑	☐

Final location of shoe(s)

(7) PERFORATIONS/SCREENS:
☑ Perforations Method Saw cut
☐ Screens Type Material

From	To	Slot size	Number	Diameter	Tele/pipe size	Casing	Liner
240'	305'	1/8"	69	6" long	4 1/2"	☐	☑
						☐	☐
						☐	☐
						☐	☐
						☐	☐

(8) WELL TESTS: Minimum testing time is 1 hour
☐ Pump ☐ Bailer ☑ Air ☐ Flowing Artesian

Yield gal/min	Drawdown	Drill stem at	Time
50 GPM		285'	1 hr.

Temperature of water 54* Depth Artesian Flow Found
Was a water analysis done? ☐ Yes By whom
Did any strata contain water not suitable for intended use? ☐ Too little
☐ Salty ☐ Muddy ☐ Odor ☐ Colored ☐ Other
Depth of strata:

(9) LOCATION OF WELL by legal description:
County Marion Latitude Longitude
Township 8 S Range 2 W WM.
Section 17 SW 1/4 NW 1/4
Tax Lot 1902 Lot Block Subdivision
Street Address of Well (or nearest address) 5583 Jenniches Ln SE
Salem, OR 97301

(10) STATIC WATER LEVEL:
197' ft. below land surface. Date 8/14/2005
Artesian pressure lb. per square inch. Date

(11) WATER BEARING ZONES:
Depth at which water was first found 62'

From	To	Estimated Flow Rate	SWL
62'	72'	5 GPM	43'
264'	284'	50 GPM	197'

(12) WELL LOG:
Ground Elevation

Material	From	To	SWL
Red brown clay	0	16	
Tan brown clay with boulders	16	24	
Rock weathered brown with boulders	24	62	
Rock black basalt broken	62	72	
Rock brown weathered	72	83	
Weathered brown rock w/white clay seams	83	85	
Brown black weathered	85	91	
Rock black basalt hard	91	218	197'
Rock gray basalt hard	218	264	
Black basalt w/white brown clay seams	264	305	
Ron Robinson Well Drilling			
4520 Salem Dallas Hwy NW			
Salem, OR 97304			
503.371.1844 office			

RECEIVED
SEP 15 2005
WATER RESOURCES DEPT
SALEM OREGON

Date started 9/6/2005 Completed 9/14/2005

(unbonded) Water Well Constructor Certification:

I certify that the work I performed on the construction, alteration, or abandonment of this well is in compliance with Oregon water supply well construction standards. Materials used and information reported above are true to the best of my knowledge and belief.

WWC Number
Signed Date

(bonded) Water Well Constructor Certification:

I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. All work performed during this time is in compliance with Oregon water supply well construction standards. This report is true to the best of my knowledge and belief.

WWC Number 1585
Signed Date 9/15/2005

ORIGINAL & FIRST COPY-WATER RESOURCES DEPARTMENT SECOND COPY-CONSTRUCTOR THIRD COPY-CUSTOMER

MARI 65657

STATE OF OREGON
GEOTECHNICAL HOLE REPORT
(as required by OAR 690-240-0035)

3/31/2015

(1) OWNER/PROJECT

Hole Number B1

PROJECT NAME/NBR: 7-184/ODOT-162-01

First Name Last Name
Company OREGON STATE CORRECTIONS DEPARTMENT
Address 2575 CENTER ST.
City SALEM State OR Zip 97301-4667

(2) TYPE OF WORK

☒ New ☐ Deepening ☒ Abandonment
☐ Alteration (repair/recondition)

(3) CONSTRUCTION

☐ Rotary Air ☐ Hand Auger ☐ Hollow stem auger
☒ Rotary Mud ☐ Cable ☐ Push Probe
☐ Other

(4) TYPE OF HOLE:

● Uncased Temporary ○ Cased Permanent
○ Uncased Permanent ○ Slope Stablity
○ Other
Other:

(5) USE OF HOLE

GEOTECHNICAL

(6) BORE HOLE CONSTRUCTION

Special Standard ☐ (Attach copy)

Depth of Completed Hole 45.00 ft.

BORE HOLE			SEAL				
Dia	From	To	Material	From	To	Amt	sacks/ lbs
4	0	45	Bentonite Chips	0	45	6	S

Backfill placed from ft. to ft. Material
Filter pack from ft. to ft. Material Size

(7) CASING/SCREEN

Casing	Screen	Dia	+	From	To	Gauge	Stl	Plstc	Wld	Thrd
○	○		☐				○	○	☐	☐
○	○		☐				○	○	☐	☐
○	○		☐				○	○	☐	☐
○	○		☐				○	○	☐	☐
○	○		☐				○	○	☐	☐

(8) WELL TESTS

○ Pump ○ Bailer ○ Air ○ Flowing Artesian

Yield gal/min	Drawdown	Drill stem/Pump depth	Duration(hr)

Temperature °F Lab analysis ☐ Yes By
Supervising Geologist/Engineer
Water quality concerns? ☐ Yes (describe below) TDS amount

From	To	Description	Amount	Units

(9) LOCATION OF HOLE (legal description)

County MARION Twp 8.00 S N/S Range 2.00 W E/W WM
Sec 17 SW 1/4 of the NW 1/4 Tax Lot 100
Tax Map Number Lot
Lat ° ' " or 44.87861111 DMS or DD
Long ° ' " or -122.96397222 DMS or DD
○ Street address of hole ● Nearest address
5358 DEER PARK DR SE SALEM, OR

(10) STATIC WATER LEVEL

	Date	SWL(psi)	+	SWL(ft)
Existing Well / Predeepening			☐	
Completed Well			☐	

Flowing Artesian? ☐
WATER BEARING ZONES
Depth water was first found

SWL Date	From	To	Est Flow	SWL(psi)	+	SWL(ft)
					☐	
					☐	
					☐	
					☐	
					☐	

(11) SUBSURFACE LOG

Ground Elevation

Material	From	To
Sandy Silt	0	18
Weathered Basalt	18	45

Date Started 3/30/2015 Completed 3/30/2015

(12) ABANDONMENT LOG:

Material	From	To	Amt	sacks/ lbs
Bentonite Chips	0	45	6	S

Date Started 3/30/2015 Completed 3/30/2015

Professional Certification (to be signed by an Oregon licensed water or monitoring well constructor, Oregon registered geologist or professional engineer).

I accept responsibility for the construction, deepening, alteration, or abandonment work performed during the construction dates reported above. All work performed during this time is in compliance with Oregon geotechnical hole construction standards. This report is true to the best of my knowledge and belief.

License/Registration Number 10591 Date 3/31/2015
First Name JEFF Last Name CRISMAN
Affiliation WESTERN STATES SOIL CONSERVATION, INC.

ORIGINAL - WATER RESOURCES DEPARTMENT
THIS REPORT MUST BE SUBMITTED TO THE WATER RESOURCES DEPARTMENT WITHIN 30 DAYS OF COMPLETION OF WORK

Form Version:

MARI 65658

STATE OF OREGON
GEOTECHNICAL HOLE REPORT
(as required by OAR 690-240-0035)

3/31/2015

(1) OWNER/PROJECT

Hole Number B2

PROJECT NAME/NBR: 7-184/ODOT-162-01

First Name Last Name

Company OREGON STATE CORRECTIONS DEPARTMENT

Address 2575 CENTER ST.

City SALEM State OR Zip 97301-4667

(2) TYPE OF WORK

☒ New ☐ Deepening ☒ Abandonment

☐ Alteration (repair/recondition)

(3) CONSTRUCTION

☐ Rotary Air ☐ Hand Auger ☐ Hollow stem auger

☒ Rotary Mud ☐ Cable ☐ Push Probe

☒ Other HQ CORE

(4) TYPE OF HOLE:

● Uncased Temporary ○ Cased Permanent

○ Uncased Permanent ○ Slope Stablity

○ Other

Other:

(5) USE OF HOLE

GEOTECHNICAL

(6) BORE HOLE CONSTRUCTION

Special Standard ☐ (Attach copy)

Depth of Completed Hole 38.00 ft.

BORE HOLE		
Dia	From	To
5	0	33
4	33	38

SEAL				
Material	From	To	Amt	sacks/ lbs
Bentonite Chips	0	38	7	S

Backfill placed from ft. to ft. Material

Filter pack from ft. to ft. Material Size

(7) CASING/SCREEN

Casing	Screen	Dia	+	From	To	Gauge	Stl	Plstc	Wld	Thrd
○	○		☐				○	○	☐	☐
○	○		☐				○	○	☐	☐
○	○		☐				○	○	☐	☐
○	○		☐				○	○	☐	☐
○	○		☐				○	○	☐	☐

(8) WELL TESTS

○ Pump ○ Bailer ○ Air ○ Flowing Artesian

Yield gal/min	Drawdown	Drill stem/Pump depth	Duration(hr)

Temperature °F Lab analysis ☐ Yes By

Supervising Geologist/Engineer

Water quality concerns? ☐ Yes (describe below) TDS amount

From	To	Description	Amount	Units

(9) LOCATION OF HOLE (legal description)

County MARION Twp 8.00 S N/S Range 2.00 W E/W WM

Sec 17 SW 1/4 of the NW 1/4 Tax Lot 100

Tax Map Number Lot

Lat ° ' " or 44.87680556 DMS or DD

Long ° ' " or -122.96461111 DMS or DD

○ Street address of hole ● Nearest address

5358 DEER PARK DR SE SALEM, OR

(10) STATIC WATER LEVEL

	Date	SWL(psi)	+	SWL(ft)
Existing Well / Predeepening			☐	
Completed Well			☐	

Flowing Artesian? ☐

WATER BEARING ZONES

Depth water was first found

SWL Date	From	To	Est Flow	SWL(psi)	+	SWL(ft)
					☐	
					☐	
					☐	
					☐	
					☐	

(11) SUBSURFACE LOG

Ground Elevation

Material	From	To
Sandy Silt	0	9
Weathered Basalt	9	27
Basalt	27	38

Date Started 3/30/2015 Completed 3/30/2015

(12) ABANDONMENT LOG:

Material	From	To	Amt	sacks/ lbs
Bentonite Chips	0	38	7	S

Date Started 3/30/2015 Completed 3/30/2015

Professional Certification (to be signed by an Oregon licensed water or monitoring well constructor, Oregon registered geologist or professional engineer).

I accept responsibility for the construction, deepening, alteration, or abandonment work performed during the construction dates reported above. All work performed during this time is in compliance with Oregon geotechnical hole construction standards. This report is true to the best of my knowledge and belief.

License/Registration Number 10591 Date 3/31/2015

First Name JEFF Last Name CRISMAN

Affiliation WESTERN STATES SOIL CONSERVATION, INC.

ORIGINAL - WATER RESOURCES DEPARTMENT

THIS REPORT MUST BE SUBMITTED TO THE WATER RESOURCES DEPARTMENT WITHIN 30 DAYS OF COMPLETION OF WORK

Form Version:

GEOTECHNICAL HOLE REPORT - Map with location identified must be attached and shall include an approximate scale and north arrow

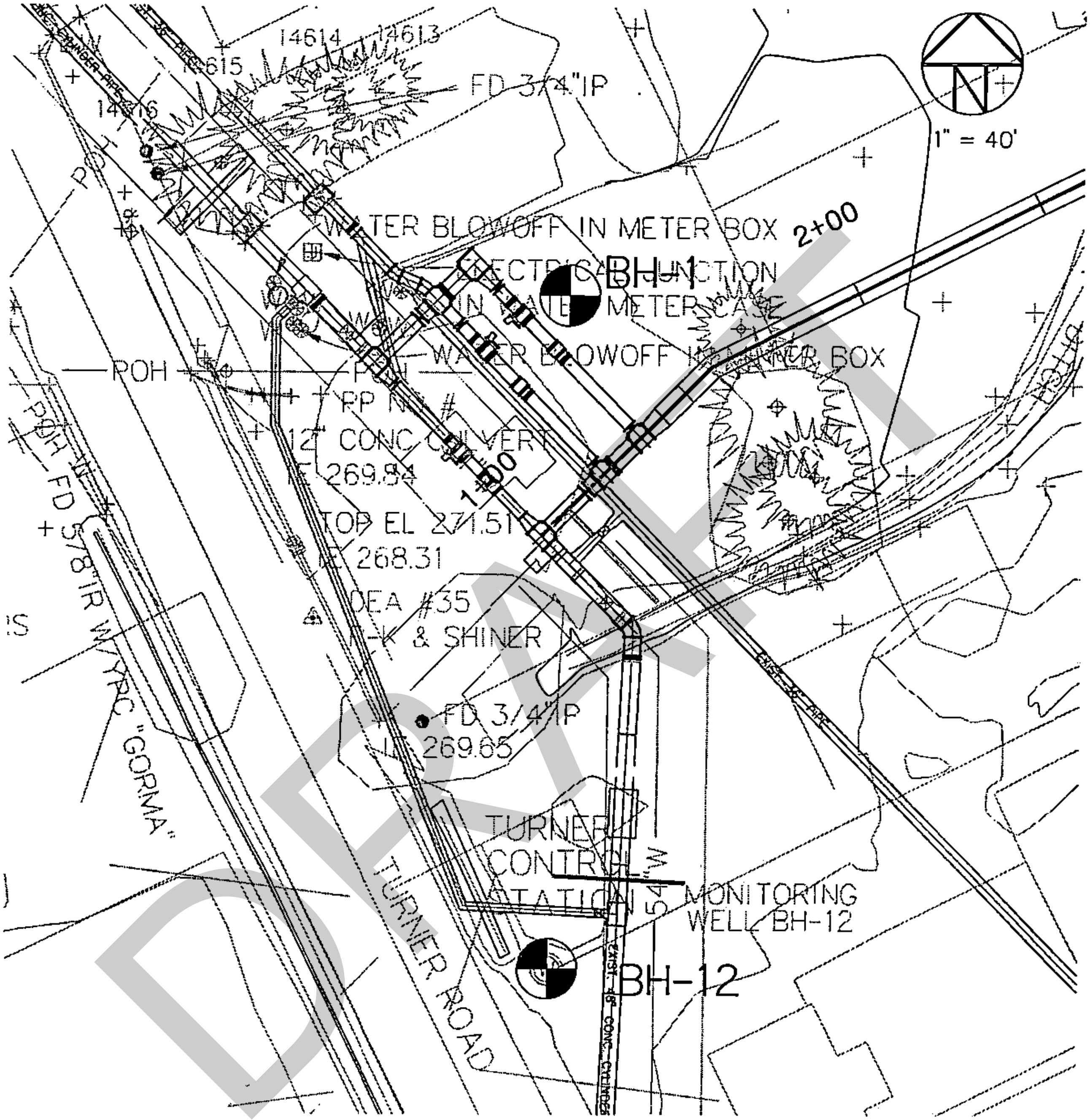
MARI 65658

3/31/2015

Map of Hole

APPENDIX K

EXISTING INFORMATION
SITE 20 – TURNER CONTROL FACILITY



NOTES:
1. BORING LOCATIONS WERE ESTABLISHED BY PACING AND ARE APPROXIMATE ONLY.
2. SEE MEMORANDUM FOR A DISCUSSION OF SUBSURFACE CONDITIONS.
3. BASE MAP WAS PROVIDED BY BLACK & VEATCH CORPORATION.

DATE APR 2005
DWN. DLR
APPR.
REVIS.
PROJECT NO.
2051029

FOUNDATION ENGINEERING INC.
PROFESSIONAL GEOTECHNICAL SERVICES
820 NW CORNELL AVENUE
CORVALLIS, OR 97330-4517
BUS. (541) 757-7645 FAX (541) 757-7650

SITE LAYOUT AND BORING LOCATIONS
TURNER CONTROL STATION
75 MGD TRANSMISSION CONDUIT - PHASE 2
TURNER, OREGON

FIGURE NO.
1A

FILE NAME: FIG 1A

DISTINCTION BETWEEN FIELD LOGS AND FINAL LOGS

A field log is prepared for each boring or test pit by our field representative. The log contains information concerning sampling depths and the presence of various materials such as gravel, cobbles, and fill, and observations of ground water. It also contains our interpretation of the soil conditions between samples. The final logs presented in this report represent our interpretation of the contents of the field logs and the results of the laboratory examinations and tests. Our recommendations are based on the contents of the final logs and the information contained therein and not on the field logs.

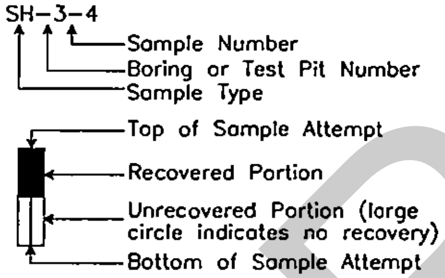
VARIATION IN SOILS BETWEEN TEST PITS AND BORINGS

The final log and related information depict subsurface conditions only at the specific location and on the date indicated. Those using the information contained herein should be aware that soil conditions at other locations or on other dates may differ. Actual foundation or subgrade conditions should be confirmed by us during construction.

TRANSITION BETWEEN SOIL OR ROCK TYPES

The lines designating the interface between soil, fill or rock on the final logs and on subsurface profiles presented in the report are determined by interpolation and are therefore approximate. The transition between the materials may be abrupt or gradual. Only at boring or test pit locations should profiles be considered as reasonably accurate and then only to the degree implied by the notes thereon.

SAMPLE OR TEST SYMBOLS



S – Grab Samples
SS – Standard Penetration Test Sample (split-spoon)
SH – Thin-walled Shelby Tube Sample
C – Core Sample
CS – Continuous Sample

▲ Standard Penetration Test Resistance equals the number of blows a 140 lb. weight falling 30 in. is required to drive a standard split-spoon sampler 1 ft. Practical refusal is equal to 50 or more blows per 6 in. of sampler penetration.

● Water Content (%).

UNIFIED SOIL CLASSIFICATION SYMBOLS

G – Gravel
S – Sand
M – Silt
C – Clay
Pt – Peat

W – Well Graded
P – Poorly Graded
L – Low Plasticity
H – High Plasticity
O – Organic

FIELD SHEAR STRENGTH TEST

Shear strength measurements on test pit side walls, blocks of soil or Shelby tube samples are typically made with Torvane or pocket penetrometer devices.

TYPICAL SOIL/ROCK SYMBOLS

░ Sand
▥ Silt
▨ Clay
▒ Gravel
▓ Basalt
▤ Siltstone

WATER TABLE

▼ Water Table Location
(1/31/00) Date of Measurement
Piezometer Tip Location (if used)

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SYMBOL KEY
BORING AND TEST PIT LOGS

Explanation of Common Terms Used in Soil Descriptions

Field Identification	Cohesive Soils			Granular Soils	
	SPT	S_u^* (tsf)	Term	SPT	Term
Easily penetrated several inches by fist.	0 – 1	< 0.125	Very Soft	0 – 4	Very Loose
Easily penetrated several inches by thumb.	2 – 4	0.125–0.25	Soft	5 – 10	Loose
Can be penetrated several inches by thumb with moderate effort.	5 – 8	0.25 – 0.50	Medium Stiff (Firm)	11 – 30	Medium Dense
Readily indented by thumb but penetrated only with great effort.	9 – 15	0.50 – 1.0	Stiff	31 – 50	Dense
Readily indented by thumbnail.	16 – 30	1.0 – 2.0	Very Stiff	> 50	Very Dense
Indented with difficulty by thumbnail.	31 – 60	> 2.0	Hard		

* Undrained shear strength

Term	Soil Moisture Field Description
Dry	Absence of moisture. Dusty. Dry to the touch.
Damp	Soil has moisture. Cohesive soils are below plastic limit and usually moldable.
Moist	Grains appear darkened, but no visible water. Silt/clay will clump. Sand will bulk. Soils are often at or near plastic limit.
Wet	Visible water on larger grain surfaces. Sand and cohesionless silt exhibit dilatancy. Cohesive silt/clay can be readily remolded. Soil leaves wetness on the hand when squeezed. "Wet" indicates that the soil is wetter than the optimum moisture content and above the plastic limit.

Term	PI	Plasticity Field Test
Nonplastic	0 – 3	Cannot be rolled into a thread.
Low Plasticity	3 – 15	Can be rolled into a thread with some difficulty.
Medium Plasticity	15 – 30	Easily rolled into thread.
High Plasticity	> 30	Easily rolled and rerolled into thread.

Term	Soil Structure Criteria
Stratified	Alternating layers at least 1 inch thick – describe variation.
Laminated	Alternating layers at less than 1 inch thick – describe variation.
Fissured	Contains shears and partings along planes of weakness.
Slickensides	Partings appear glossy or striated.
Blocky	Breaks into lumps – crumbly.
Lensed	Contains pockets of different soils – describe variation.

Term	Soil Cementation Criteria
Weak	Breaks under light finger pressure.
Moderate	Breaks under hard finger pressure.
Strong	Will not break with finger pressure.

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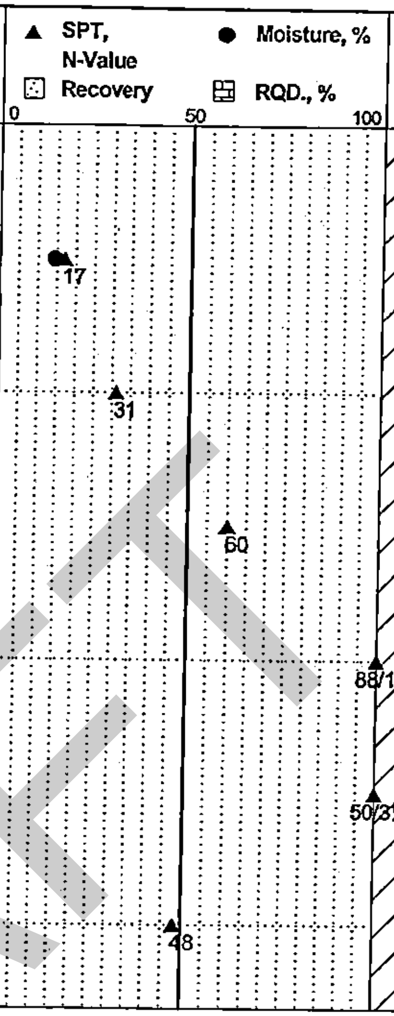
COMMON TERMS
SOIL DESCRIPTIONS

Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples
	Loose CRUSHED GRAVEL; grey, damp to dry, fine to coarse, angular to subrounded, (fill).	0.0 0.2	
1	Very stiff, silty CLAY/clayey SILT, trace organics, sand and gravel; brown, iron-stained, damp, medium plasticity, medium to coarse sand, fine, subrounded gravel, (alluvium).		
2			
3			SS-1-1
4			
5	Dense GRAVEL, some clay and silt, trace sand; brown, iron-stained, damp, medium to coarse sand, fine to coarse, subangular to subrounded gravel, (alluvium).	5.0	SS-1-2
6			
7			
8	Grades to very dense GRAVEL, some sand, trace silt at ±7.5 feet.	7.5	SS-1-3
9			
10			
11			SS-1-4
12			
13			SS-1-5
14			
15			
16	Augering encountered refusal on a cobble or boulder at ±16.6 feet.		SS-1-6
	BOTTOM OF BORING	16.6	



Installations/ Water Table

Backfilled with bentonite chips

Ground water level (3/10/05)

Project No.: 2051029

Surface Elevation: N/A (Approx.)

Date of Boring: March 10, 2005

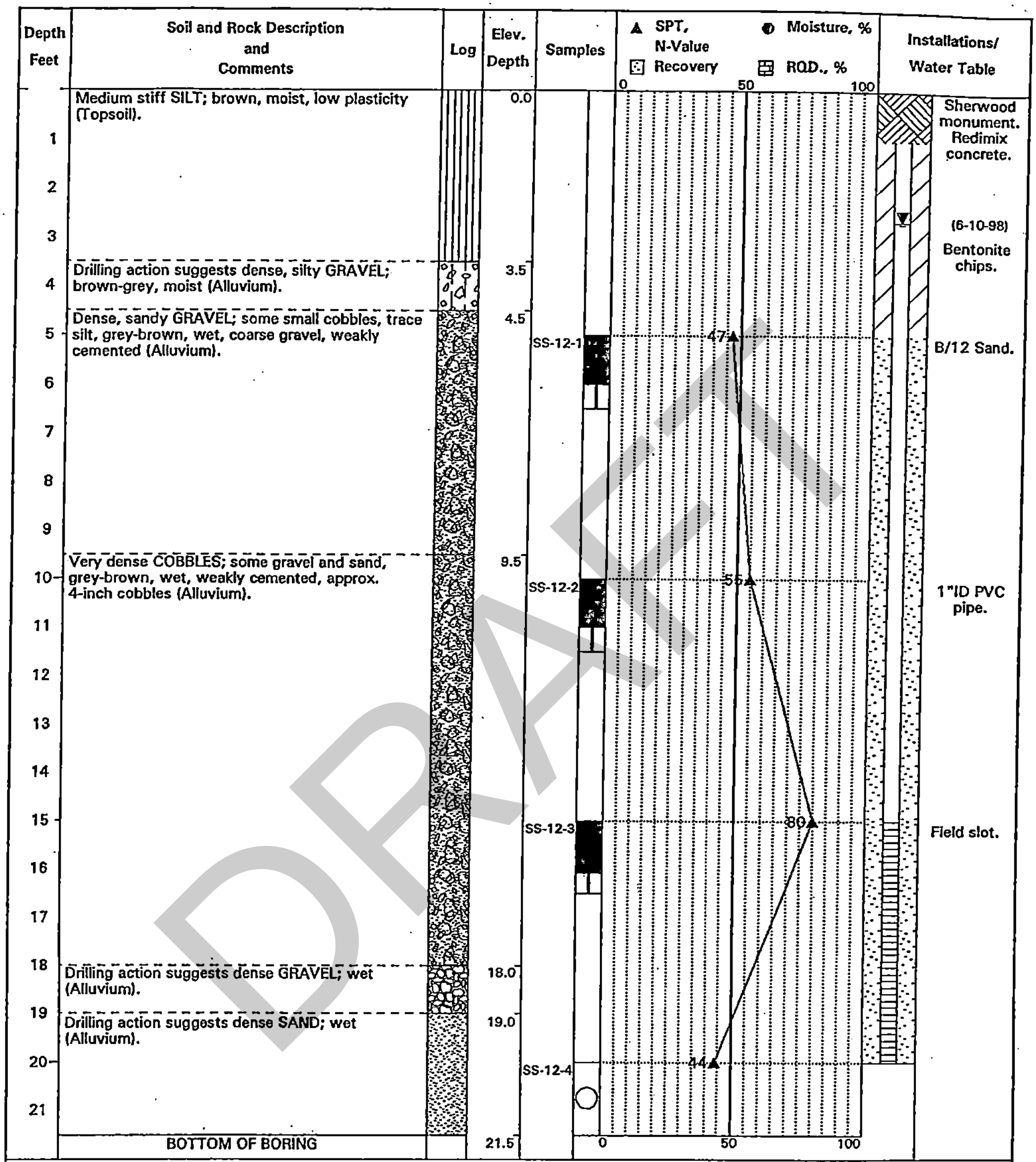
Foundation Engineering, Inc.

Boring Log: BH- 1

75 MGD Transmission Conduit - Phase 2

Turner, Oregon

Soil and Rock Description and Comments	Elev. Depth	Samples
Medium stiff SILT; brown, moist, low plasticity (Topsoil).	0.0	
Drilling action suggests dense, silty GRAVEL; brown-grey, moist (Alluvium).	3.5	
Dense, sandy GRAVEL; some small cobbles, trace silt, grey-brown, wet, coarse gravel, weakly cemented (Alluvium).	4.5	SS-12-1
Very dense COBBLES; some gravel and sand, grey-brown, wet, weakly cemented, approx. 4-inch cobbles (Alluvium).	9.5	SS-12-2 SS-12-3
Drilling action suggests dense GRAVEL; wet (Alluvium).	18.0	
Drilling action suggests dense SAND; wet (Alluvium).	19.0	SS-12-4
BOTTOM OF BORING	21.5	



Project No.: 97100135

Surface Elevation: N/A

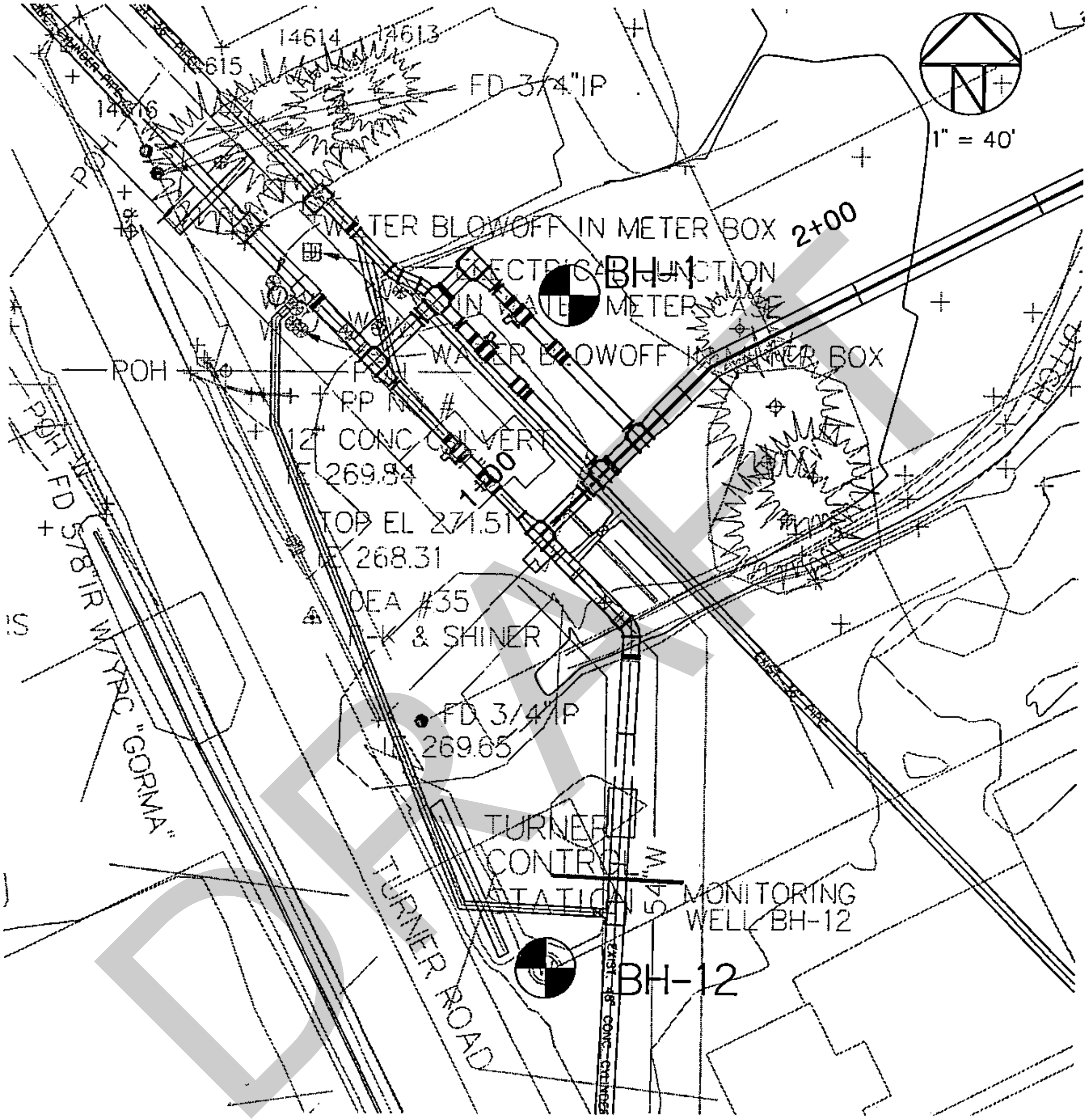
Date of Boring: May 1, 1998

Foundation Engineering, Inc.

Boring Log: BH-12

75 MGD Potable Water Transmission Conduit

Salem, Oregon



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DATE APR 2005
DWN. DLR
APPR.
REVIS.
PROJECT NO.
2051029

FOUNDATION ENGINEERING INC.
PROFESSIONAL GEOTECHNICAL SERVICES
820 NW CORNELL AVENUE
CORVALLIS, OR 97330-4517
BUS. (541) 757-7645 FAX (541) 757-7650

SITE LAYOUT AND BORING LOCATIONS
TURNER CONTROL STATION
75 MGD TRANSMISSION CONDUIT - PHASE 2
TURNER, OREGON

FIGURE NO.
1A

FILE NAME: FIG 1A

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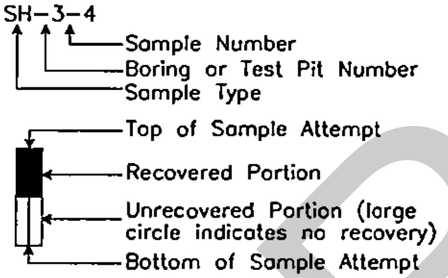

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▨	Clay	▩	Gravel
▩	Basalt	▤	Siltstone

WATER TABLE

▼ Water Table Location
(1/31/00) Date of Measurement
▯ Piezometer Tip Location (if used)

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Can be penetrated several inches by thumb with moderate effort.	5 – 8	0.25 – 0.50	Medium Stiff (Firm)	11 – 30	Medium Dense
Readily indented by thumb but penetrated only with great effort.	9 – 15	0.50 – 1.0	Stiff	31 – 50	Dense
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Indented with difficulty by thumbnail.	31 – 60	> 2.0	Hard		

* Undrained shear strength

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Nonplastic	0 – 3	Cannot be rolled into a thread.
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Stratified	Alternating layers at least 1 inch thick – describe variation.
Laminated	Alternating layers at less than 1 inch thick – describe variation.
Fissured	Contains shears and partings along planes of weakness.
Slickensides	Partings appear glossy or striated.
Blocky	Breaks into lumps – crumbly.
Lensed	Contains pockets of different soils – describe variation.

Term	Soil Cementation Criteria
Weak	Breaks under light finger pressure.
Moderate	Breaks under hard finger pressure.
Strong	Will not break with finger pressure.

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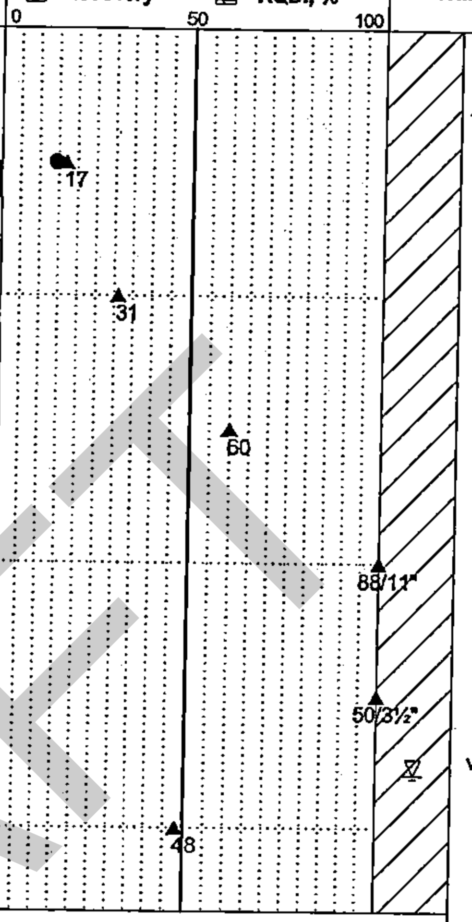
BUS. (541) 757-7645 FAX (541) 757-7650

COMMON TERMS
SOIL DESCRIPTIONS

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth	Samples	▲ SPT, N-Value ● Moisture, % ⚀ Recovery ⊞ RQD., %	Installations/ Water Table
	Loose CRUSHED GRAVEL; grey, damp to dry, fine to coarse, angular to subrounded, (fill).		0.0 0.2			
1	Very stiff, silty CLAY/clayey SILT, trace organics, sand and gravel; brown, iron-stained, damp, medium plasticity, medium to coarse sand, fine, subrounded gravel, (alluvium).					
2						Backfilled with bentonite chips
3				SS-1-1	17	
4						
5	Dense GRAVEL, some clay and silt, trace sand; brown, iron-stained, damp, medium to coarse sand, fine to coarse, subangular to subrounded gravel, (alluvium).		5.0	SS-1-2	31	
6						
7						
8	Grades to very dense GRAVEL, some sand, trace silt at ±7.5 feet.		7.5	SS-1-3	60	
9						
10						
11				SS-1-4	88/11"	
12						
13				SS-1-5	50/3½"	
14						Ground water level (3/10/05)
15						
16	Augering encountered refusal on a cobble or boulder at ±16.6 feet.			SS-1-6	48	
	BOTTOM OF BORING		16.6			



Project No.: 2051029

Surface Elevation: N/A (Approx.)

Date of Boring: March 10, 2005

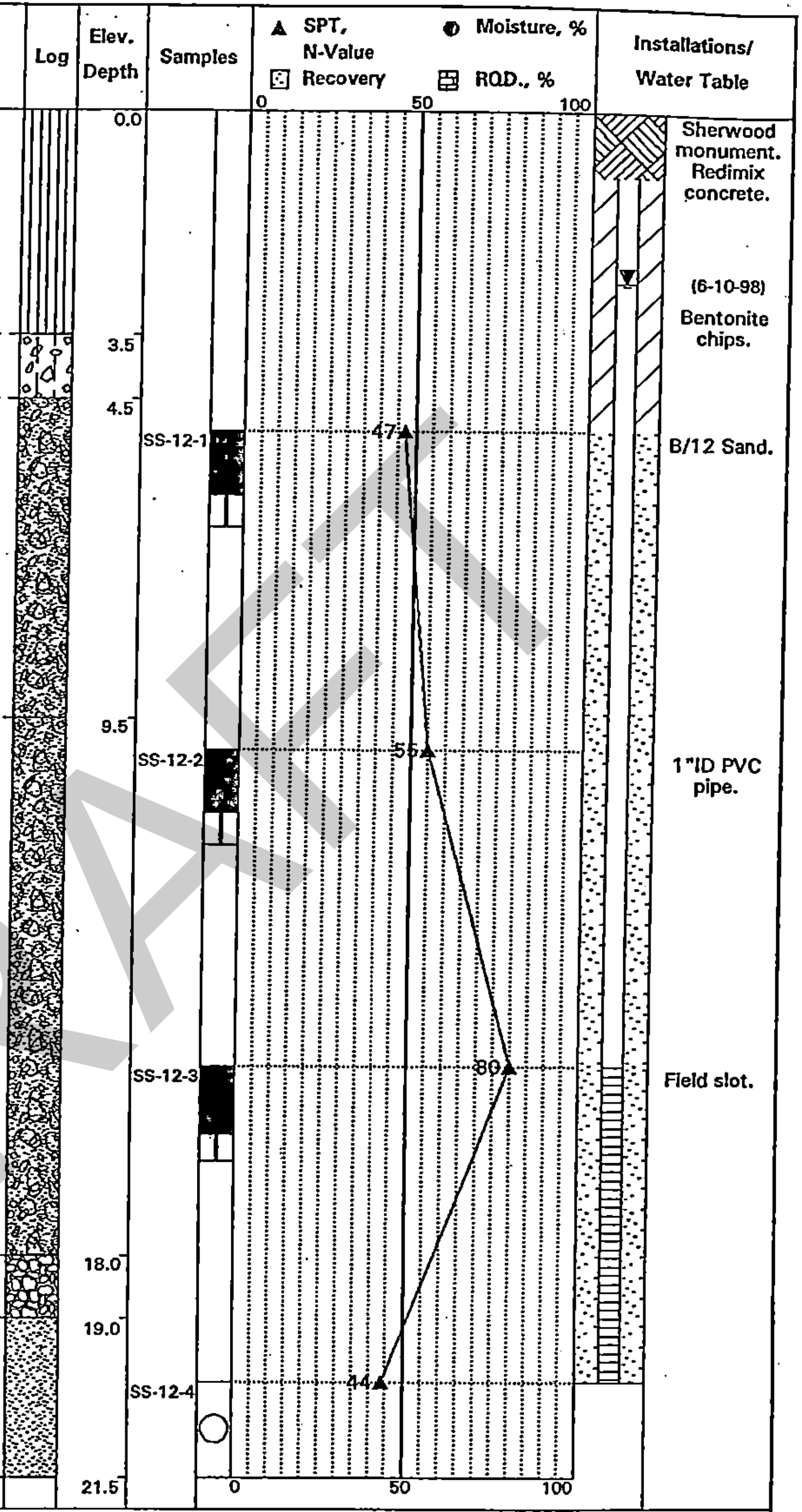
Foundation Engineering, Inc.

Boring Log: BH- 1

75 MGD Transmission Conduit - Phase 2

Turner, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth
	Medium stiff SILT; brown, moist, low plasticity (Topsoil).	0.0
1		
2		
3		
4	Drilling action suggests dense, silty GRAVEL; brown-grey, moist (Alluvium).	3.5
5	Dense, sandy GRAVEL; some small cobbles, trace silt, grey-brown, wet, coarse gravel, weakly cemented (Alluvium).	4.5
6		
7		
8		
9		
10	Very dense COBBLES; some gravel and sand, grey-brown, wet, weakly cemented, approx. 4-inch cobbles (Alluvium).	9.5
11		
12		
13		
14		
15		
16		
17		
18	Drilling action suggests dense GRAVEL; wet (Alluvium).	18.0
19	Drilling action suggests dense SAND; wet (Alluvium).	19.0
20		
21		
	BOTTOM OF BORING	21.5



Project No.: 97100135

Surface Elevation: N/A

Date of Boring: May 1, 1998

Foundation Engineering, Inc.

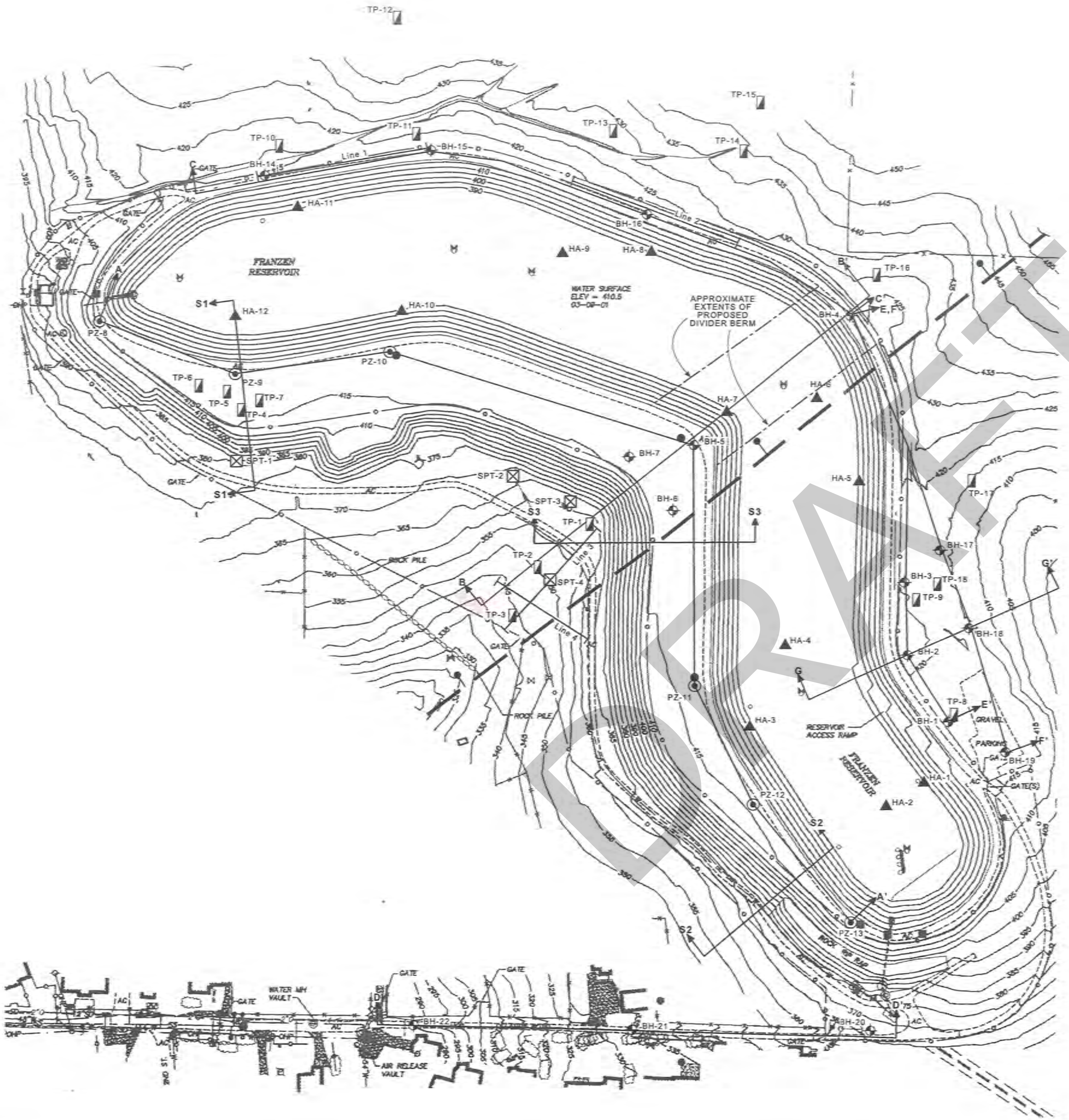
Boring Log: BH-12

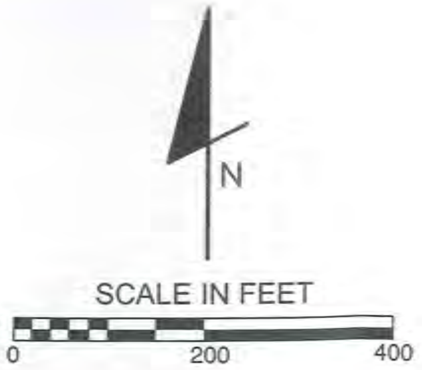
75 MGD Potable Water Transmission Conduit

Salem, Oregon

APPENDIX L

EXISTING INFORMATION
SITE 21 - FRANZEN RESERVOIR
&
REPEATER TOWER





⊕	BH-14	Boring
◉	PZ-12	Boring with dual standpipe piezometer installation
◩	TP-4	Test pit
▲	HA-2	Asphalt coring with hand auger hole
☒	STP-1	Seepage observation point
●		6" steel well casing above surface 1.5'
	A A'	Interpretive subsurface profile location
		Seismic refraction line
		Fault - dashed where concealed, ball on down dropped side
	S1 S2	Cross-section for seepage and stability analyses

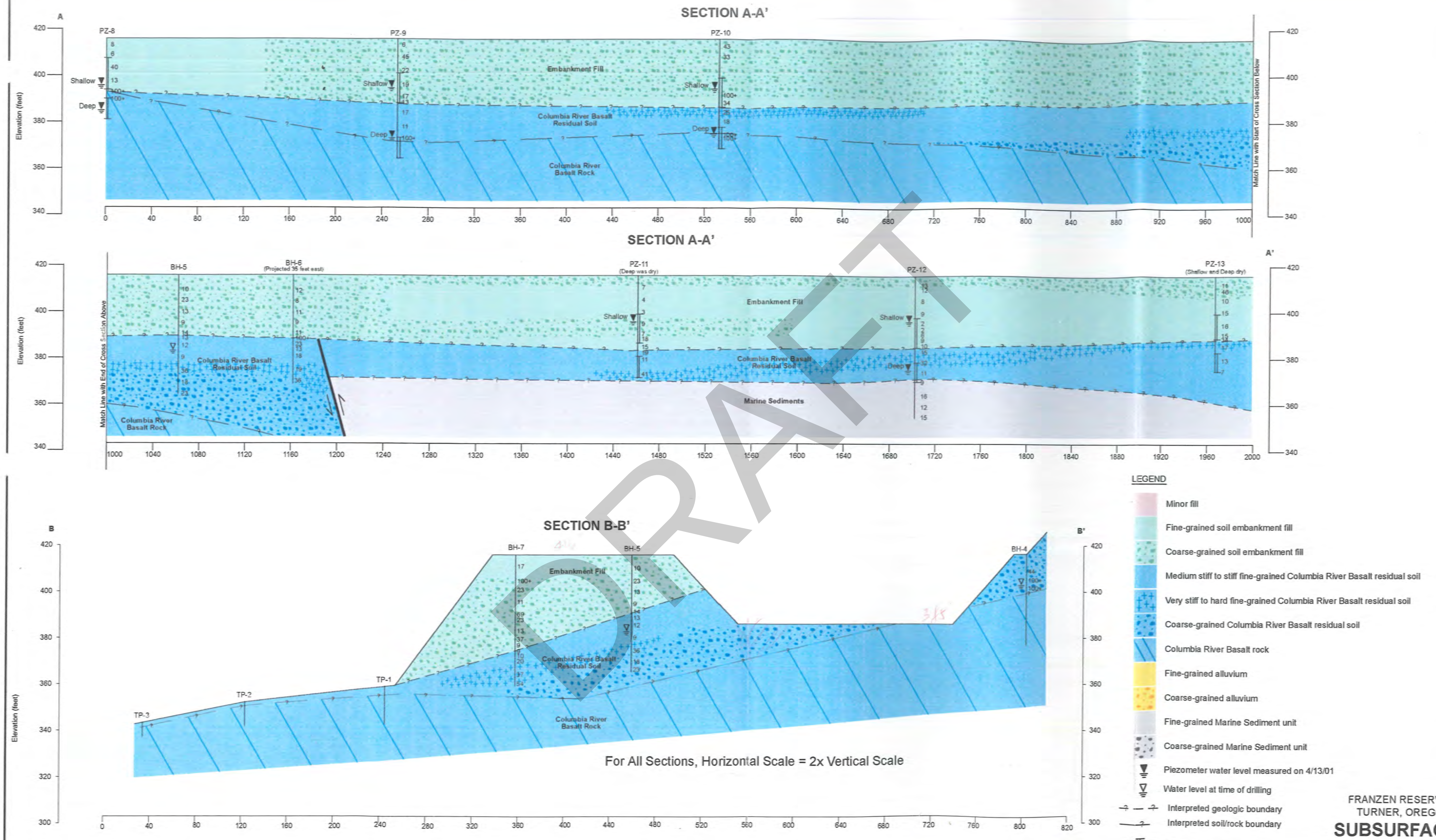
FRANZEN RESERVOIR
TURNER, OREGON

SITE PLAN

SQUIER ASSOCIATES

FIGURE 1

Drawn by: KEB | Checked by: BM | File: 201570\GRAPHICS\FIG-1.FH8 | Date: 05/02 | Job. No. 201570



NOTES:
1. Features shown are for illustrative purposes only and are approximate.
2. Contacts between units are interpretive and may represent a gradual transition. Conditions may vary between borings.

FRANZEN RESERVOIR
TURNER, OREGON

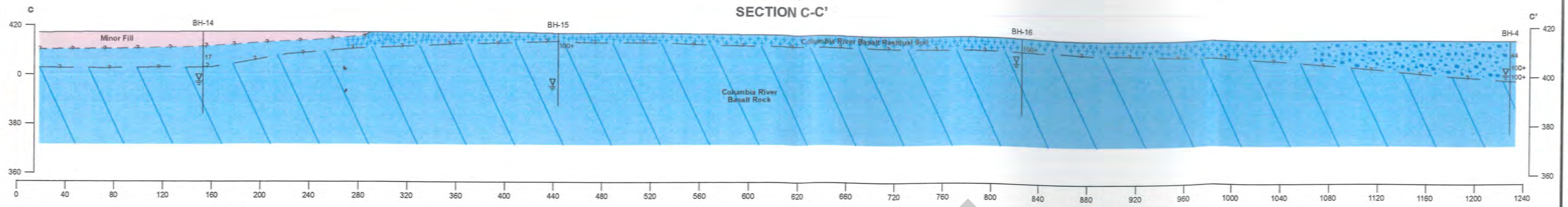
SUBSURFACE STRATIGRAPHY SECTIONS

SQUIER ASSOCIATES

FIGURE 2

Drawn By: KEB	Checked By:	Graphics Check:	File: 201570\GRAPHICS\FIG-4.FH8	Date: 05/02	Job No. 201570

SECTION C-C'

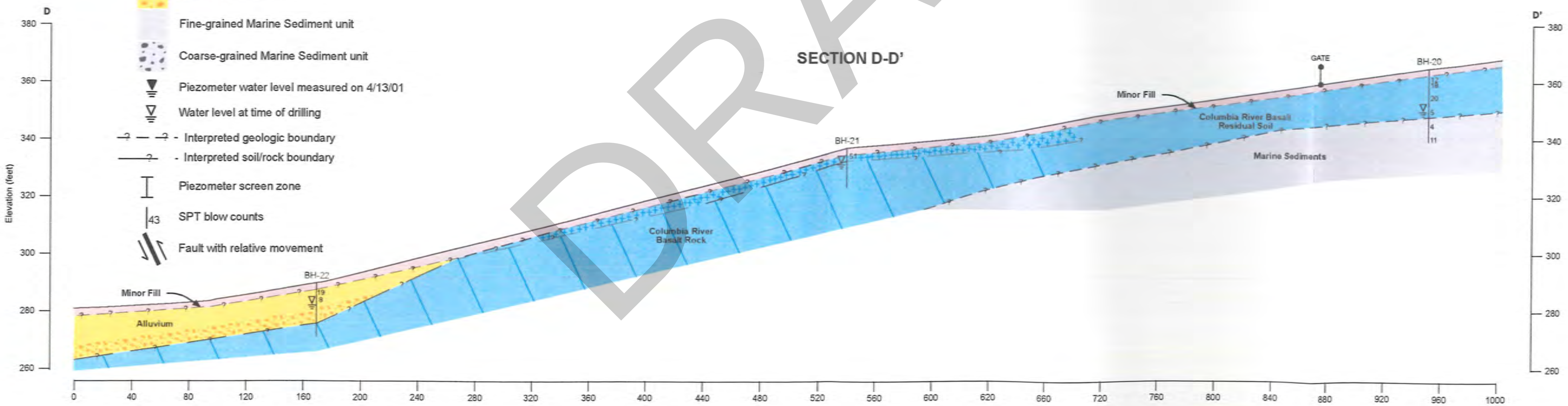


LEGEND

- ▨ Minor fill
- ▨ Fine-grained soil embankment fill
- ▨ Coarse-grained soil embankment fill
- ▨ Medium stiff to stiff fine-grained Columbia River Basalt residual soil
- ▨ Very stiff to hard fine-grained Columbia River Basalt residual soil
- ▨ Coarse-grained Columbia River Basalt residual soil
- ▨ Columbia River Basalt rock
- ▨ Fine-grained alluvium
- ▨ Coarse-grained alluvium
- ▨ Fine-grained Marine Sediment unit
- ▨ Coarse-grained Marine Sediment unit
- ▼ Piezometer water level measured on 4/13/01
- ▽ Water level at time of drilling
- -?- — -?- Interpreted geologic boundary
- —?— Interpreted soil/rock boundary
- I Piezometer screen zone
- |43 SPT blow counts
- ⫽ Fault with relative movement

SCALE
Horizontal: 1 inch = 80 feet
Vertical: 1 inch = 40 feet

SECTION D-D'



TES:
. . Features shown are for illustrative purposes only and are approximate.
2. Contacts between units are interpretive and may represent a gradual transition. Conditions may vary between borings.

FRANZEN RESERVOIR
TURNER, OREGON

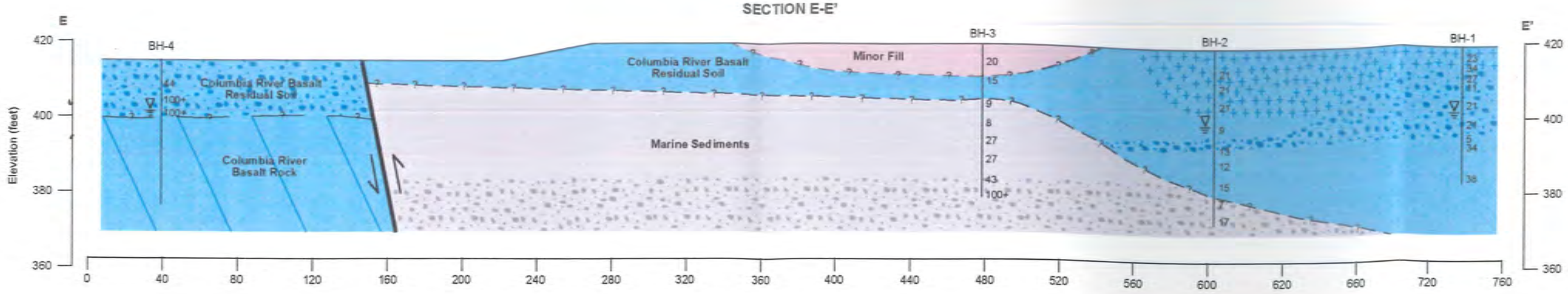
SUBSURFACE STRATIGRAPHY SECTIONS

SQUIER ASSOCIATES

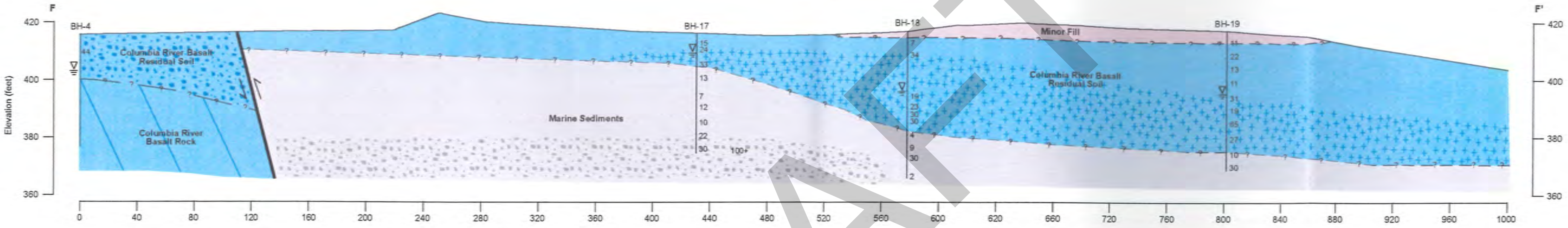
FIGURE 3

Drawn By: KEB	Checked By: BM		File: 201570\GRAPHICS\FIG-5.FH8	Date: 05/02	Job No. 201570

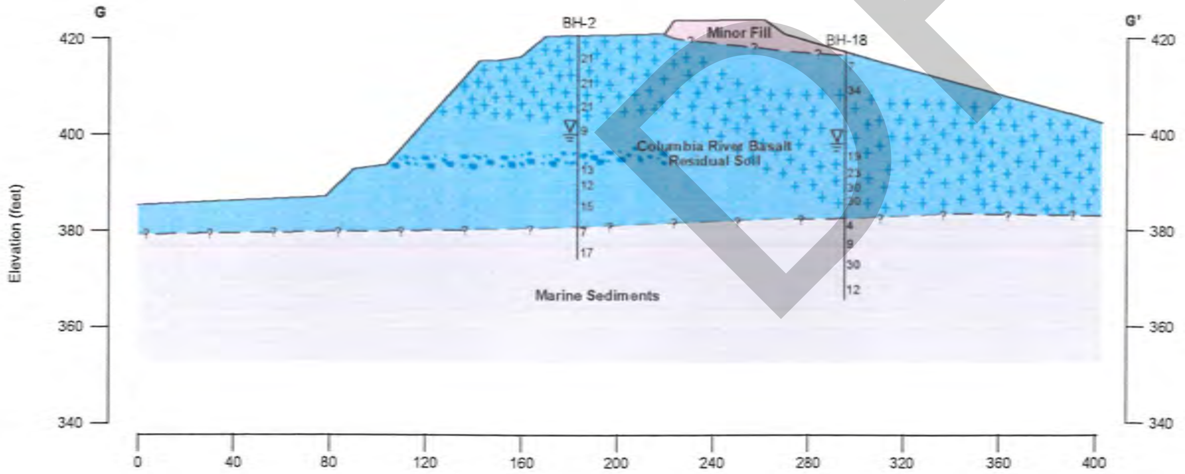
SECTION E-E'



SECTION F-F'



SECTION G-G'



LEGEND

- 🟪 Minor fill
- 🟦 Fine-grained soil embankment fill
- ▨ Coarse-grained soil embankment fill
- 🟦 Medium stiff to stiff fine-grained Columbia River Basalt residual soil
- ▨ Very stiff to hard fine-grained Columbia River Basalt residual soil
- ▨ Coarse-grained Columbia River Basalt residual soil
- ▨ Columbia River Basalt rock
- 🟨 Fine-grained alluvium
- ▨ Coarse-grained alluvium
- ⬜ Fine-grained Marine Sediment unit
- ▨ Coarse-grained Marine Sediment unit
- ▼ Piezometer water level measured on 4/13/01
- ▽ Water level at time of drilling
- -?- — -?- Interpreted geologic boundary
- —?— Interpreted soil/rock boundary
- I Piezometer screen zone
- 43 SPT blow counts
- Fault with relative movement

SCALE

Horizontal: 1 inch = 40 feet
Vertical: 1 inch = 20 feet

NOTES:

1. Features shown are for illustrative purposes only and are approximate.
2. Contacts between units are interpretive and may represent a gradual transition. Conditions may vary between borings.

FRANZEN RESERVOIR
TURNER, OREGON

SUBSURFACE STRATIGRAPHY SECTIONS

SQUIER ASSOCIATES

FIGURE 4

Drawn By: KEB
Checked By:
File: 201570\GRAPHICS\FIG-6.FH8
Date: 05/02
Job No. 201570

APPENDIX M

EXISTING INFORMATION
SITE 22 - GEREN ISLAND
WATER TREATMENT PLANT

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
No base rock observed.								ASPHALT (4.0 TO 4.5 inches thick). Grey-brown, 6-inch minus, moist, dense to very dense, sandy, cobbly GRAVEL with trace silt.
	1-							
	2-							
No ground water infiltration noted.	3-							
	4-							BOTTOM OF TEST PIT
	5-							
	6-							
	7-							
	8-							
	9-							
	10–							

Project No.: 96100011

Surface Elevation: 470 feet (Approx.)

Date of Test Pit: July 24, 1996

Test Pit Log: TP-1
Geren Island Treatment Facility Improvements, Marion County, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Roots extend to 5 feet.								Brown, moist, medium stiff, sandy SILT.
	1-	S-2-1	■					Grey-brown, slightly moist to moist, medium grained, medium dense SAND.
	2-	S-2-2	■					
	3-	S-2-3	■					Grey-brown, slightly moist to moist, dense to very dense, coarse, sandy, cobbly GRAVEL.
	4-							
No ground water infiltration noted.	5-							BOTTOM OF TEST PIT
	6-							
	7-							
	8-							
	9-							
	10–							

Project No.: 96100011

Surface Elevation: 481 feet (Approx.)

Date of Test Pit: July 24, 1996

Test Pit Log: TP-2
Geren Island Treatment Facility Improvements, Marion County, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Roots and organics (logs) extend to about 3 feet.								Brown, moist, medium stiff, sandy, organic SILT with trace fine sand.
	1-	S-3-1	■					
	2-							Brown, moist, medium dense, fine to medium SAND.
	3-							
	4-	S-3-2	■					
	5-							
	6-							
	7-							
Significant ground water infiltration noted at 8 feet.	8-							Grey-brown, wet, very dense, gravelly COBBLES.
Large (2-foot diameter) boulder encountered at 9 feet.	9-							BOTTOM OF TEST PIT
	10–							

Project No.: 96100011

Surface Elevation: 480 feet (Approx.)

Date of Test Pit: July 24, 1996

Test Pit Log: TP-3

Geren Island Treatment Facility Improvements, Marion County, Oregon

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Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
								ASPHALT (3.25 inches thick). Grey, moist, dense, sandy, crushed GRAVEL with some silt (base rock).
	1-							Grey-brown, moist, dense to very dense, gravelly COBBLES.
	2-							
	3-							
No ground water infiltration noted.	4-							BOTTOM OF TEST PIT
	5-							
	6-							
	7-							
	8-							
	9-							
	10–							

Project No.: 96100011

Surface Elevation: 464 feet (Approx.)

Date of Test Pit: July 24, 1996

Test Pit Log: TP-4

Geren Island Treatment Facility Improvements, Marion County, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Roots extend to about 12 inches.								Brown, slightly moist, medium dense, silty SAND with some gravel.
	1-							Grey-brown, dry to slightly moist, dense to very dense, gravelly COBBLES.
	2-							
	3-	S-5-1	■					
	4-							
No ground water infiltration noted.	5-							BOTTOM OF TEST PIT
	6-							
	7-							
	8-							
	9-							
	10-							

Project No.: 96100011

Surface Elevation: 462 feet (Approx.)

Date of Test Pit: July 24, 1996

Test Pit Log: TP-5

Geren Island Treatment Facility

Improvements, Marion County, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Roots extend to about 2 feet.		S-6-1	■					Brown, slightly moist, medium stiff, gravelly, cobbly SILT with trace sand.
	1-							Grey-brown, dry, dense to very dense, coarse, sandy, gravelly COBBLES.
	2-	S-6-2	■					
	3-							
	4-							
	5-							
	6-							
Significant ground water infiltration noted at 7.5 feet.	7-							
	8-							
Large (2 to 3-foot diameter) boulder encounterd at 10 feet. Caving prevented further excavation.	9-							
	10-							BOTTOM OF TEST PIT

Project No.: 96100011

Surface Elevation: 476 feet (Approx.)

Date of Test Pit: July 24, 1996

Test Pit Log: TP-6

Geren Island Treatment Facility

Improvements, Marion County, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Piezometer pipe extends above ground about 2 feet.								Grey-brown, dense, non-cemented, well-rounded COBBLES and GRAVEL with some medium to coarse sand.
	1-							
	2-							
Repeated caving from the ground surface to the bottom of the excavation.	3-							Roots from about 0 to 2.5 feet.
	4-							
	5-							Cobbles to about 8 inches in diameter, most from 4 to 5 inches in diameter.
Piezometer consists of 1.5-inch PVC pipe. The bottom 5 feet is slotted and wrapped in geotextile.	6-							
	7-							Moisture increases with depth.
	8-							BOTTOM OF TEST PIT
	9-							
	10-							

Project No.: 96100011

Surface Elevation: 462 feet (Approx.)

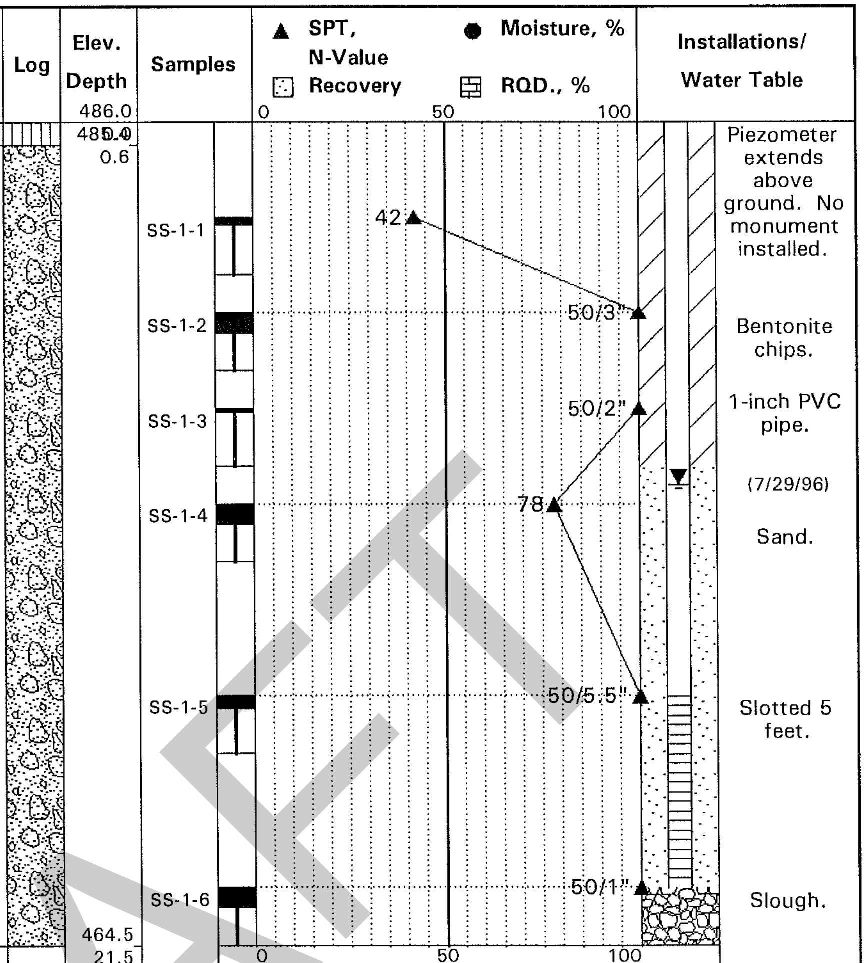
Date of Test Pit: August 9, 1996

Test Pit Log: TP-7

Geren Island Treatment Facility

Improvements, Marion County, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples	SPT, N-Value
		486.0 0.0		
	Brown, medium stiff SILT.	485.4 0.6		
1	Grey-brown, dense, coarse, sandy, gravelly COBBLES.			
2				
3			SS-1-1	42
4	Mud loss occurred between 4 and 5 feet.			
5	The soil becomes very dense below 5 feet.		SS-1-2	50/3"
6				
7				
8			SS-1-3	50/2"
9				
10			SS-1-4	78
11				
12				
13				
14	Boulder encountered between 13.5 and 15 feet.			
15	Mud loss occurred between 15 and 16.5 feet.		SS-1-5	50/5.5"
16				
17				
18				
19				
20			SS-1-6	50/1"
21		464.5 21.5		
	BOTTOM OF BORING			



Piezometer extends above ground. No monument installed.

Bentonite chips.

1-inch PVC pipe.

Sand.

Slotted 5 feet.

Slough.

Project No.: 96100011

Surface Elevation: 486 feet (Approx.)

Date of Boring: July 25, 1996

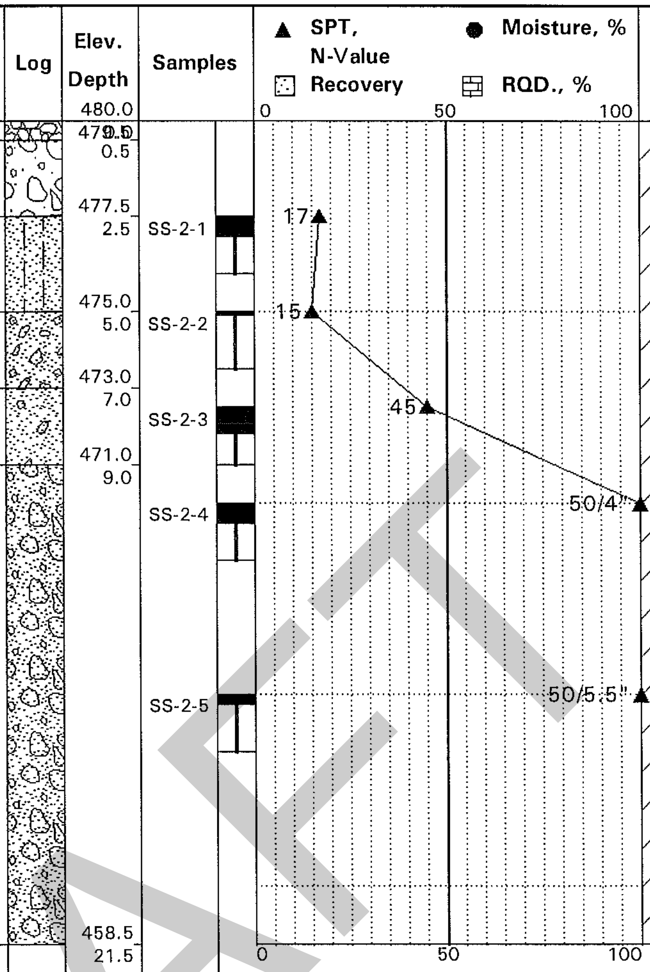
Foundation Engineering, Inc.

Boring Log: BH-1

Geren Island Treatment Facility

Improvements, Marion County, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth 480.0	Samples	SPT, N-Value	Installations/ Water Table
	3/4-inch minus, crushed GRAVEL (road base).	479.5 0.5			Backfilled with bentonite chips to surface.
1	Cobbly GRAVEL (dike fill).	477.5 2.5			
3	Brown, medium dense, medium to coarse SAND with trace silt (dike fill).	475.0 5.0	SS-2-1	17	
5	Dark grey, medium dense, sandy GRAVEL (dike fill).	473.0 7.0	SS-2-2	15	
7	Brown, dense SAND with trace gravel (dike fill).	471.0 9.0	SS-2-3	45	
9	Grey-brown, very dense, sandy, gravelly COBBLES (native alluvium).		SS-2-4	50/4"	
15			SS-2-5	50/5.5"	
20	Sampling at 20 feet prevented by caving.	458.5 21.5			
	BOTTOM OF BORING				



Project No.: 96100011

Surface Elevation: 480 feet (Approx.)

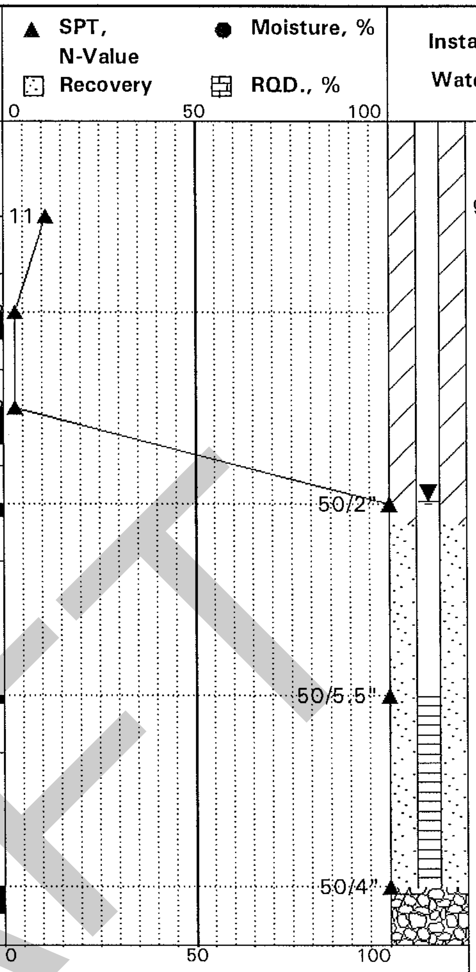
Date of Boring: July 25, 1996

Foundation Engineering, Inc.

Boring Log: BH-2

Geren Island Treatment Facility Improvements, Marion County, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples	SPT, N-Value	Installations/ Water Table
		480.0 0.0			
0	Sandy, gravelly COBBLES (site fill).				Piezometer extends above ground with cap. No monument installed.
1					
2					
3	Brown, very loose, fine, silty SAND.	477.5 2.5	SS-3-1	11	
4					
5			SS-3-2	2	Bentonite chips.
6					
7					1-inch PVC pipe.
8	Scattered gravel encountered below 8 feet.	472.0 8.0	SS-3-3	2	
9					
10		469.5 10.5	SS-3-4	50/2"	(7/30/96)
11	Grey-brown, very dense, coarse sandy, gravelly COBBLES.				
12					Sand.
13					
14					
15			SS-3-5	50/5.5"	
16					
17					Slotted 5 feet.
18					
19					
20			SS-3-6	50/4"	Slough.
21	BOTTOM OF BORING	458.5 21.5			



Project No.: 96100011

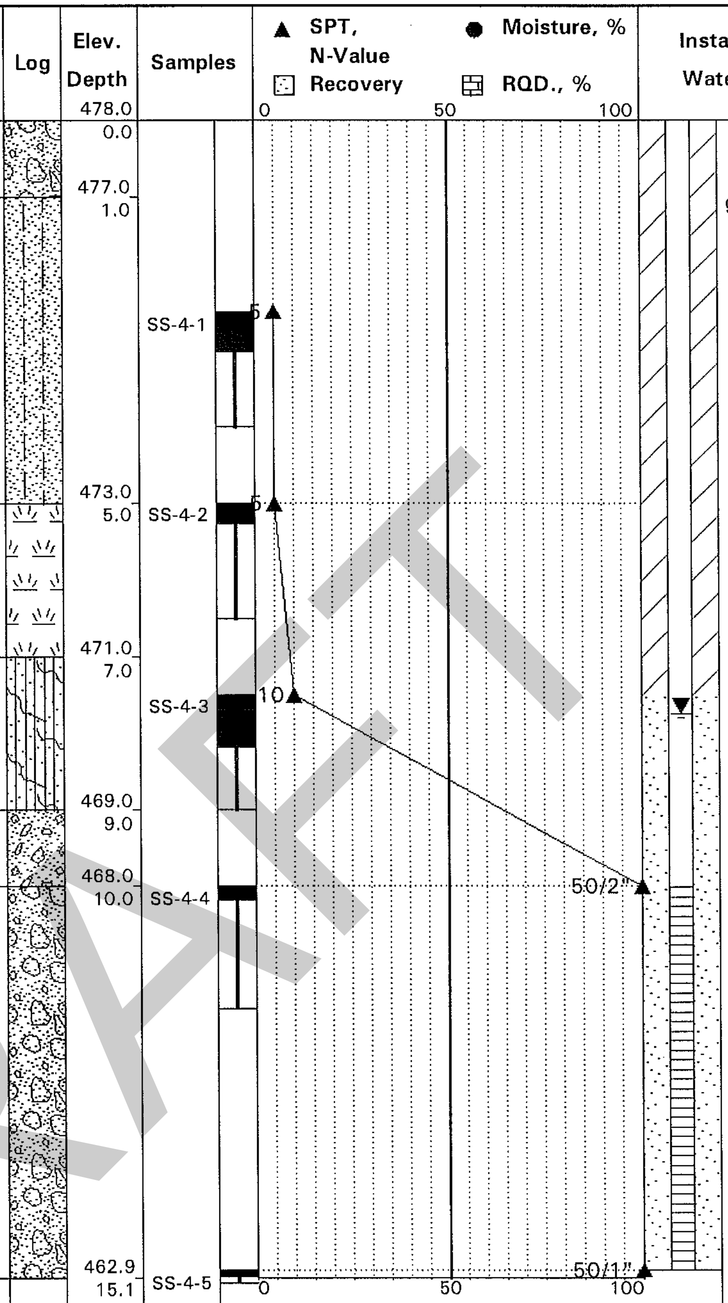
Surface Elevation: 480 feet (Approx.)

Date of Boring: July 26, 1996

Foundation Engineering, Inc.

Boring Log: BH-3

Geren Island Treatment Facility Improvements, Marion County, Oregon



Depth Feet	Soil and Rock Description and Comments	Elev. Depth 478.0	Samples	SPT, N-Value	Installations/ Water Table
	SAND with some gravel and cobbles (old road base or site fill).	0.0			Piezometer extends above ground. No monument installed.
1	Brown, loose, silty, fine to medium SAND (fill?).	477.0 1.0			
2					
3			SS-4-1	5	
4	Mud loss into decomposed tree occurred at 5 feet.				
5	Dark brown, medium stiff, medium to coarse, sandy PEAT (decomposed wood). Likely represents the original ground surface.	473.0 5.0	SS-4-2	5	Bentonite chips.
6					
7	Grey-brown, stiff SILT with some fine sand and trace organics (decomposed wood).	471.0 7.0			
8			SS-4-3	10	(7/30/96)
9	Sandy GRAVEL.	469.0 9.0			
10	Grey-brown, very dense, gravelly COBBLES with some sand.	468.0 10.0	SS-4-4	50/2"	
11					Sand.
12					
13					Slotted 5 feet.
14	Discontinued boring due to extremely slow drilling.				
15	BOTTOM OF BORING	462.9 15.1	SS-4-5	50/1"	

Project No.: 96100011

Surface Elevation: 478 feet (Approx.)

Date of Boring: July 26, 1996

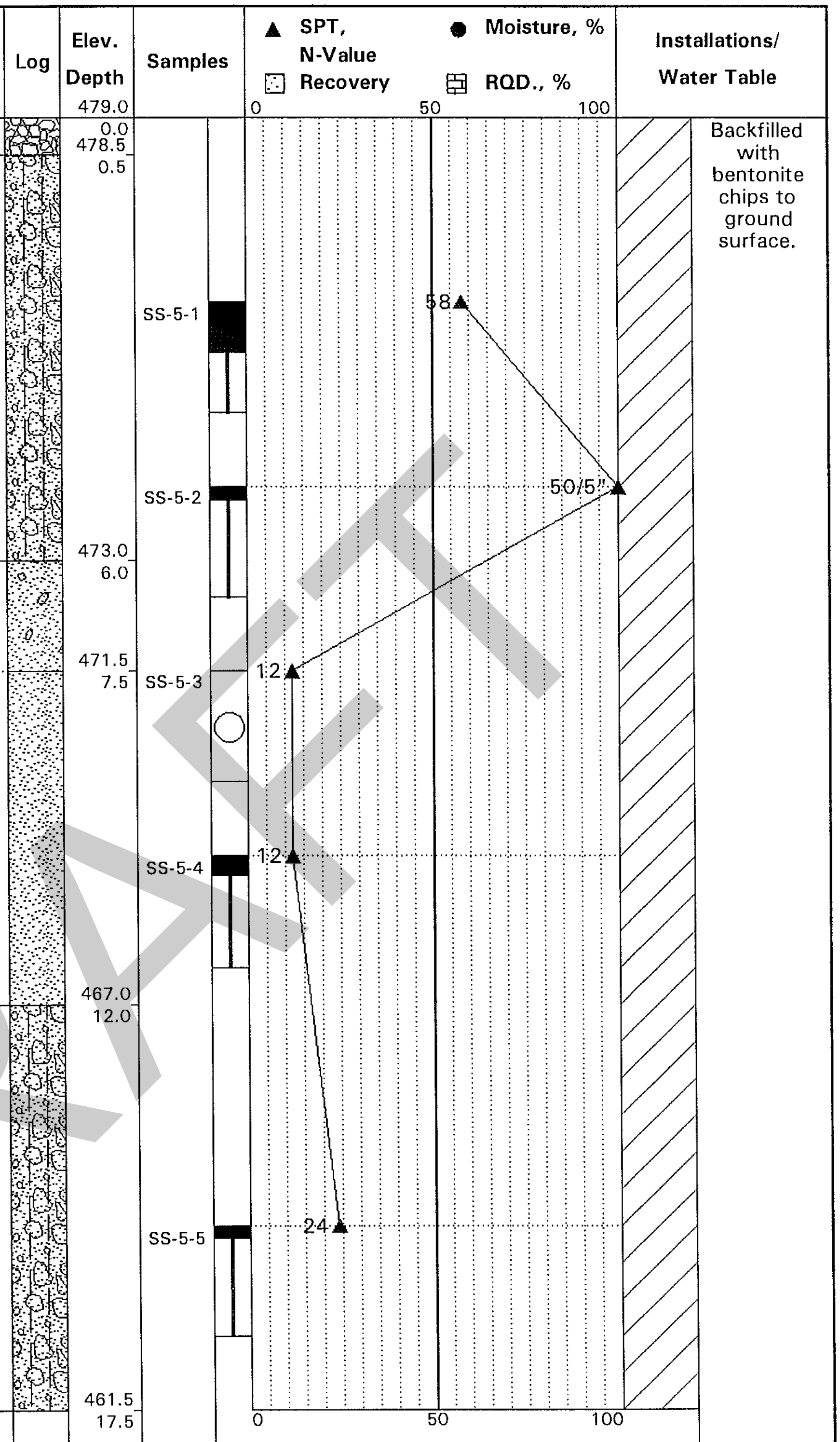
Foundation Engineering, Inc.

Boring Log: BH-4

Geren Island Treatment Facility

Improvements, Marion County, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth
0	3/4-inch minus, crushed GRAVEL (road base).	479.0 0.0
0.5	Brown, very dense, slightly cemented, sandy GRAVEL with trace silt and cobbles (dike fill).	478.5 0.5
6	Brown, medium dense, medium SAND with some gravel (filter dike fill).	473.0 6.0
7.5	Gravel not encountered between 7.5 and 12 feet.	471.5 7.5
12	Grey-brown, medium dense, sandy GRAVEL with trace cobbles and silt (native alluvium). Mud loss and caving prevented drilling below 17.5 feet.	467.0 12.0
17.5	BOTTOM OF BORING	461.5 17.5



Project No.: 96100011

Surface Elevation: 479 feet (Approx.)

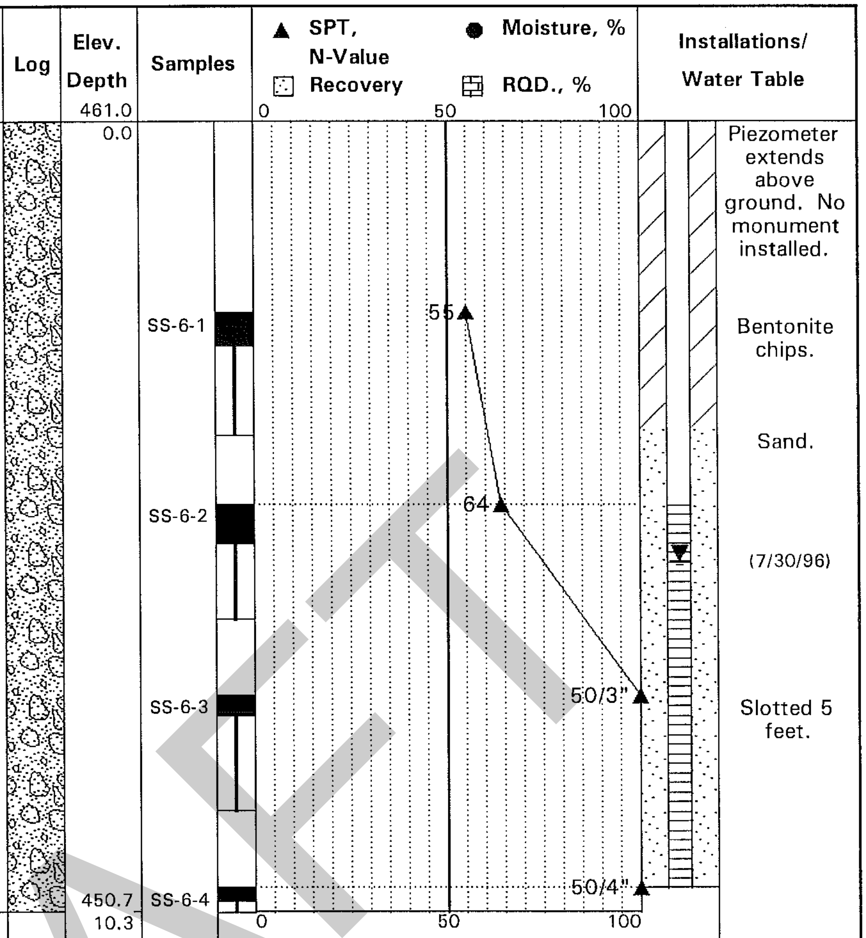
Date of Boring: July 29, 1996

Foundation Engineering, Inc.

Boring Log: BH-5

Geren Island Treatment Facility Improvements, Marion County, Oregon

Depth Feet	Soil and Rock Description and Comments
0	Grey-brown, very dense, gravelly COBBLES with some coarse sand.
5	Gradual mud loss to 5 feet. No circulation of mud between 5 and 6 feet. Continuous caving below 5 feet.
10	BOTTOM OF BORING



Piezometer extends above ground. No monument installed.

Bentonite chips.

Sand.

Slotted 5 feet.

Project No.: 96100011

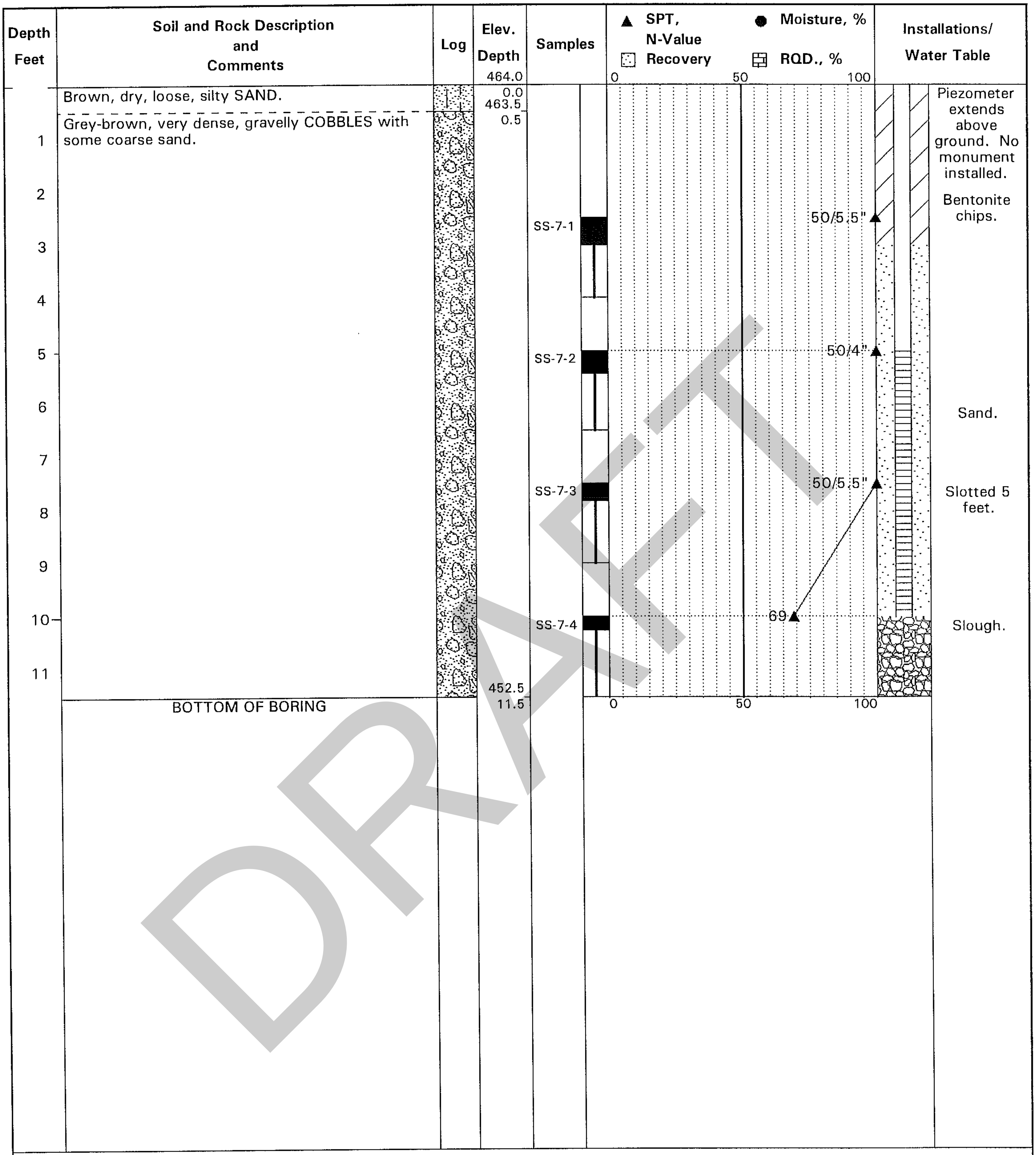
Surface Elevation: 461 feet (Approx.)

Date of Boring: July 29, 1996

Foundation Engineering, Inc.

Boring Log: BH-6

Geren Island Treatment Facility Improvements, Marion County, Oregon



Brown, dry, loose, silty SAND.

Grey-brown, very dense, gravelly COBBLES with some coarse sand.

BOTTOM OF BORING

Piezometer extends above ground. No monument installed.

Bentonite chips.

Sand.

Slotted 5 feet.

Slough.

Project No.: 96100011

Surface Elevation: 464 feet (Approx.)

Date of Boring: July 30, 1996

Foundation Engineering, Inc.

Boring Log: BH-7

Geren Island Treatment Facility Improvements, Marion County, Oregon

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth	Samples	▲ SPT, N-Value ☐ Recovery ● Moisture, % ⊞ RQD., % (0 50 100)	Installations/ Water Table
			462.0			
	Grey-brown, medium dense GRAVEL with some sand and cobbles.		0.0			Backfilled with bentonite chips to surface.
1						
2				SS-8-1	18▲	
3	Abundant caving and mud loss between 4 and 5 feet.					
4			458.0 4.0			
	Grey-brown, dense to very dense, gravelly COBBLES with some sand (based on drilling action). Boring discontinued at 5 feet due to extremely difficult drilling.					
5			457.0 5.0		0 50 100	

BOTTOM OF BORING

Abandoned, replaced with TP-7.

Project No.: 96100011

Surface Elevation: 462 feet (Approx.)

Date of Boring: July 30, 1996

Foundation Engineering, Inc.

Boring Log: BH-8

Geren Island Treatment Facility Improvements, Marion County, Oregon

Project: Geren Island Water Treatment Facility
Project Location: 2700 E. Santiam St. Stayton, Oregon 97203
Project Number: 5966.0

Log of Boring B-01

Date(s) Drilled	**04/08/2019 - 04/09/2019**	Geotechnical Consultant	**McMillen Jacobs Associates**	Logged By	**J. Fissel**	Checked By	**J. Quinn**
Drilling Method/ Rig Type	**Sonic Drilling/TSi 150CC Track-Mounted Drill Rig**	Drilling Contractor	**Holt Services Inc.**	Total Depth of Borehole	**75.0 ft**		
Hole Diameter	**6.00 in**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**485.0 ft** (approximate)		
Location	**Near SW corner of Filter #3 West**	Coordinates		Elevation Source	**30% Submittal Drawings, Aug 2019**		

☐ PENETRATION RESISTANCE BLOWS/FT 10 20 30 40
○ WATER CONTENT (MC)
▎ ATTERBERG LL/PL 20 40 60 80

ELEV. (FT)	WATER LEVEL	DEPTH (FT)	RECOVERY (%)	BLOW COUNTS	SAMPLE NUMBER	USCS	MATERIAL DESCRIPTION	REMARKS AND TESTS
						ML	Brown, Sandy SILT (ML); rootlets. [Topsoil]	
			100		R-1	GW	Medium dense, moist, gray, Well-graded GRAVEL (GW); well-graded, subrounded, fine to coarse gravel, fine to coarse sand. [Fill] *Gravel coarsening upward*	
480		5						
			67	8-16-16 (N=32)	SPT_1	GW-GM	Dense, moist, gray, Well-graded GRAVEL with silt and sand (GW-GM); low plasticity fines, well-graded, subangular, fine to coarse gravel, fine to coarse sand. [Fill] *Gravel coarsening upward*	
			100		R-2			
						GW	Dense, moist, gray, Well-graded GRAVEL with sand (GW); well-graded, angular, fine gravel, fine to coarse sand, trace silt. [Fill]	
475		10						
			45	13-16-21 (N=37)	SPT_2	GW-GM	Dense, moist, gray, Well-graded GRAVEL with silt and sand (GW-GM); low plasticity fines, well-graded, angular, fine to coarse gravel, fine to coarse sand. [Fill] *Moist grading to wet*	
			75		R-3			
470		15						
	▽		73	9-50/5" (Refusal)	SPT_3			Water Level at 15.75 feet below ground surface after drilling.
			60		R-4	GW-GM	Dense, wet, gray, Well-graded GRAVEL with silt and sand (GW-GM); low plasticity fines, well-graded, subrounded to rounded, fine to coarse gravel, fine to coarse sand. [Alluvial Deposits]	
465		20						
			45	6-20-22 (N=42)	SPT_4			
			100		R-5			
460		25						
			106	3-10-50/5" (Refusal)	SPT_5			
			100		R-6		Very dense, wet, brown, Silty GRAVEL with sand, cobbles, and boulders (GM); low plasticity fines, trace boulders, trace subrounded to rounded cobbles, subrounded to rounded gravel, fine to	

Project: Geren Island Water Treatment Facility
Project Location: 2700 E. Santiam St. Stayton, Oregon 97203
Project Number: 5966.0

Log of Boring B-01

Date(s) Drilled	**04/08/2019 - 04/09/2019**	Geotechnical Consultant	**McMillen Jacobs Associates**	Logged By	**J. Fissel**	Checked By	**J. Quinn**
Drilling Method/ Rig Type	**Sonic Drilling/TSi 150CC Track-Mounted Drill Rig**	Drilling Contractor	**Holt Services Inc.**	Total Depth of Borehole	**75.0 ft**		
Hole Diameter	**6.00 in**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**485.0 ft** (approximate)		
Location	**Near SW corner of Filter #3 West**	Coordinates		Elevation Source	**30% Submittal Drawings, Aug 2019**		

ELEV. (FT)	WATER LEVEL	DEPTH (FT)	SAMPLE TYPE	RECOVERY (%)	BLOW COUNTS	SAMPLE NUMBER	☐ PENETRATION RESISTANCE BLOWS/FT 10 20 30 40 ○ WATER CONTENT (MC) ❙ ATTERBERG LL/PL 20 40 60 80	USCS GRAPHIC	USCS	MATERIAL DESCRIPTION	REMARKS AND TESTS	BACKFILL/INSTALL.
				100	50/5" (Refusal)	SPT_6				Very dense, wet, brown, Silty GRAVEL with sand, cobbles, and boulders (GM); low plasticity fines, trace boulders, trace subrounded to rounded cobbles, subrounded to rounded gravel, fine to coarse sand, weakly cemented. [Alluvial Deposits]		
				100		R-7						
450		35								*Encountered seam of mostly subrounded cobbles (particle size of 4").*		
				100		R-8						
445		40									4% cobbles, 59% Gravel, 12% fines per ASTM D422.	
				100		G_1						
				100		R-9						
440		45		100	50/5" (Refusal)	SPT_7			GM			
				100		R-10						
435		50										
				100		R-11						
430		55		100	50/4" (Refusal)	SPT_8				*Boulder/Cobble content increases below 55'*		
				100		R-12						

Project: Geren Island Water Treatment Facility
Project Location: 2700 E. Santiam St. Stayton, Oregon 97203
Project Number: 5966.0

Log of Boring B-01

Date(s) Drilled	**04/08/2019 - 04/09/2019**	Geotechnical Consultant	**McMillen Jacobs Associates**	Logged By	**J. Fissel**	Checked By	**J. Quinn**
Drilling Method/ Rig Type	**Sonic Drilling/TSi 150CC Track-Mounted Drill Rig**	Drilling Contractor	**Holt Services Inc.**	Total Depth of Borehole	**75.0 ft**		
Hole Diameter	**6.00 in**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**485.0 ft** (approximate)		
Location	**Near SW corner of Filter #3 West**	Coordinates		Elevation Source	**30% Submittal Drawings, Aug 2019**		

ELEV. (FT)	WATER LEVEL	DEPTH (FT)	SAMPLE TYPE	RECOVERY (%)	BLOW COUNTS	SAMPLE NUMBER	☐ PENETRATION RESISTANCE BLOWS/FT 10 20 30 40 ○ WATER CONTENT (MC) ❙ ATTERBERG LL/PL 20 40 60 80	USCS GRAPHIC	USCS	MATERIAL DESCRIPTION	REMARKS AND TESTS	BACKFILL/INSTALL.
				100		R-13				Very dense, wet, brown, Silty GRAVEL with sand, cobbles, and boulders (GM); low plasticity fines, trace boulders, trace subrounded to rounded cobbles, subrounded to rounded gravel, fine to coarse sand, weakly cemented. [Alluvial Deposits]		
420		65										
				100		R-14						
415		70								Dense, moist, brown, Poorly graded GRAVEL with clay and sand (GP-GC); medium plasticity fines, poorly graded, subrounded to rounded, fine to coarse gravel, fine to coarse sand, trace silt, weak cementation. [Alluvial Deposits] *Moisture content increases. Brown, red-brown, yellow and black (weathering/oxidation)*		
				100		R-15			GP-GM			
410		75									Borehole completed at 75 feet below ground surface (bgs).	
405		80										
400		85										

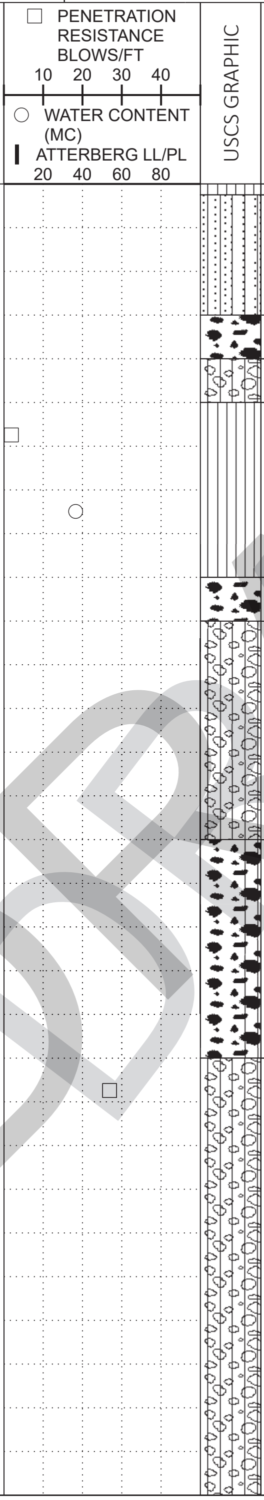
McMILLEN JACOBS ASSOCIATES

Boring B-01

Project: Geren Island Water Treatment Facility
Project Location: 2700 E. Santiam St. Stayton, Oregon 97203
Project Number: 5966.0

Log of Boring B-02

Date(s) Drilled	**04/09/2019**	Geotechnical Consultant	**McMillen Jacobs Associates**	Logged By	**J. Fissel**
Checked By	**J. Quinn**				
Drilling Method/ Rig Type	**Sonic Drilling/TSi 150CC Track-Mounted Drill Rig**	Drilling Contractor	**Holt Services Inc.**	Total Depth of Borehole	**40.9 ft**
Hole Diameter	**6.00 in**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**485.0 ft** (approximate)
Location	**West-Central Perimeter of Filter #3 West**	Coordinates		Elevation Source	**30% Submittal Drawings, Aug 2019**



ELEV. (FT)	WATER LEVEL	DEPTH (FT)	RECOVERY (%)	BLOW COUNTS	SAMPLE NUMBER	USCS	MATERIAL DESCRIPTION	REMARKS AND TESTS
						ML	Dark brown, Sandy SILT (ML); roots. [Topsoil]	
						SM	Loose, moist, brown, Silty SAND with cobbles (SM); trace subrounded to rounded cobbles, trace subrounded, fine to coarse gravel, trace roots. [Fill]	
			70		R-1			
						GW		
						GM	Medium dense, moist, gray, Well-graded GRAVEL with sand (GW); well-graded, angular gravel, fine to coarse sand. [Fill]	
480		5						
			0	1-1-1 (N=2)	SPT_1			
						ML	Medium dense, moist, brown, Silty GRAVEL with sand (GM); subrounded, fine to coarse gravel, fine to coarse sand. [Fill]	
			100		G-2			57% Fines per ASTM D1140
			100		R-2			
						GW	Soft, moist, brown, Sandy SILT (ML); low plasticity, fine to medium sand. [Fill] *Below 7 feet, color changes from brown to gray.*	
475		10						
			77	17-29-30/ 1" (N=30/1")	SPT_2			
							Very dense, moist, gray, Well-graded GRAVEL with sand (GW); well-graded, subangular, gravel, fine to coarse sand. [Fill]	
			100		R-3	GM		
							Very dense, moist, brown and gray, Silty GRAVEL with sand and cobbles (GM); trace subrounded to rounded cobbles, subangular, fine to coarse gravel, fine to coarse sand. [Fill]	
470		15						
			79	50/5" (Refusal)	SPT_3			
	▽							Water level inside borehole after drilling was 16.25 feet bgs.
						GW-GM		
			70		R-4			
							Very dense, wet, gray, Well-graded GRAVEL with silt, sand, and cobbles (GW-GM); low plasticity fines, trace subrounded to rounded cobbles, well-graded, subangular gravel, with fine to coarse sand. [Fill] *Color is entirely gray from 19 to 20 feet (possible cobble).*	
465		20						5% cobbles, 56% Gravel, 29% Sand, 10% fines
			100 100	5-8-19 (N=27)	SPT_4 G-2			
			100		R-5		Very dense, wet, gray, Silty GRAVEL with sand, boulders, and cobbles (GM); low plasticity fines, trace boulders, trace subrounded to rounded cobbles, subangular coarse gravel, medium to coarse sand. [Alluvial Deposits]	
460		25						
			50	30-25-30 (N=55)	SPT_5			
			100		R-6			

McMILLEN JACOBS ASSOCIATES

Boring B-02

Project: Geren Island Water Treatment Facility
Project Location: 2700 E. Santiam St. Stayton, Oregon 97203
Project Number: 5966.0

Log of Boring B-02

Date(s) Drilled	**04/09/2019**	Geotechnical Consultant	**McMillen Jacobs Associates**	Logged By	**J. Fissel**	Checked By	**J. Quinn**
Drilling Method/ Rig Type	**Sonic Drilling/TSi 150CC Track-Mounted Drill Rig**	Drilling Contractor	**Holt Services Inc.**	Total Depth of Borehole	**40.9 ft**		
Hole Diameter	**6.00 in**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**485.0 ft** (approximate)		
Location	**West-Central Perimeter of Filter #3 West**	Coordinates		Elevation Source	**30% Submittal Drawings, Aug 2019**		

ELEV. (FT)	WATER LEVEL	DEPTH (FT)	SAMPLE TYPE	RECOVERY (%)	BLOW COUNTS	SAMPLE NUMBER	☐ PENETRATION RESISTANCE BLOWS/FT 10 20 30 40 ○ WATER CONTENT (MC) ❙ ATTERBERG LL/PL 20 40 60 80	USCS GRAPHIC	USCS	MATERIAL DESCRIPTION	REMARKS AND TESTS	BACKFILL/INSTALL.
				7	50/15" (Refusal)	SPT_6	☐			Very dense, wet, gray, Silty GRAVEL with sand, boulders, and cobbles (GM); low plasticity fines, trace boulders, trace subrounded to rounded cobbles, subangular coarse gravel, medium to coarse sand. [Alluvial Deposits]		
				100		R-7						
450		35		79	50/5" (Refusal)	SPT_7			GM	*Below 35.5 feet, cobble percentage increases.*		
				100		R-8						
445		40		54	43-50/5" (Refusal)	SPT_8					Borehole completed at 40.92 feet below ground surface (bgs).	
440		45										
435		50										
430		55										

McMILLEN JACOBS ASSOCIATES

Boring B-02

Project: Geren Island Water Treatment Facility
Project Location: 2700 E. Santiam St. Stayton, Oregon 97203
Project Number: 5966.0

Log of Boring B-03

Date(s) Drilled	**04/10/2019**	Geotechnical Consultant	**McMillen Jacobs Associates**	Logged By	**J. Fissel**	Checked By	**J. Quinn**
Drilling Method/ Rig Type	**Sonic Drilling/TSi 150CC Track-Mounted Drill Rig**	Drilling Contractor	**Holt Services Inc.**	Total Depth of Borehole	**40.8 ft**		
Hole Diameter	**6.00 in**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**485.0 ft** (approximate)		
Location	**South-Central Perimeter of Filter #3 West**	Coordinates		Elevation Source	**30% Submittal Drawings, Aug 2019**		

PENETRATION RESISTANCE BLOWS/FT ☐ 10 20 30 40; ○ WATER CONTENT (MC); ❙ ATTERBERG LL/PL 20 40 60 80

ELEV. (FT)	WATER LEVEL	DEPTH (FT)	RECOVERY (%)	BLOW COUNTS	SAMPLE NUMBER	USCS	MATERIAL DESCRIPTION	REMARKS AND TESTS
						ML ML	Dark brown, Sandy SILT (ML); trace fine gravel, roots. [Fill]	
			100		R-1		Soft, brown, Sandy SILT with gravel (ML); low plasticity, subrounded, coarse gravel, fine to coarse sand. [Fill]	
480		5				GM	Very dense, moist, brown, Silty GRAVEL with sand and cobbles (GM); low plasticity fines, trace subrounded to rounded cobbles, subrounded, coarse gravel, fine to coarse sand.	
			50	5-40-20 (N=60)	SPT_1			
			100		R-2	GP	[Fill]	
							Dense, moist, gray, Poorly graded GRAVEL with sand (GP); poorly graded, subrounded, coarse gravel, fine to coarse sand. [Fill]	
475		10						9% Cobbles, 56% Gravel, 27% Sand, 8% Fines
			50	6-15-34 (N=49)	SPT_2		Dense, moist, brown, Well-graded GRAVEL with silt, sand, and cobbles (GW-GM); low plasticity fines, subrounded to rounded cobbles, subrounded gravel, fine to coarse sand, weakly cemented.	
			100 100		G-1 R-3	GW-GM	[Fill]	
470		15					*Becomes wet at 15 feet.*	
			36	22-50/5" (Refusal)	SPT_3			
	▽							Water Level inside borehole at 16.3 feet bgs after drilling
			100		R-4			
							Dense, wet, gray, Poorly graded GRAVEL with sand and cobbles (GP); low plasticity fines, trace subrounded to rounded cobbles, poorly graded, subrounded, coarse gravel, fine to coarse sand, trace silt, weakly cemented.	
465		20						
			67	26-15-20 (N=35)	SPT_4	GP		
			75		R-5		[Alluvial Deposits]	
							Very dense, wet, brown, Silty GRAVEL with sand (GM); low plasticity fines, subrounded, coarse gravel, trace gray gravel, medium to coarse sand, weakly cemented. [Alluvial Deposits]	
460		25						
			50	41-49-50/ 4" (Refusal)	SPT_5			
			100		R-6		*At 29 feet encountered some red-brown fine, angular gravel*	

McMILLEN JACOBS ASSOCIATES

Project: Geren Island Water Treatment Facility
Project Location: 2700 E. Santiam St. Stayton, Oregon 97203
Project Number: 5966.0

Log of Boring B-03

Date(s) Drilled	**04/10/2019**	Geotechnical Consultant	**McMillen Jacobs Associates**	Logged By	**J. Fissel**	Checked By	**J. Quinn**
Drilling Method/ Rig Type	**Sonic Drilling/TSi 150CC Track-Mounted Drill Rig**	Drilling Contractor	**Holt Services Inc.**	Total Depth of Borehole	**40.8 ft**		
Hole Diameter	**6.00 in**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**485.0 ft** (approximate)		
Location	**South-Central Perimeter of Filter #3 West**	Coordinates		Elevation Source	**30% Submittal Drawings, Aug 2019**		

ELEV. (FT)	WATER LEVEL	DEPTH (FT)	SAMPLE TYPE	RECOVERY (%)	BLOW COUNTS	SAMPLE NUMBER	☐ PENETRATION RESISTANCE BLOWS/FT 10 20 30 40 ○ WATER CONTENT (MC) ❙ ATTERBERG LL/PL 20 40 60 80	USCS GRAPHIC	USCS	MATERIAL DESCRIPTION	REMARKS AND TESTS	BACKFILL/INSTALL.
				100	50/4" (Refusal)	SPT_6				Very dense, wet, brown, Silty GRAVEL with sand (GM); low plasticity fines, subrounded, coarse gravel, trace gray gravel, medium to coarse sand, weakly cemented. [Alluvial Deposits]		
				100		R-7						
450		35		75	38-50/2" (Refusal)	SPT_7			GM			
				100		R-8						
445		40		60	8-50/4" (Refusal)	SPT_8						
											Borehole completed at 40.83 feet below ground surface (bgs).	
440		45										
435		50										
430		55										

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

FIELD LOG OF TEST PIT 1

JOB NO. 0-1854-01 **DATE** 7/8/86 **LOCATION** See Fig. 1
PROJECT Geren Island Water Intake Facilities
INSPECTOR D. Hilts **WEATHER** Sunny, warm

SOIL DESCRIPTION & REMARKS	GROUND WATER	SAMPLES
Silty SAND - Dense, brown, fine to medium.	None	
Sandy GRAVEL - Fine to coarse.	1.6	
Sandy GRAVEL - Dense, brown, coarse, subround gravel with occasional cobbles. Sand fraction fine to medium, trace of silt.	2.0	Bag
Sandy GRAVEL - Silty, fine, subround.	4.0	
Sandy GRAVEL - Same as 2-4 ft.	4.3	
Silty SAND - Dense, brown and gray-brown, fine, occasional fine gravel.	4.7	
Bottom of test pit	6.0	

SKETCH OF S **PIT SIDE** **SURFACE ELEVATION** 479.8 ft.

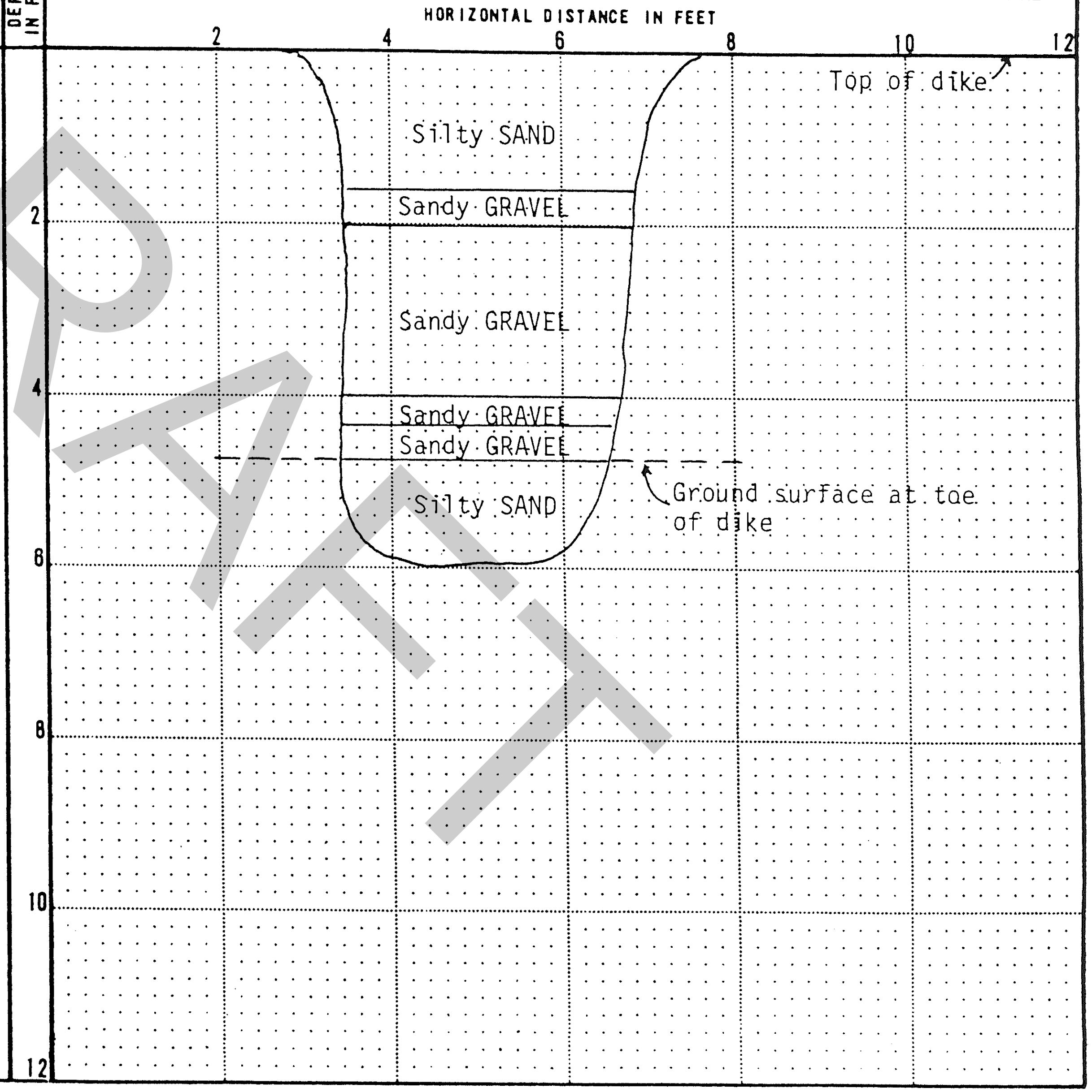


FIG. 2

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

FIELD LOG OF TEST PIT 2

JOB NO. 0-1854-01 DATE 7/8/86 LOCATION See Fig. 1
PROJECT Geren Island Water Intake Facilities
INSPECTOR D. Hilts WEATHER Sunny, warm

SOIL DESCRIPTION & REMARKS	GROUND WATER	SAMPLES
Sandy GRAVEL - Dense, brown, fine to coarse, subround gravel and fine to medium sand. Trace of silt. Many cobbles, occasional voids. Many roots in upper 2 ft.	None	1
SAND & GRAVEL - Gray, fine to coarse.	2.5	
SAND - Dense, brown, fine to medium with some coarse, slightly silty. Occasional pieces of fine to coarse gravel.	3.1	2
Bottom of test pit	7.5	

SKETCH OF N PIT SIDE SURFACE ELEVATION 479.9 ft.

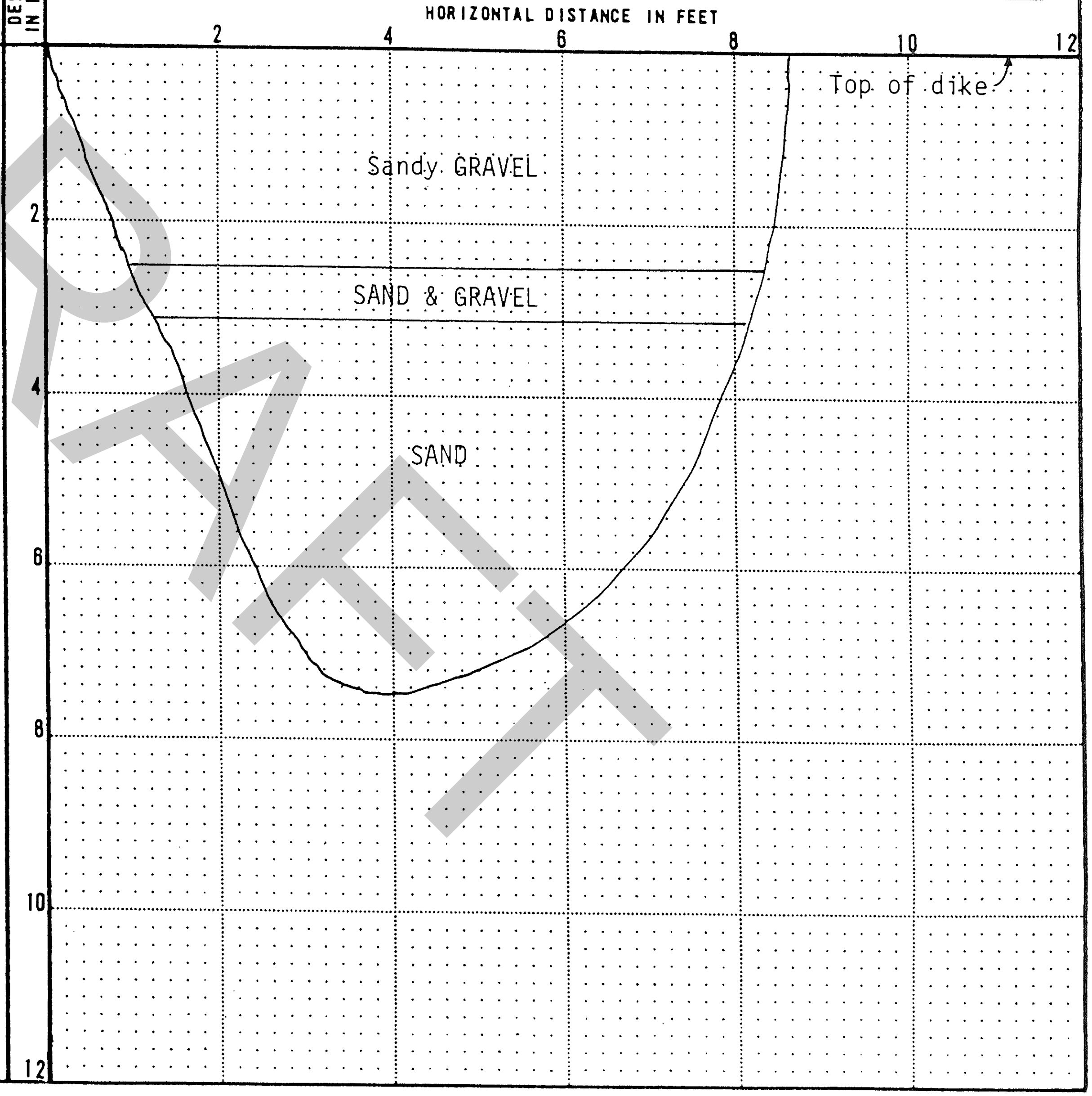


FIG. 3

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

FIELD LOG OF TEST PIT 3

JOB NO. 0-1854-01 DATE 7/8/86 LOCATION See Fig. 1

PROJECT Geren Island Water Intake Facilities

INSPECTOR D. Hilts WEATHER Sunny, warm

SOIL DESCRIPTION & REMARKS	GROUND WATER	SAMPLES
Silty SAND - Loose to medium dense, brown, fine, trace of gravel, occasional roots.	None	
	1.5	
Silty SAND - Dense, gray with brown mottling, fine to medium, occasional pieces of wood.		
	2.5	
Silty SAND - Medium dense to dense, dark gray, fine to medium with some coarse. Occasional coarse gravel and cobbles.		Bag
	7.5	
Bottom of test pit		
Note: Test pit located in area that reportedly washed out during flooding in 1965.		

SKETCH OF W PIT SIDE

SURFACE ELEVATION 480.0 ft.

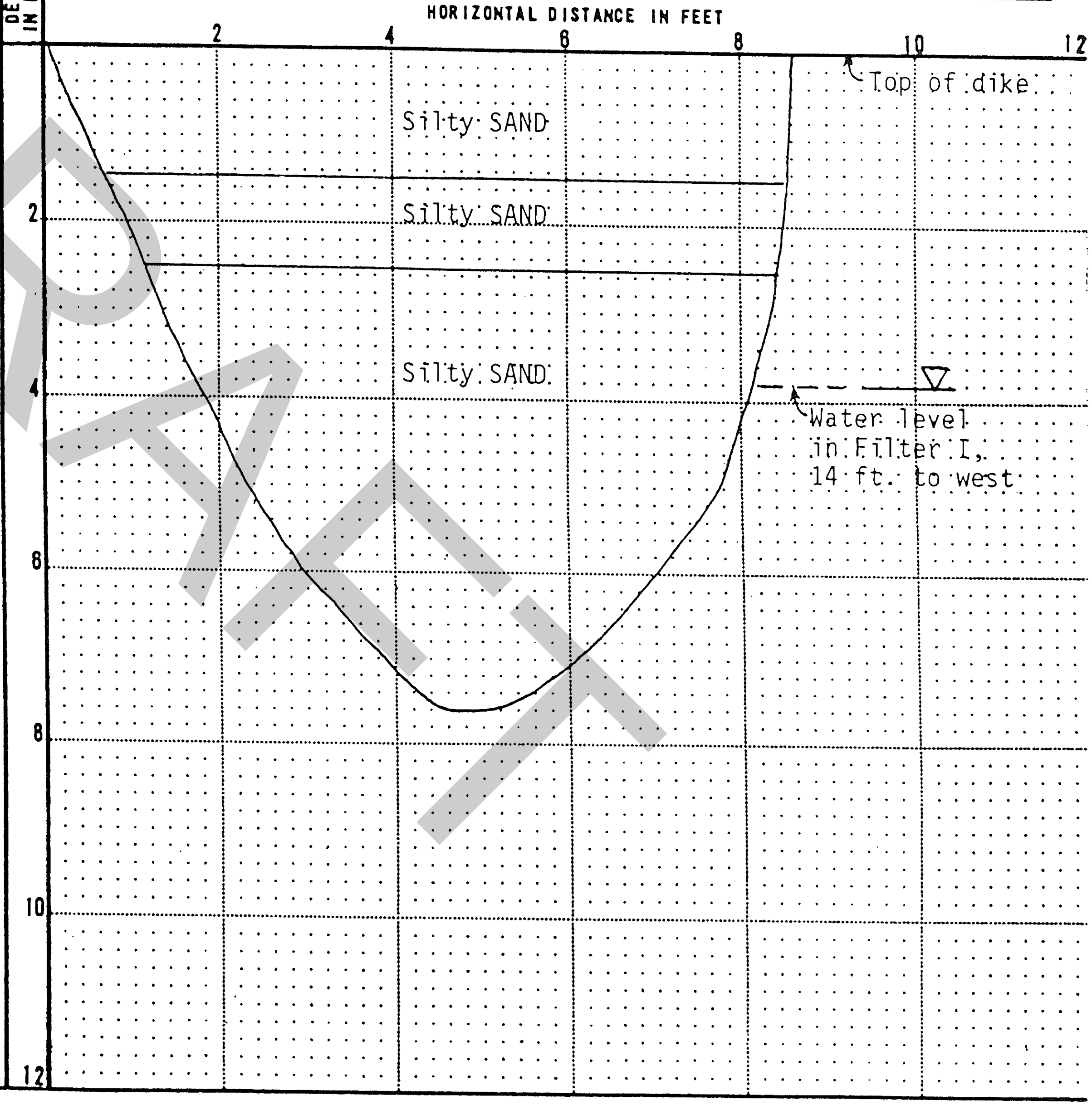


FIG. 4

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

FIELD LOG OF TEST PIT 4

JOB NO. 0-1854-01 **DATE** 7/8/86 **LOCATION** See Fig. 1
PROJECT Geren Island Water Intake Facilities
INSPECTOR D. Hilts **WEATHER** Sunny, warm

SOIL DESCRIPTION & REMARKS	GROUND WATER	SAMPLES
Sandy GRAVEL - Loose to medium dense, brown, coarse, subrounded gravel and fine to medium sand, slightly silty.	▽	
Bottom of test pit	5.6	

SKETCH OF E **PIT SIDE** **SURFACE ELEVATION** 478.8 ft.

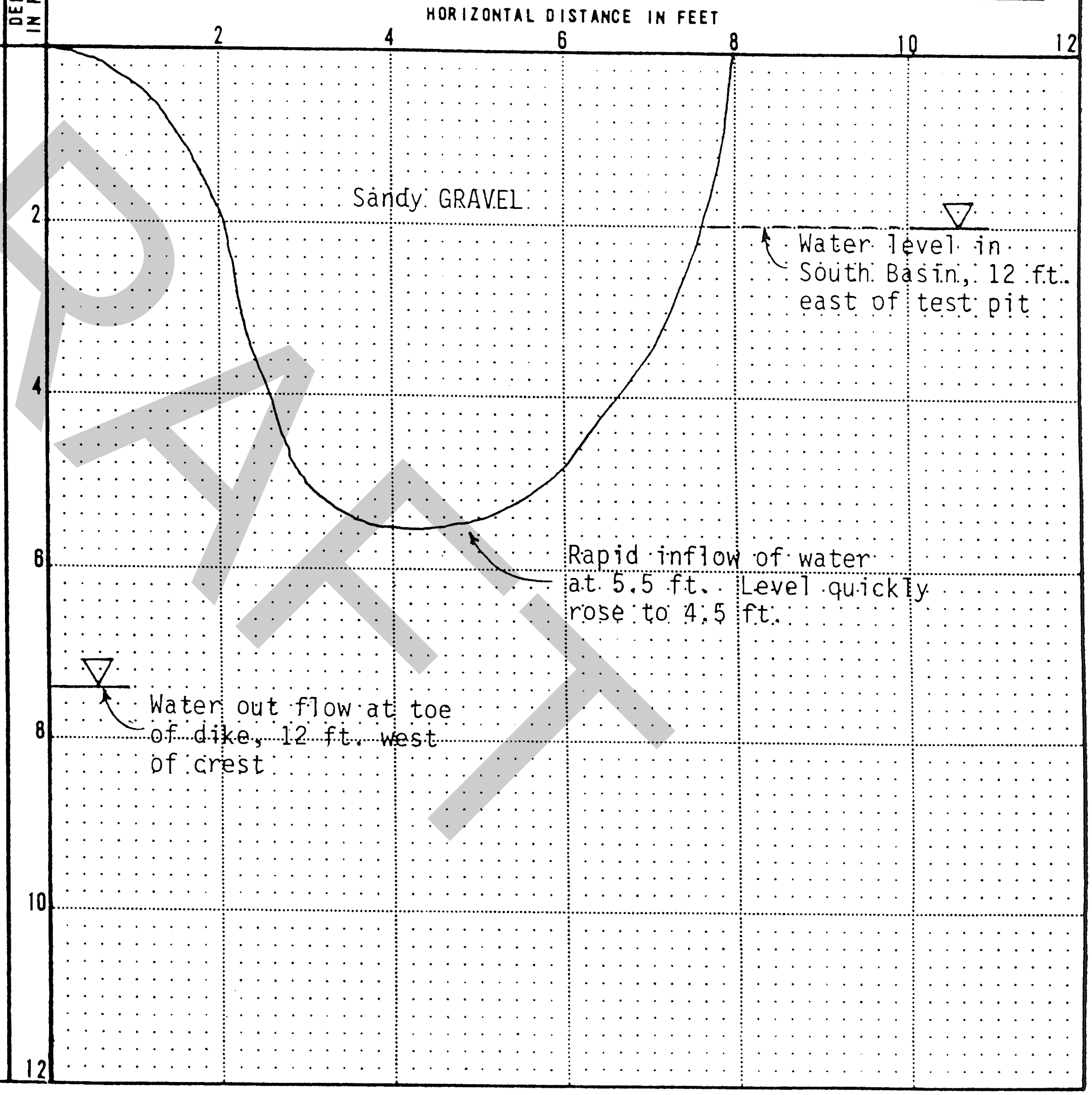


FIG. 5

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

FIELD LOG OF TEST PIT 5

JOB NO. 0-1854-01 **DATE** 7/8/86 **LOCATION** See Fig. 1
PROJECT Geren Island Water Intake Facilities
INSPECTOR D. Hilts **WEATHER** Sunny, warm

SOIL DESCRIPTION & REMARKS	GROUND WATER	SAMPLES
Sandy GRAVEL - Medium dense, brown, fine to coarse, subround gravel and fine to coarse sand, trace of silt. Abundant cobbles and occasional boulders to 16 ins.	▽	
Bottom of test pit	5.0	
Note: Pit located at south abutment of new intake structure; north abutment was inaccessible.		

SKETCH OF W **PIT SIDE** **SURFACE ELEVATION** 480.0 ft.

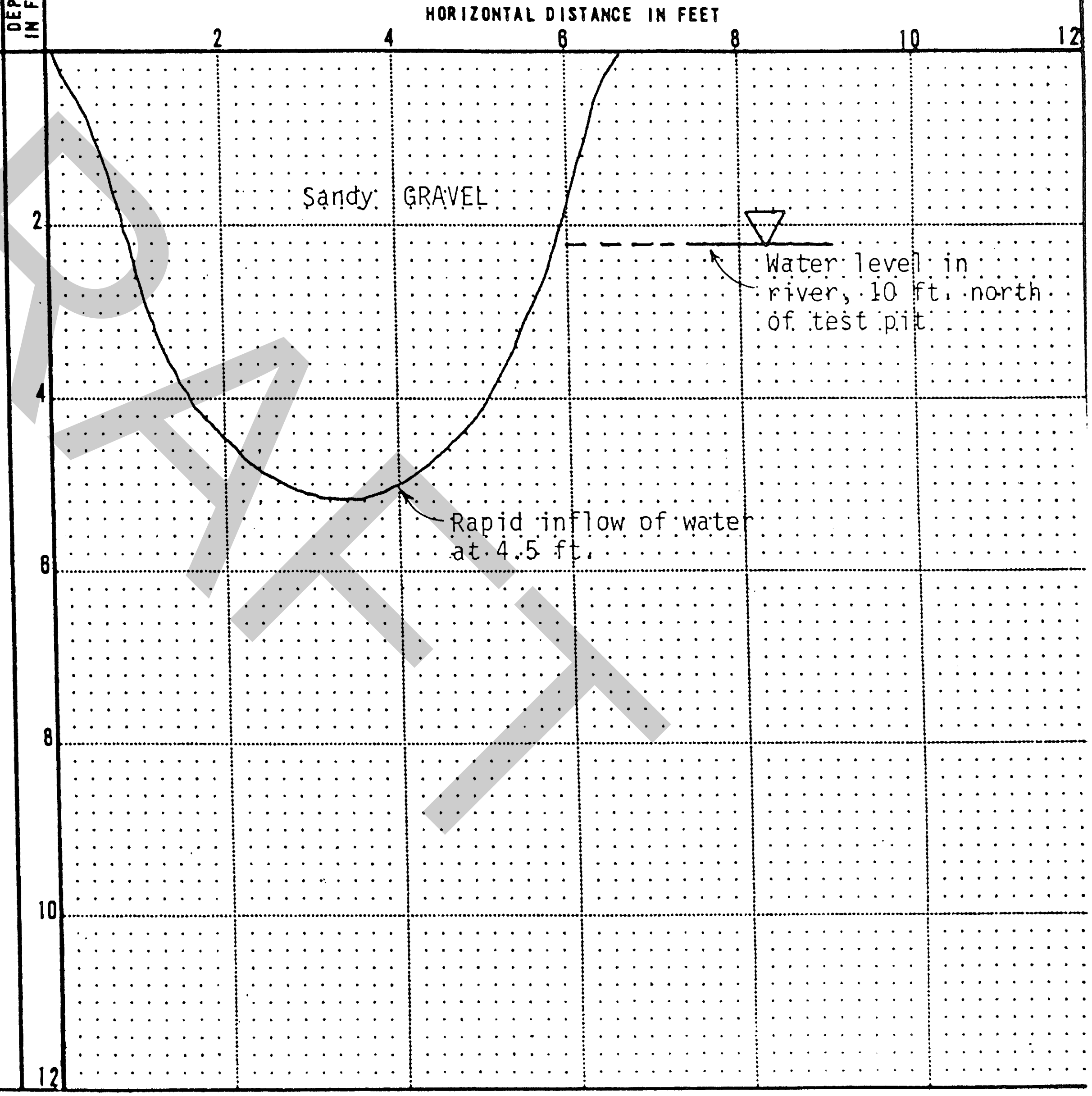
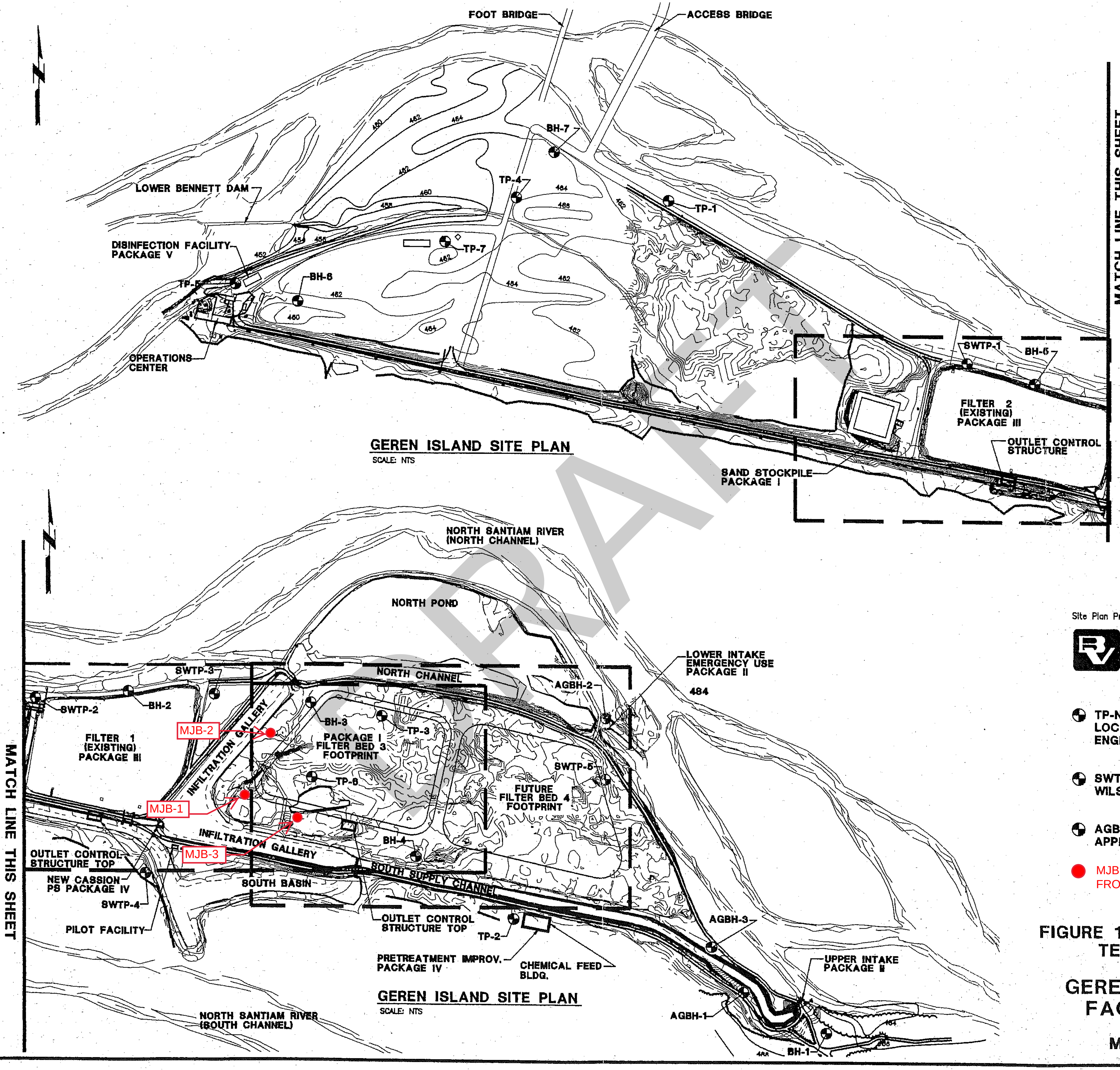


FIG. 6



Site Plan Provided By:

Black & Veatch

⊕ TP-NO. OR BH-NO. TEST PIT OR BOREHOLE LOCATION CONDUCTED BY FOUNDATION ENGINEERING, INC. JULY 1996

⊕ SWTP-NO. TEST PIT FROM SHANNON & WILSON STUDY IN 1986.

⊕ AGBH-NO. BOREHOLE LOCATION FROM APPLIED GEOTECHNOLOGY STUDY IN 1993.

● MJB-NO. APPROXIMATE BOREHOLE LOCATION FROM MCMILLEN JACOBS STUDY IN 2019.

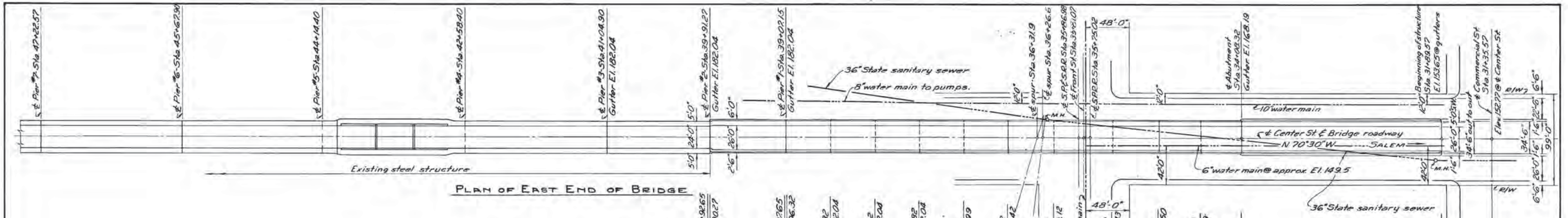
FIGURE 1. SITE PLAN (LOCATIONS OF TEST PITS AND BORINGS).

GEREN ISLAND TREATMENT FACILITY IMPROVEMENTS

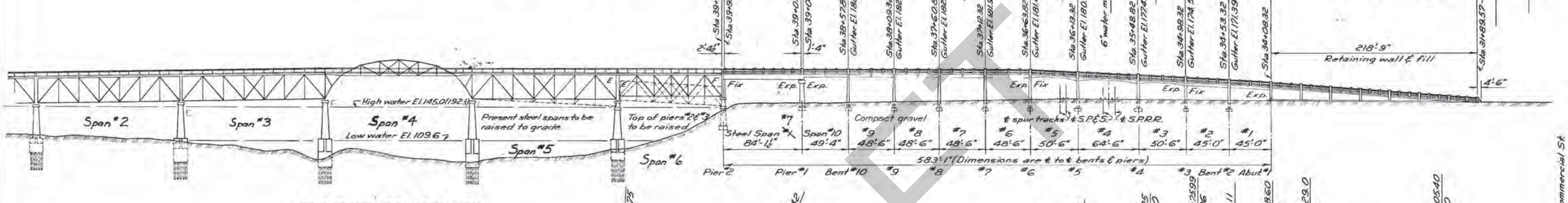
MARION COUNTY, OREGON

APPENDIX N

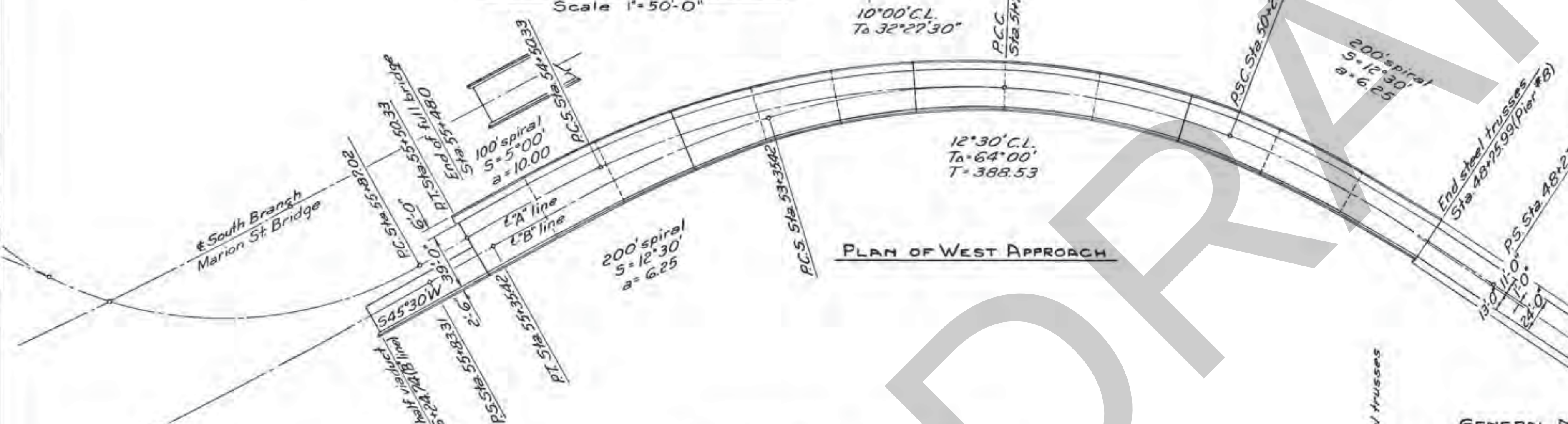
EXISTING INFORMATION
CENTER STREET BRIDGE CROSSING



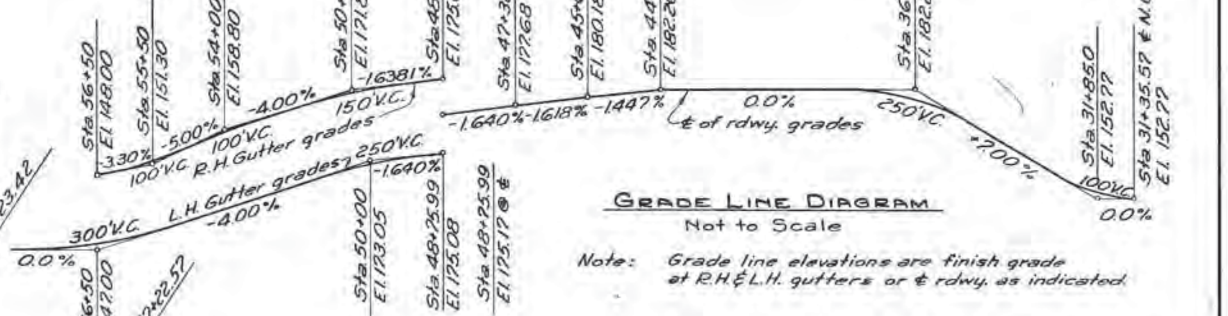
PLAN OF EAST END OF BRIDGE



ELEVATION OF EAST END
Scale 1" = 50'-0"

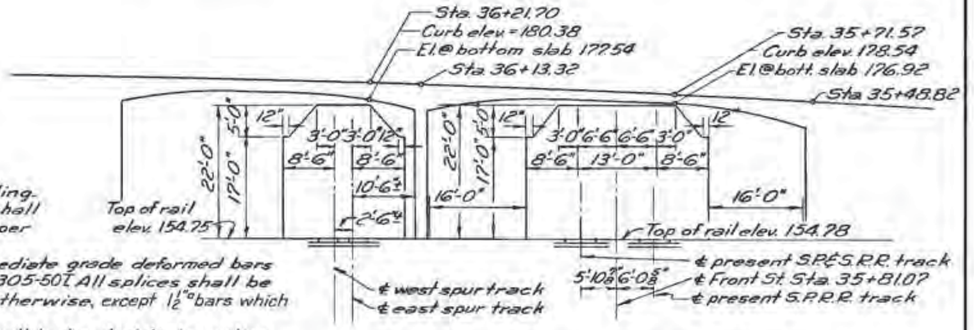


PLAN OF WEST APPROACH



GRADE LINE DIAGRAM
Not to Scale

Note: Grade line elevations are finish grade at R.H. & L.H. gutters or ℄ rdwy. as indicated.



CLEARANCE DIAGRAM AT FRONT ST.
Scale 1" = 15'-0"

GENERAL NOTES:

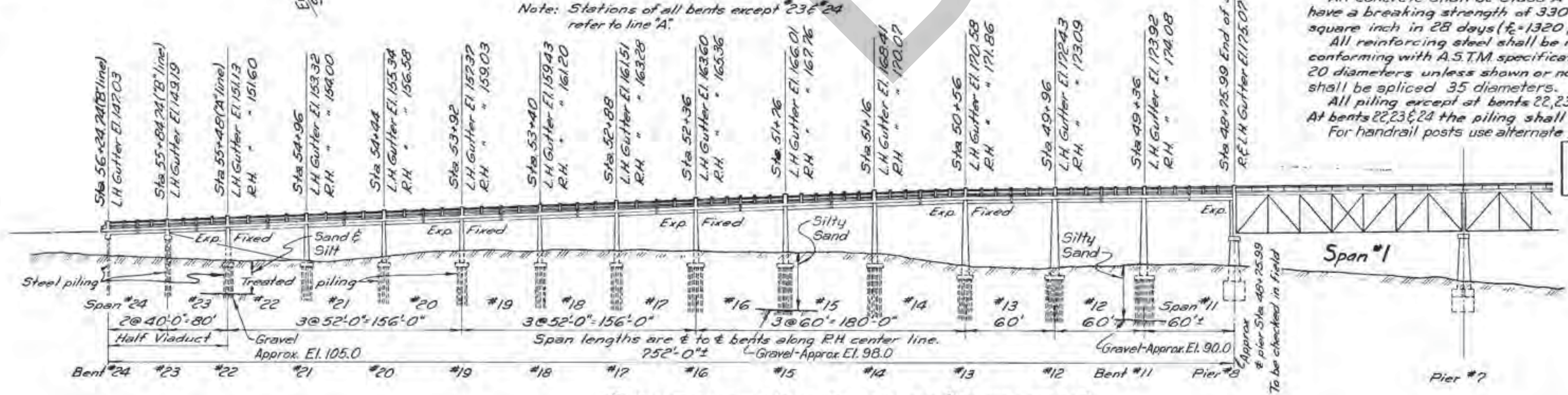
Bridge designed for H20-S16-44 loading.

All concrete shall be Class "A" and shall have a breaking strength of 3300 lbs. per square inch in 28 days (f_c = 1320 p.s.i.)

All reinforcing steel shall be intermediate grade deformed bars conforming with A.S.T.M. specification A305-50T. All splices shall be 20 diameters unless shown or noted otherwise, except 1½" ⌀ bars which shall be spliced 35 diameters.

All piling except at bents 22, 23 & 24 shall be treated timber piling. At bents 22, 23 & 24 the piling shall be 12" BP53# steel piling.

For handrail posts use alternate post cap.



ELEVATION OF WEST APPROACH (DEVELOPED)
Scale 1" = 50'

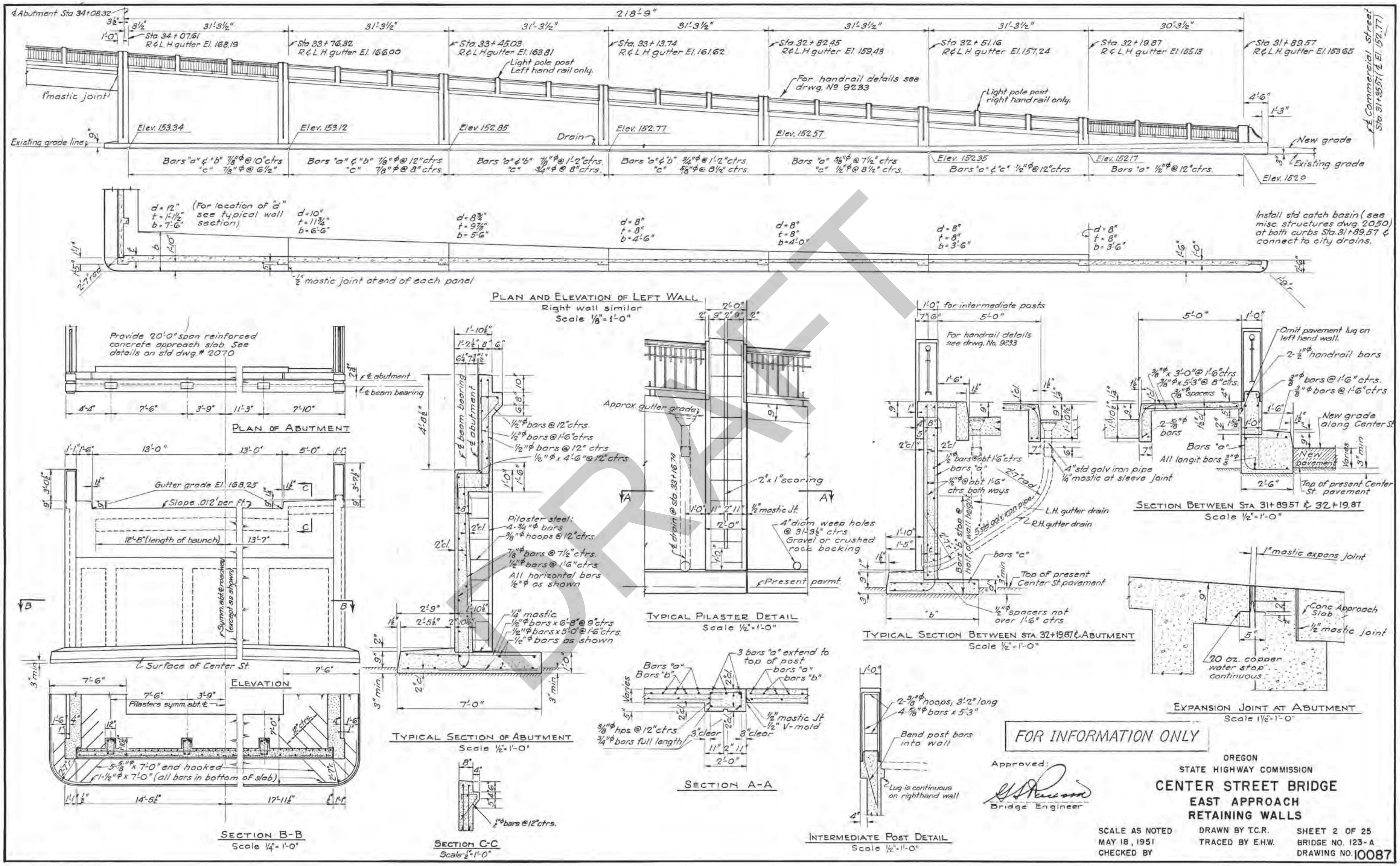
FOR INFORMATION ONLY

OREGON
STATE HIGHWAY COMMISSION
CENTER ST. BRIDGE
OVER
WILLAMETTE RIVER
AT SALEM - MARION & POLK CO.

Approved:
Bridge Engineer

SCALES NOTED
OCTOBER 15, 1951
CALC. BK. 254 & 275
DRAWN BY T.C.R.
TRACED BY C.O.F.
CHECKED BY
SHEET 1 OF 25
BRIDGE NO. 123A
DRAWING NO. 10086
ACCOMPANIED BY DRWGS. NO. 10087 TO 10108 INCL., 2050 & 9233.

3-11-69 Rev. Span Nos.



FOR INFORMATION ONLY

Approved:

G S Paxson

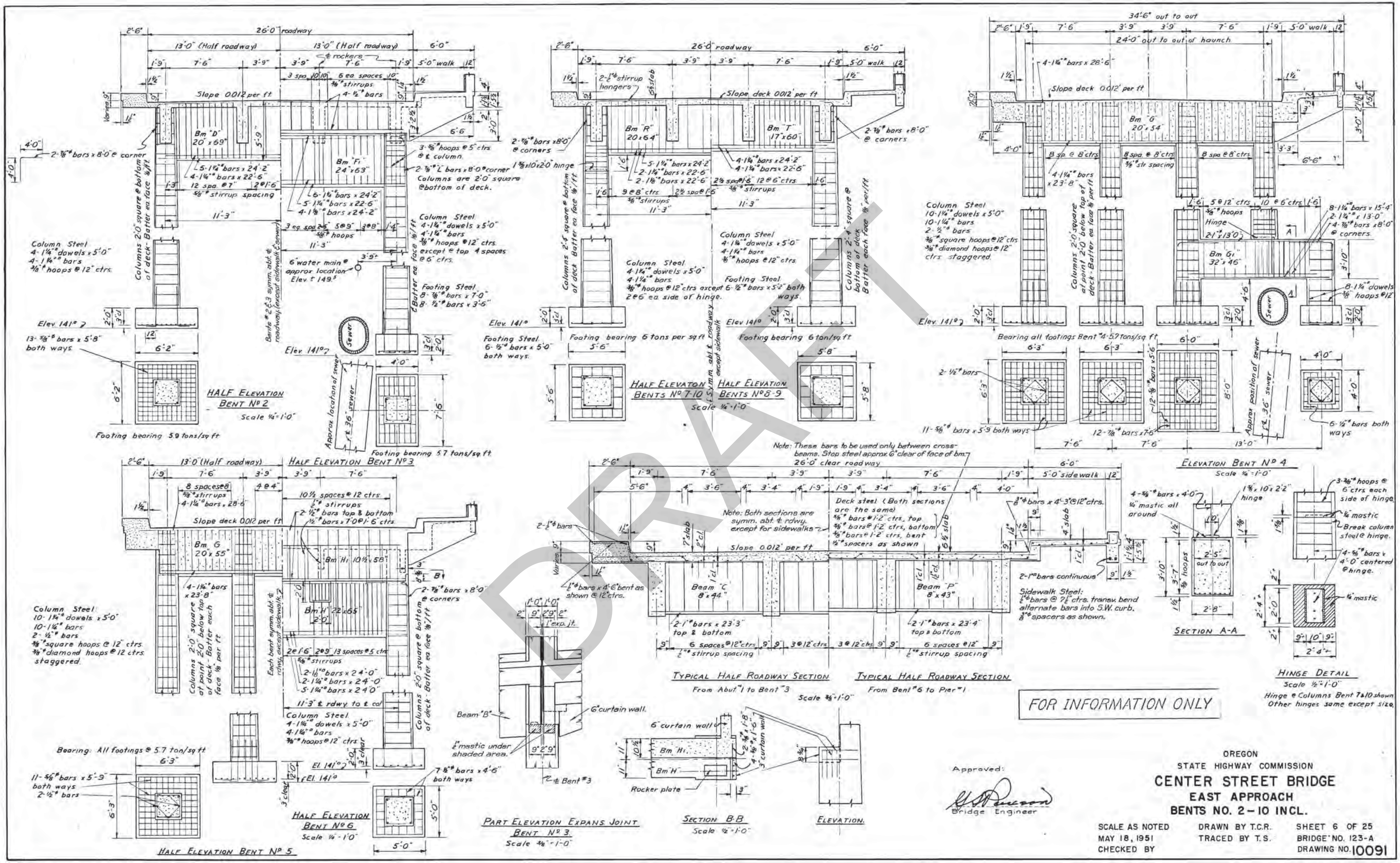
Bridge Engineer

OREGON
STATE HIGHWAY COMMISSION
CENTER STREET BRIDGE
EAST APPROACH
RETAINING WALLS

SCALE AS NOTED
MAY 18, 1951
CHECKED BY

DRAWN BY T.C.R.
TRACED BY E.H.W.

SHEET 2 OF 25
BRIDGE NO. 123-A
DRAWING NO. 10087



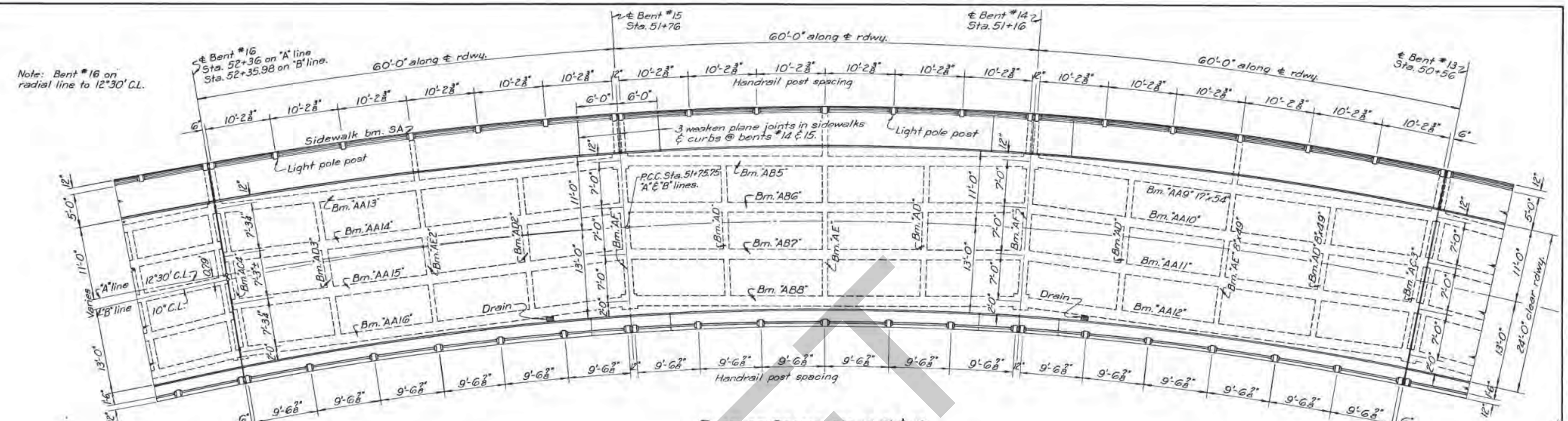
FOR INFORMATION ONLY

OREGON
STATE HIGHWAY COMMISSION

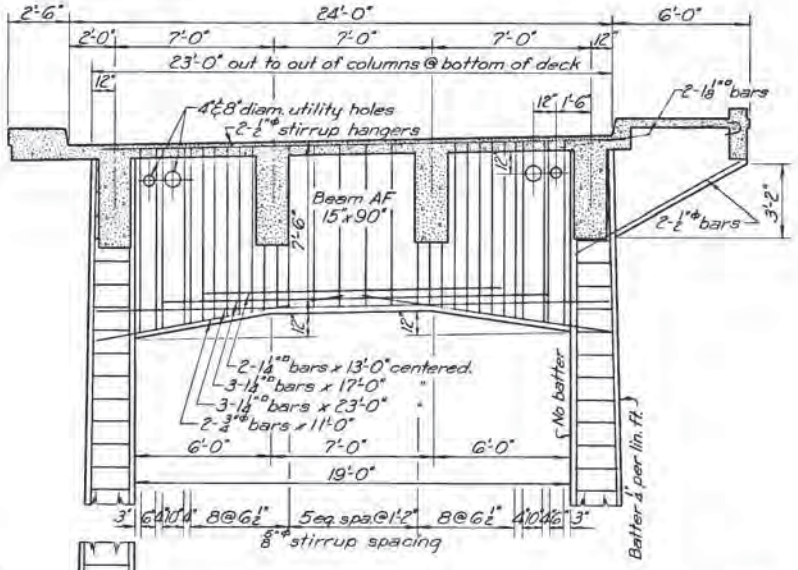
CENTER STREET BRIDGE
EAST APPROACH
BENTS NO. 2 – 10 INCL.

Approved:
G S Paxson
Bridge Engineer

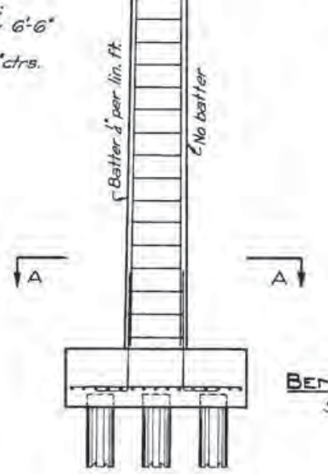
SCALE AS NOTED	DRAWN BY T.C.R.	SHEET 6 OF 25
MAY 18, 1951	TRACED BY T.S.	BRIDGE NO. 123-A
CHECKED BY		DRAWING NO. 10091



PLAN OF SPANS No. 14, 15 & 16

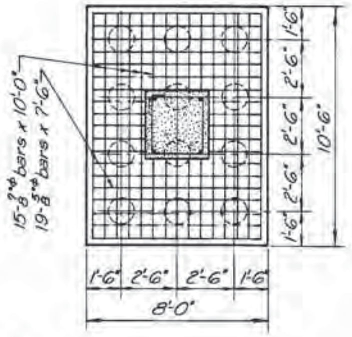


Column Steel:
4-1¼"⌀ dowels x 6'-6"
4-1¼"⌀ bars
⅜"⌀ hoops @ 12" ctrs.



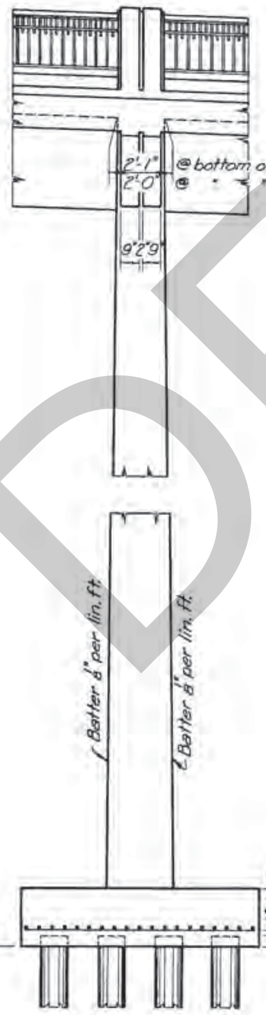
BENTS No. 14 & 15

Scale-¼"=1'-0"

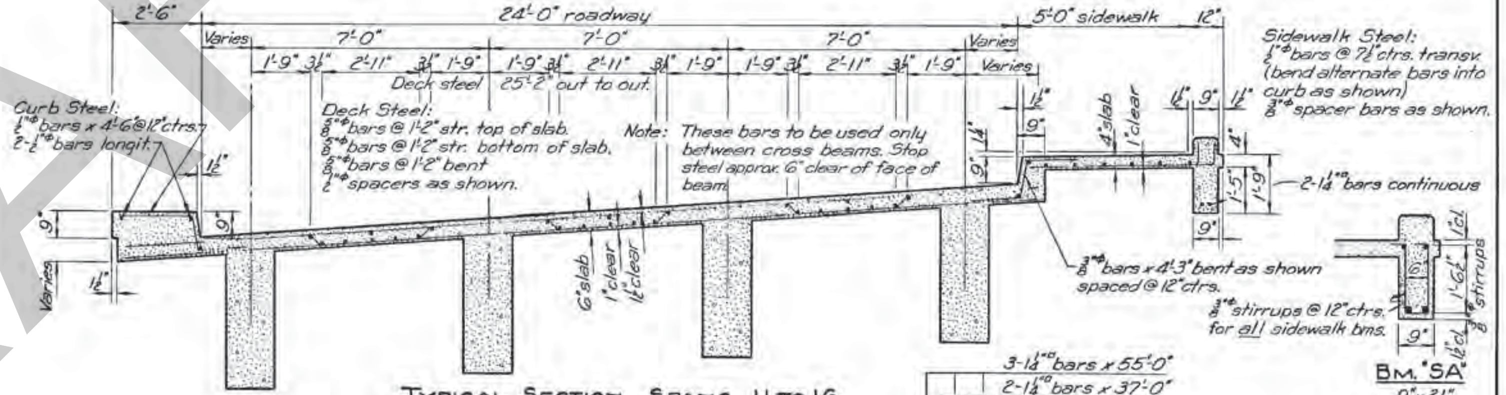


SECTION A-A

Note: All piling to be not less than 1'-2" at butt. Bearing per pile = 23 tons.



END VIEW BENTS No. 14 & 15

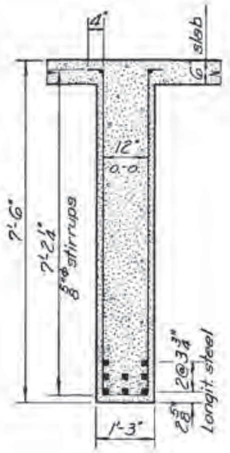


TYPICAL SECTION SPANS 11 TO 16

Scale-⅜"=1'-0"

BM. "SA"

9"x21"

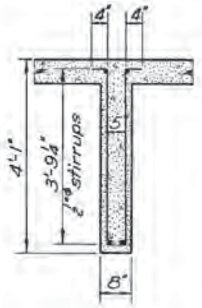


BEAM AF

15"x90"

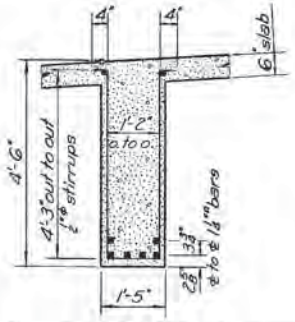
BEAM DETAILS

Scale ½"=1'-0"



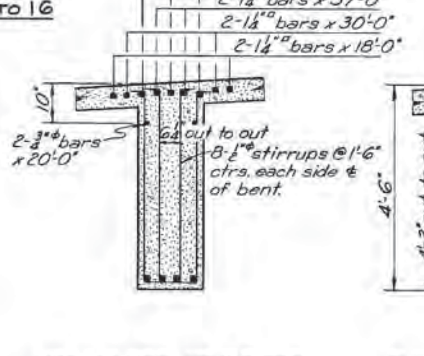
BEAMS AD & AE

8"x49"

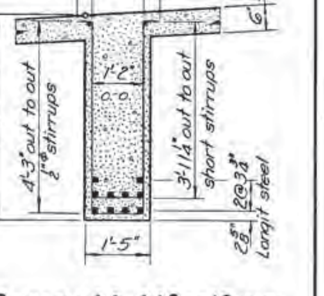


BEAMS AB, AB2 TO 8 INCL.

17"x54"

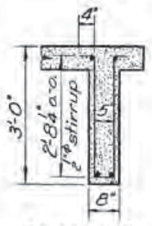


BEAM SEC. AT BENTS No. 11, 12, 14 & 15



BEAMS AA, AA2 TO 16 INCL.

17"x54"



BEAM AC

8"x36"

FOR INFORMATION ONLY

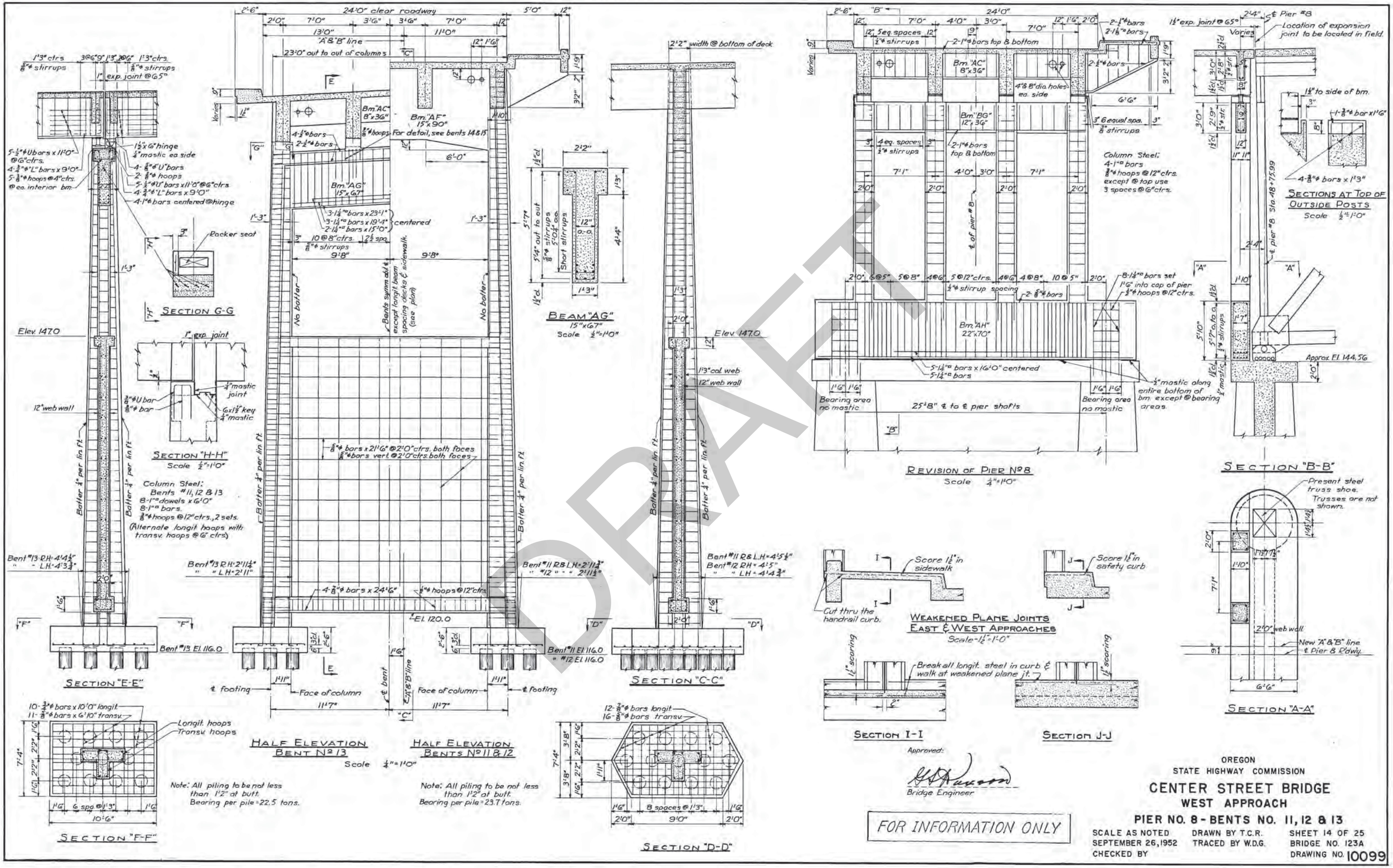
OREGON
STATE HIGHWAY COMMISSION
CENTER STREET BRIDGE
WEST APPROACH
SPANS 14, 15 & 16

Approved:
G S Paxson
Bridge Engineer

SCALE AS NOTED
MARCH 19, 1952
CHECKED BY

DRAWN BY T.C.R.
TRACED BY C.O.F.

SHEET 13 OF 25
BRIDGE NO. 123A
DRAWING NO. 10098



Approved:

G S Paxson

Bridge Engineer

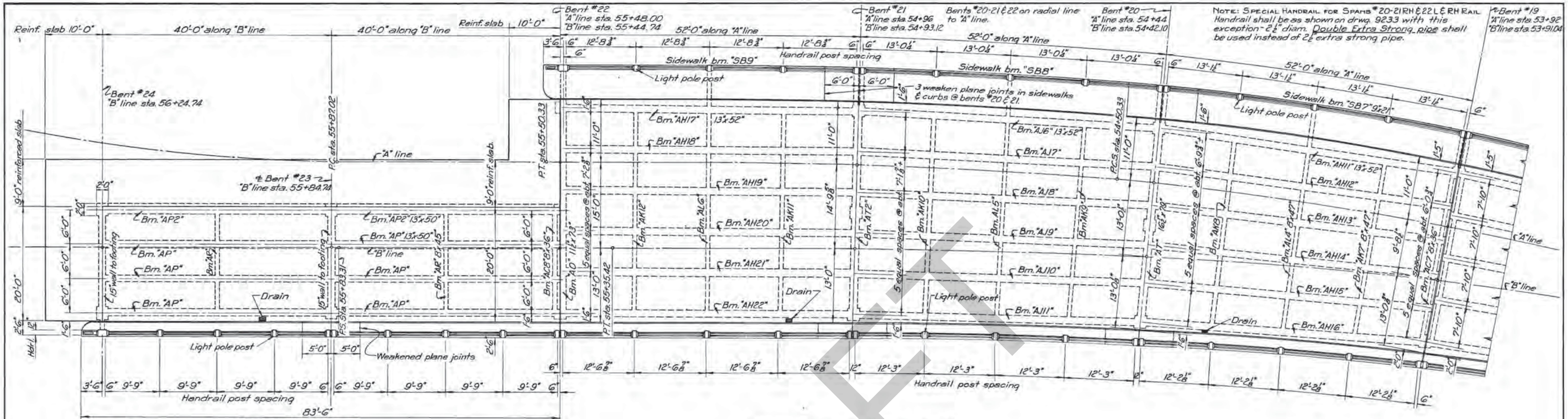
FOR INFORMATION ONLY

OREGON
STATE HIGHWAY COMMISSION
CENTER STREET BRIDGE
WEST APPROACH
PIER NO. 8 - BENTS NO. 11, 12 & 13

SCALE AS NOTED
SEPTEMBER 26, 1952
CHECKED BY

DRAWN BY T.C.R.
TRACED BY W.D.G.

SHEET 14 OF 25
BRIDGE NO. 123A
DRAWING NO. 10099

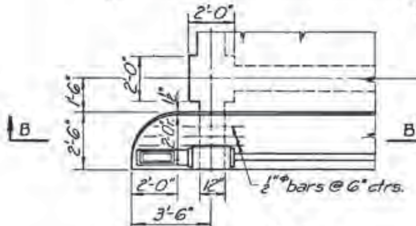


Note: Special Handrail for Spans #20-21 RH & 22 L & RH Rail. Handrail shall be as shown on drwg. 9233 with this exception- 2½" diam. Double Extra Strong pipe shall be used instead of 2½" extra strong pipe.

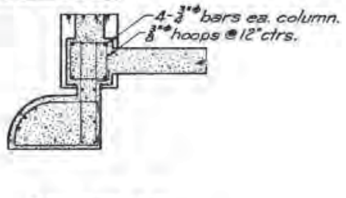
Partial Viaduct

Plan of Spans No. 20-21-22-23 & 24

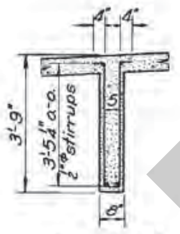
Scale-⅛"=1'-0"



Plan

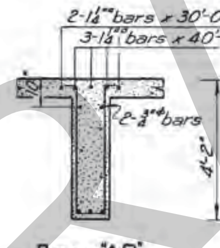


Section A-A



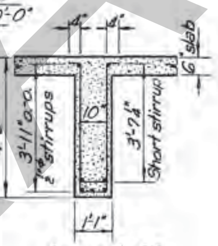
Beam "AR"

8"x48"



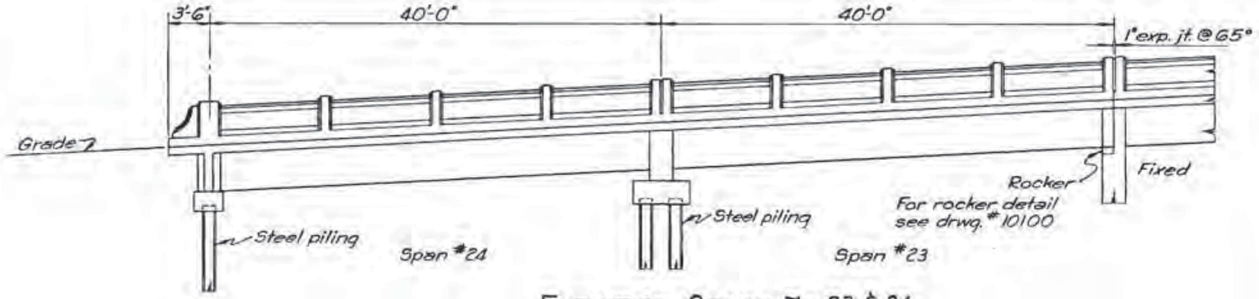
Beam "AP"

At Bent #23



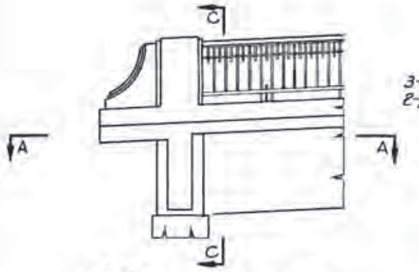
Beam "AP"

13"x50"

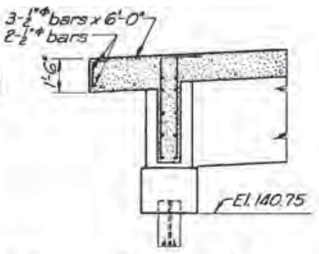


Elevation Spans No. 23 & 24

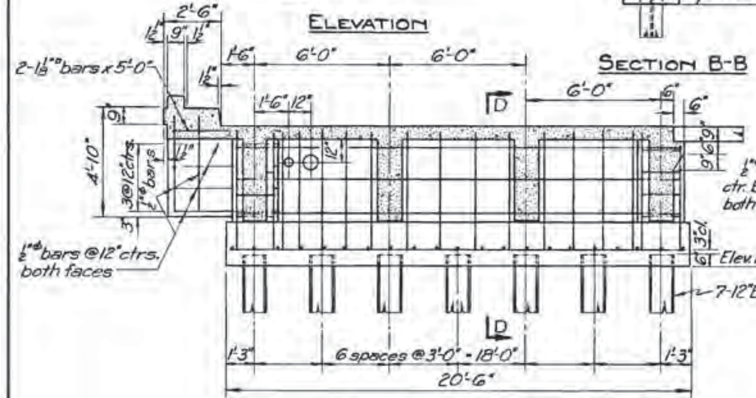
Scale-⅛"=1'-0"



Elevation

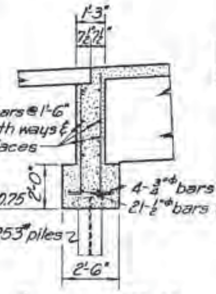


Section B-B

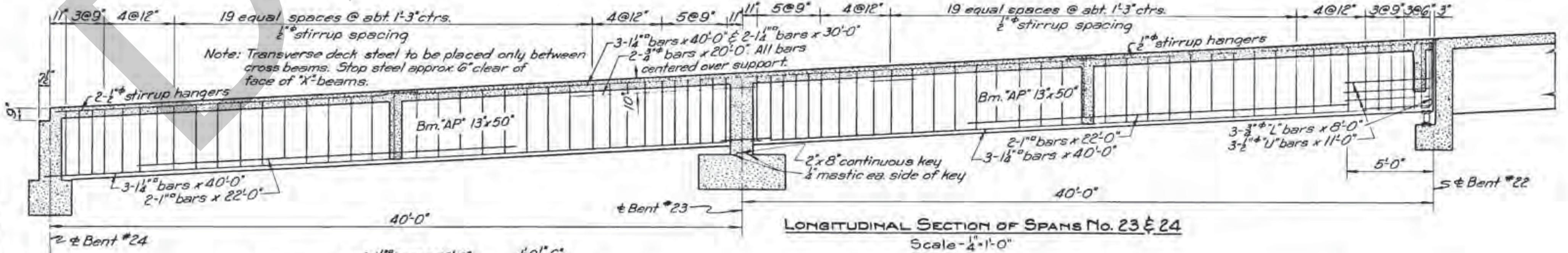


Section C-C Bent No. 24

Scale-¼"=1'-0"



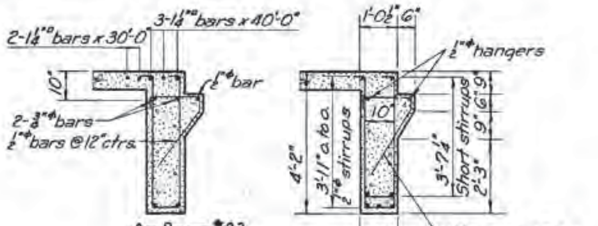
Section D-D



Note: Transverse deck steel to be placed only between cross beams. Stop steel approx. 6" clear of face of "X"-beams.

Longitudinal Section of Spans No. 23 & 24

Scale-¼"=1'-0"



Beam "AP2"

13"x50"

FOR INFORMATION ONLY

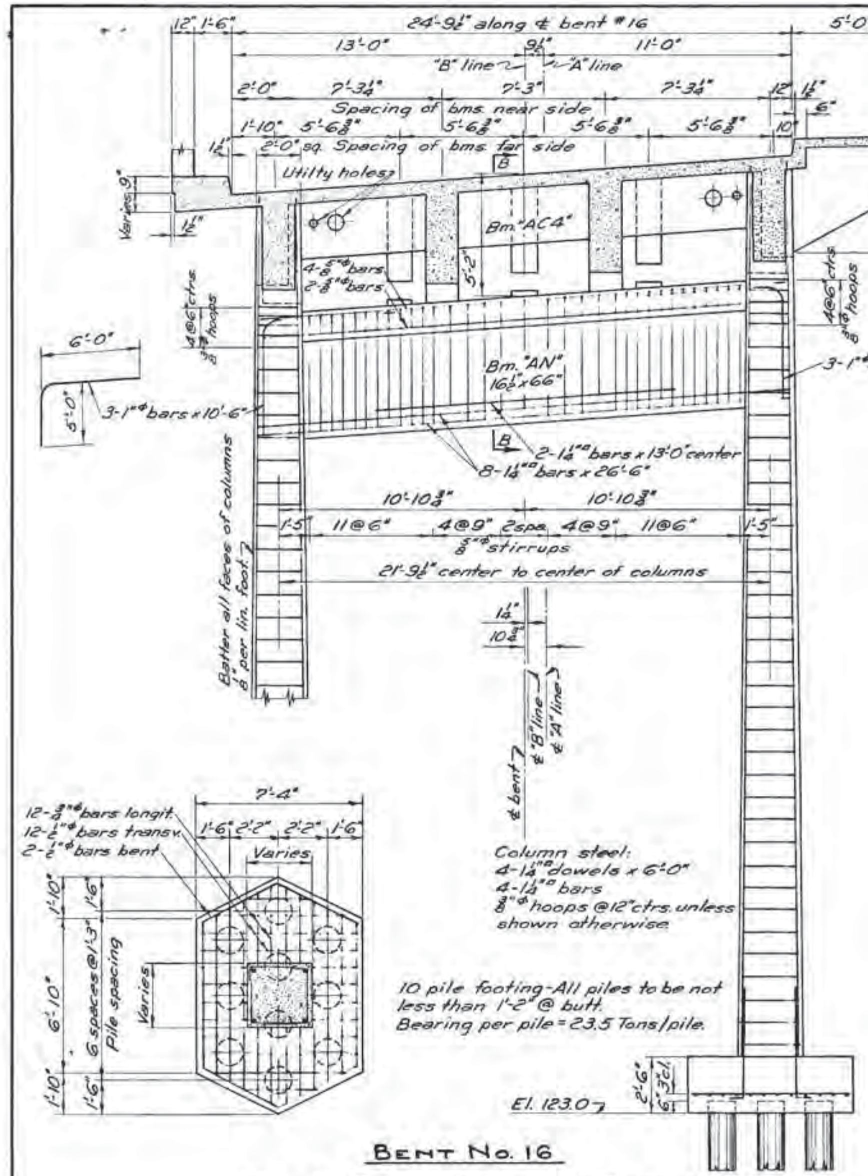
Approved:

Bridge Engineer

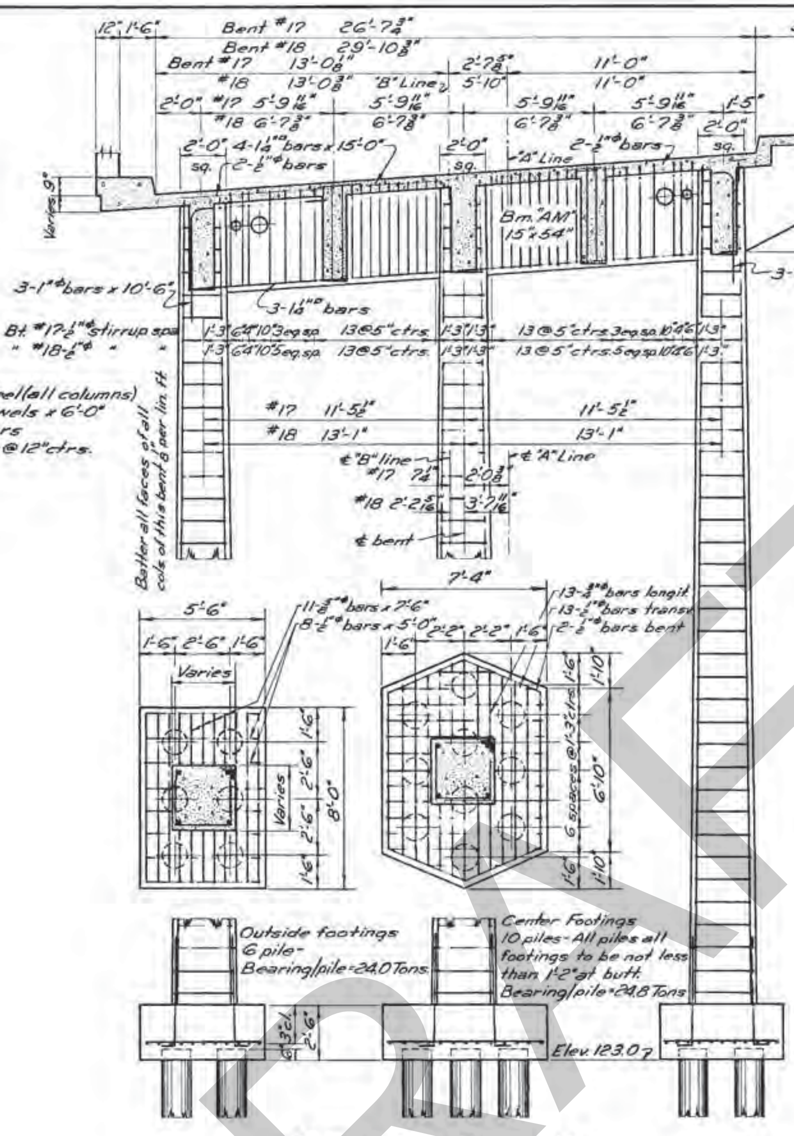
OREGON
STATE HIGHWAY COMMISSION

CENTER STREET BRIDGE
WEST APPROACH
SPANS 20, 21, 22, 23 & 24

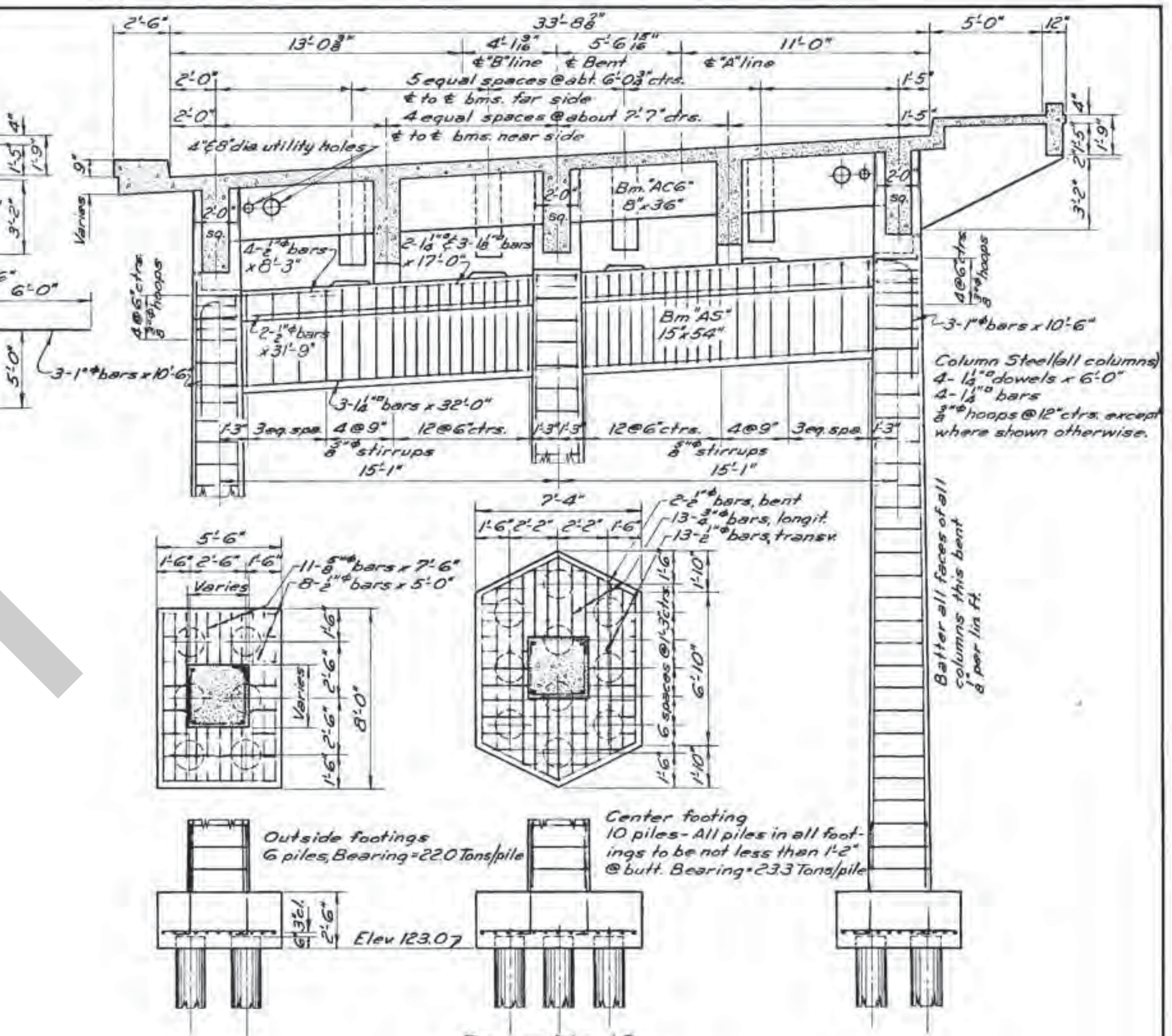
SCALE AS NOTED	DRAWN BY T.C.R.	SHEET 16 OF 25
APRIL 4, 1952	TRACED BY C.O.F.	BRIDGE NO. 123A
CHECKED BY		DRAWING NO. 10101



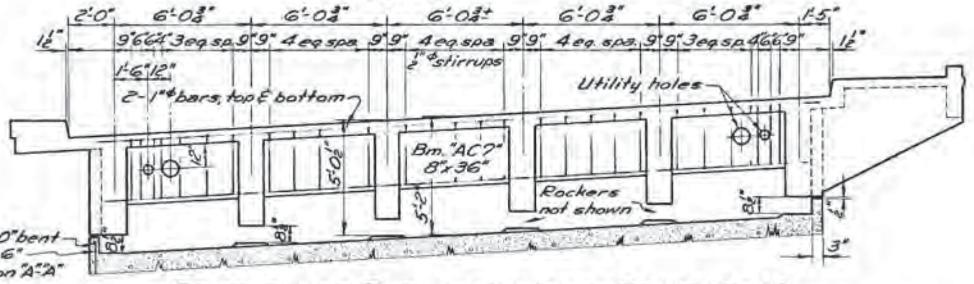
BENT No. 16



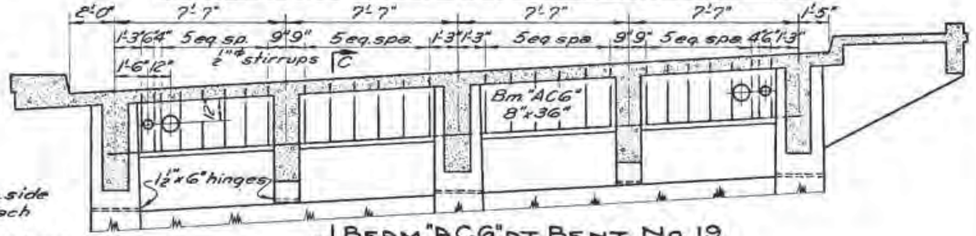
BENTS No. 17 AND 18



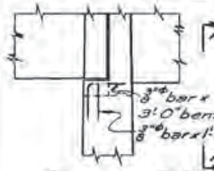
BENT No. 19
Scale ¼"=1'-0"



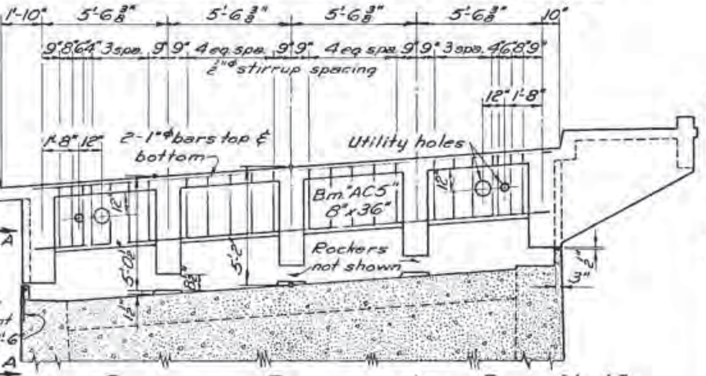
SECTION AT EXPANSION JOINT BENT No. 19



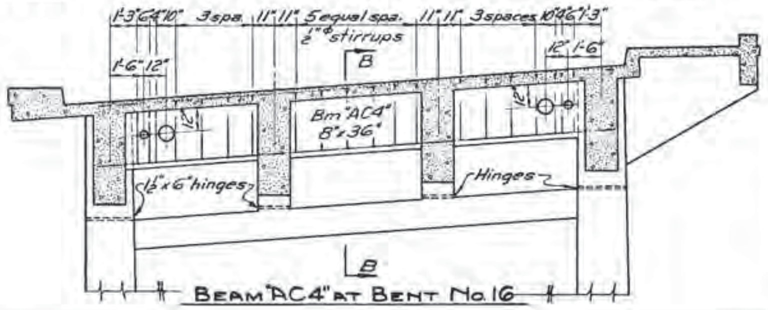
BEAM "AC6" AT BENT No. 19



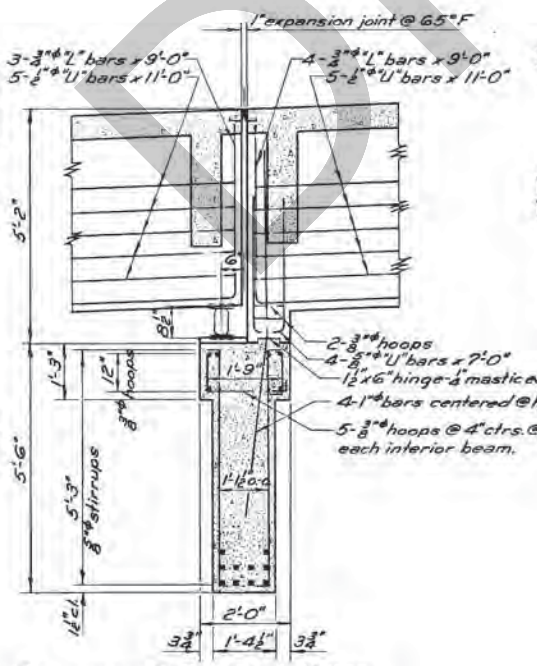
SECTION A-A



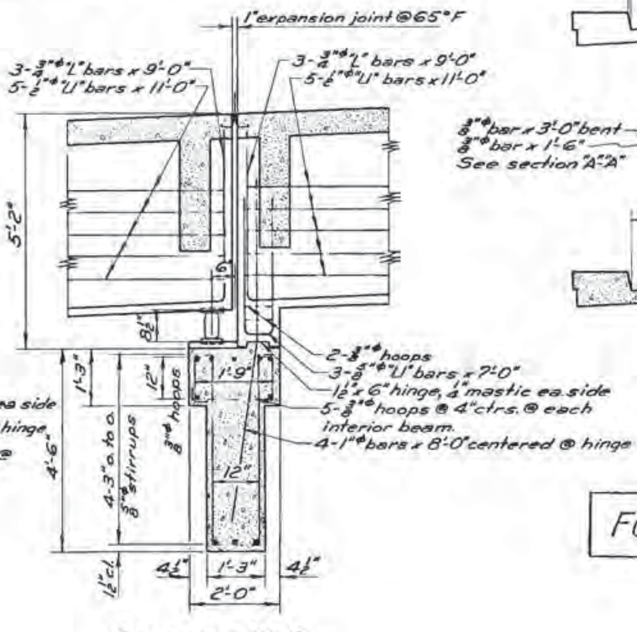
SECTION AT EXPANSION JOINT BENT No. 16



BEAM "AC4" AT BENT No. 16



SECTION B-B Scale ½"=1'-0"



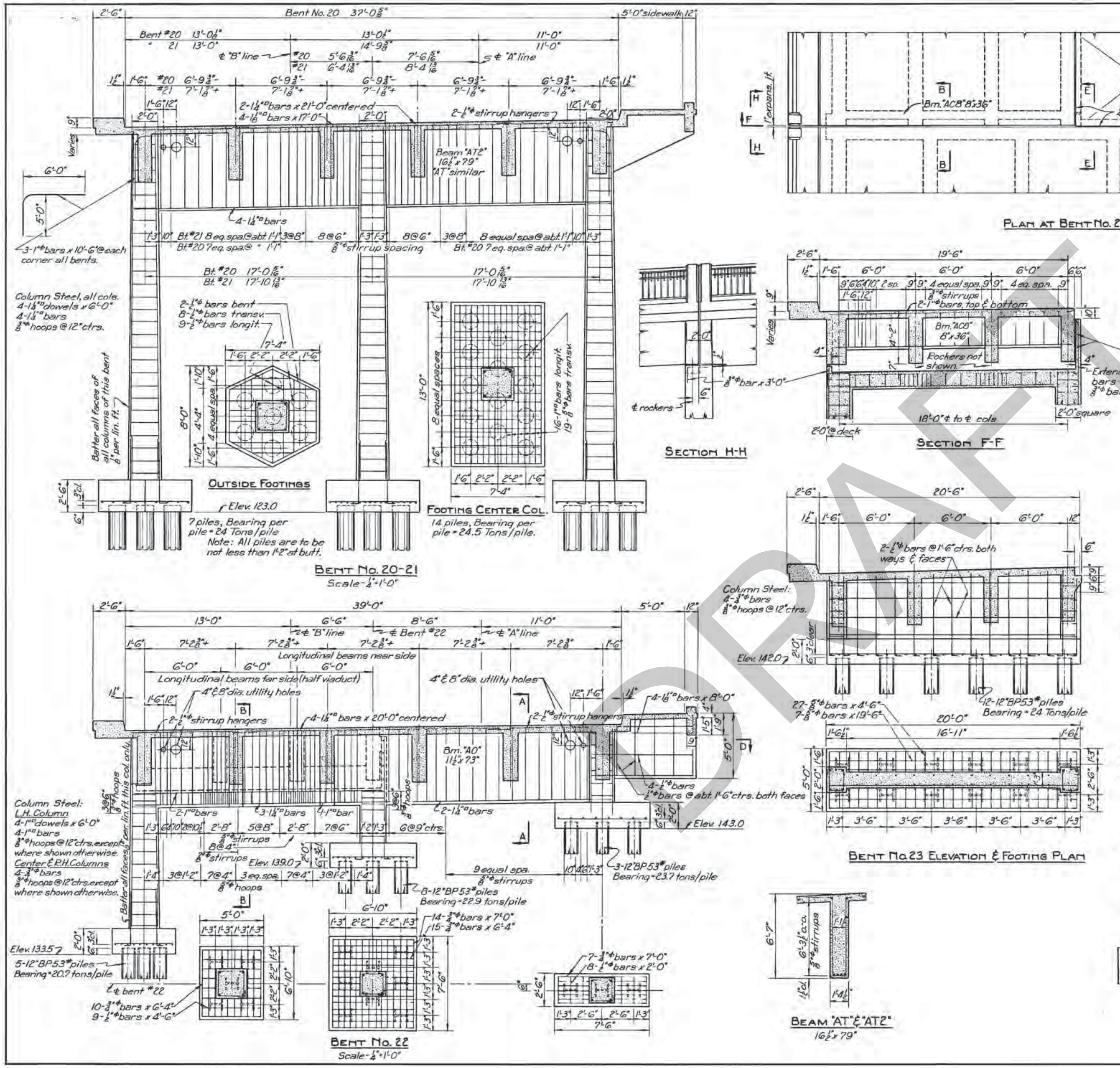
SECTION C-C
Scale ½"=1'-0"

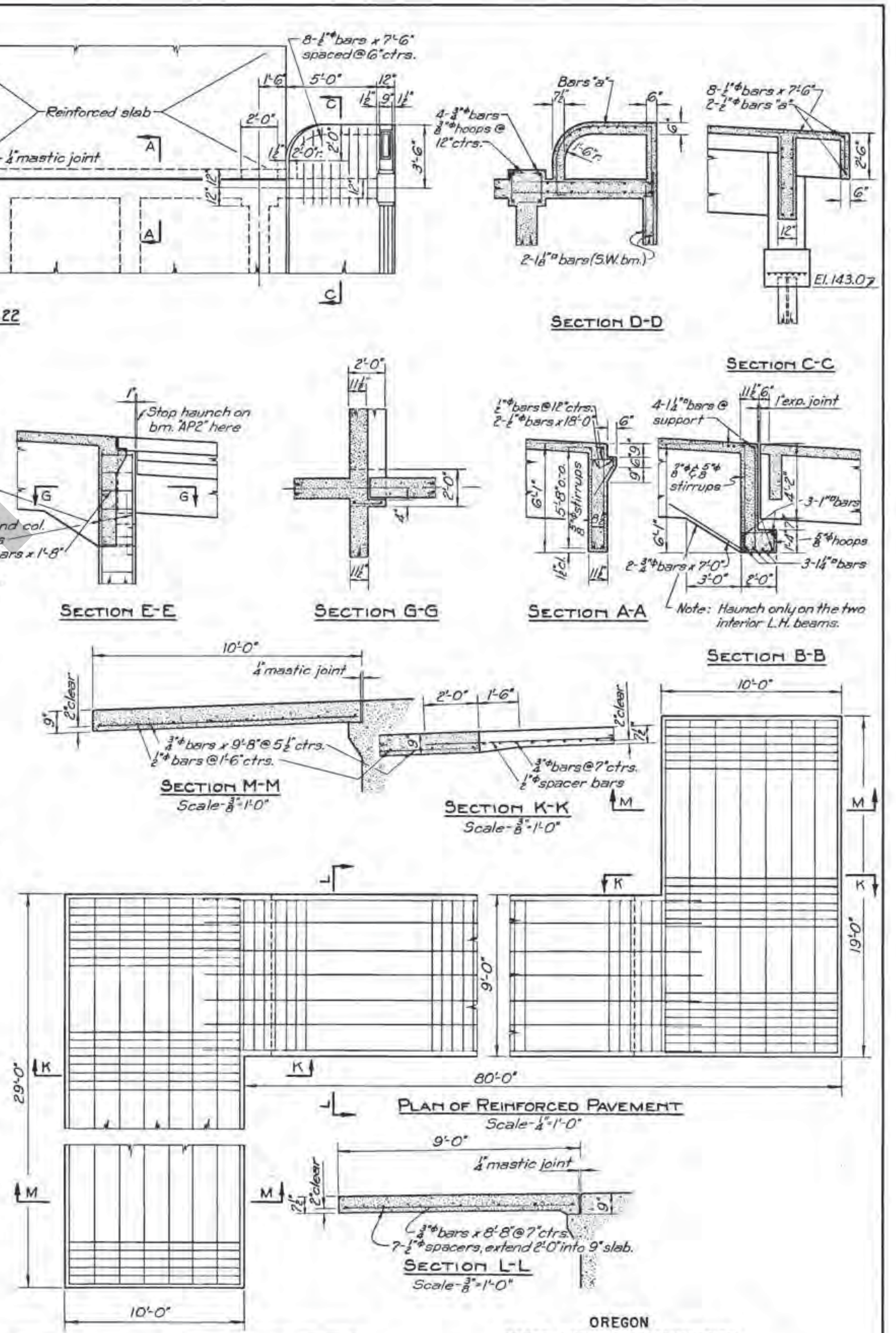
FOR INFORMATION ONLY

Approved:
GS Paxson
Bridge Engineer

OREGON
STATE HIGHWAY COMMISSION
CENTER STREET BRIDGE
WEST APPROACH
BENTS NO. 16, 17, 18 & 19

SCALE AS NOTED	DRAWN BY T.C.R.	SHEET 17 OF 25
OCTOBER 18, 1951	TRACED BY C.O.F.	BRIDGE NO. 123A
CHECKED BY		DRAWING NO. 10102





FOR INFORMATION ONLY

Approved:

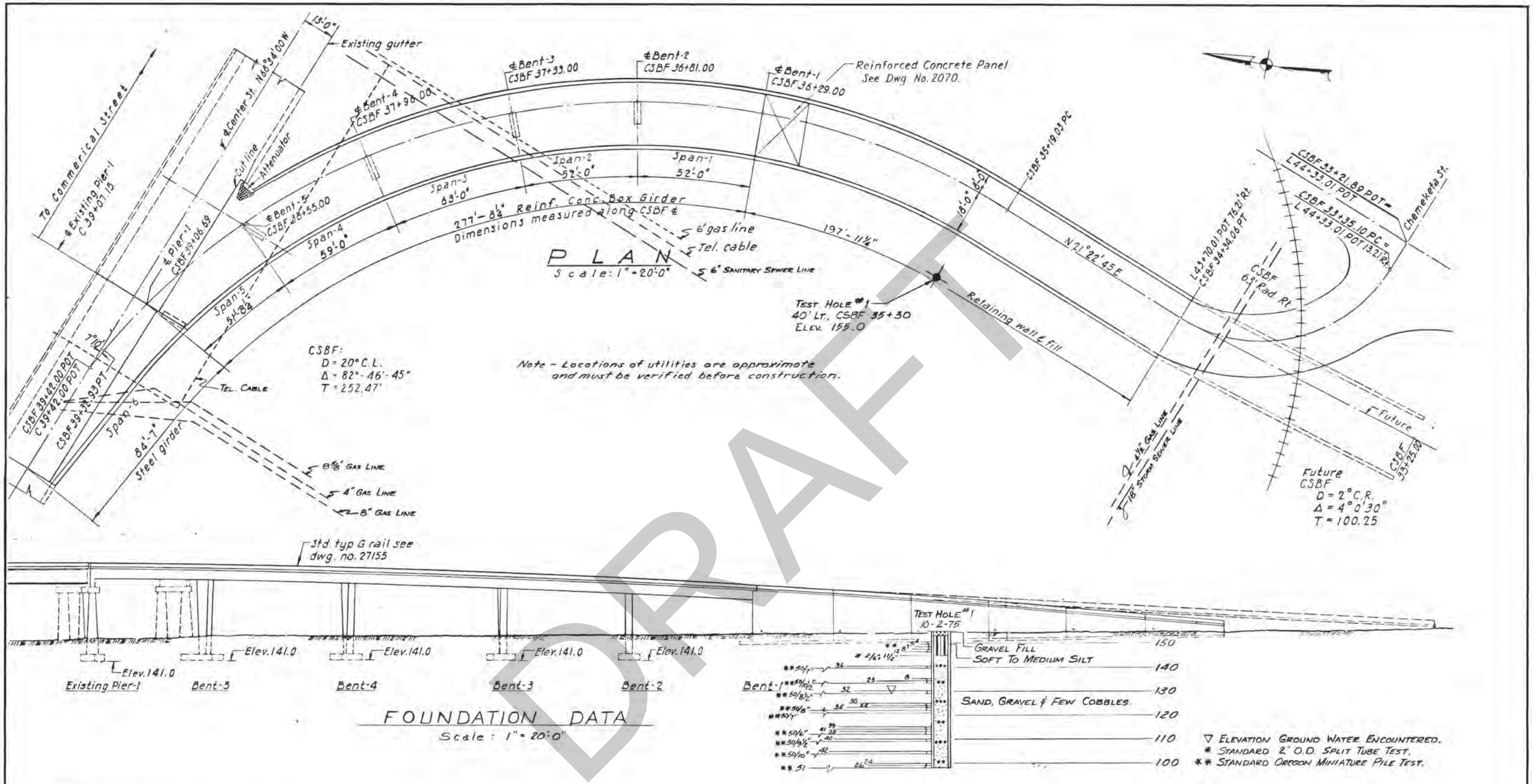
Bridge Engineer

OREGON
STATE HIGHWAY COMMISSION
CENTER STREET BRIDGE
WEST APPROACH
BENTS NO. 20, 21, 22 & 23

SCALE AS NOTED
APRIL 28, 1952
CHECKED BY

DRAWN BY T.C.R.
TRACED BY C.O.F.

SHEET 18 OF 25
BRIDGE NO. 123A
DRAWING NO. 10103



Note - Locations of utilities are approximate and must be verified before construction.

Foundation data shown on this drawing, in some instances, is a consolidation of and a revision in terminology from the original field drilling logs. The original field drilling logs are available for review in the office of the Bridge Engineer in Salem.

LEGEND OF MATERIALS

① Gravels, cobbles and boulders.
② Sand.
③ Silts.
④ Clays.
⑤ Silty gravels, gravel-silt mixture.
⑥ Silty sand, sand-silt mixture.
⑦ Clayey gravel, gravel-clay mixture.
⑧ Clayey sand, sand-clay mixture.
⑨ Silty clay.
⑩ Sand-silt-clay mixture, near equal proportions.
⑪ Sandstone.
⑫ Shale.
⑬ Basalt rock, broken, solid, weathered, lava.
⑭

APPROVED: Walter J Hart
BRIDGE ENGINEER
DESIGNED
DRAWN Phil Amaya
CHECKED J.D.
REVIEWED
REVIEWED
CALC. BOOK

DATE	REVISION

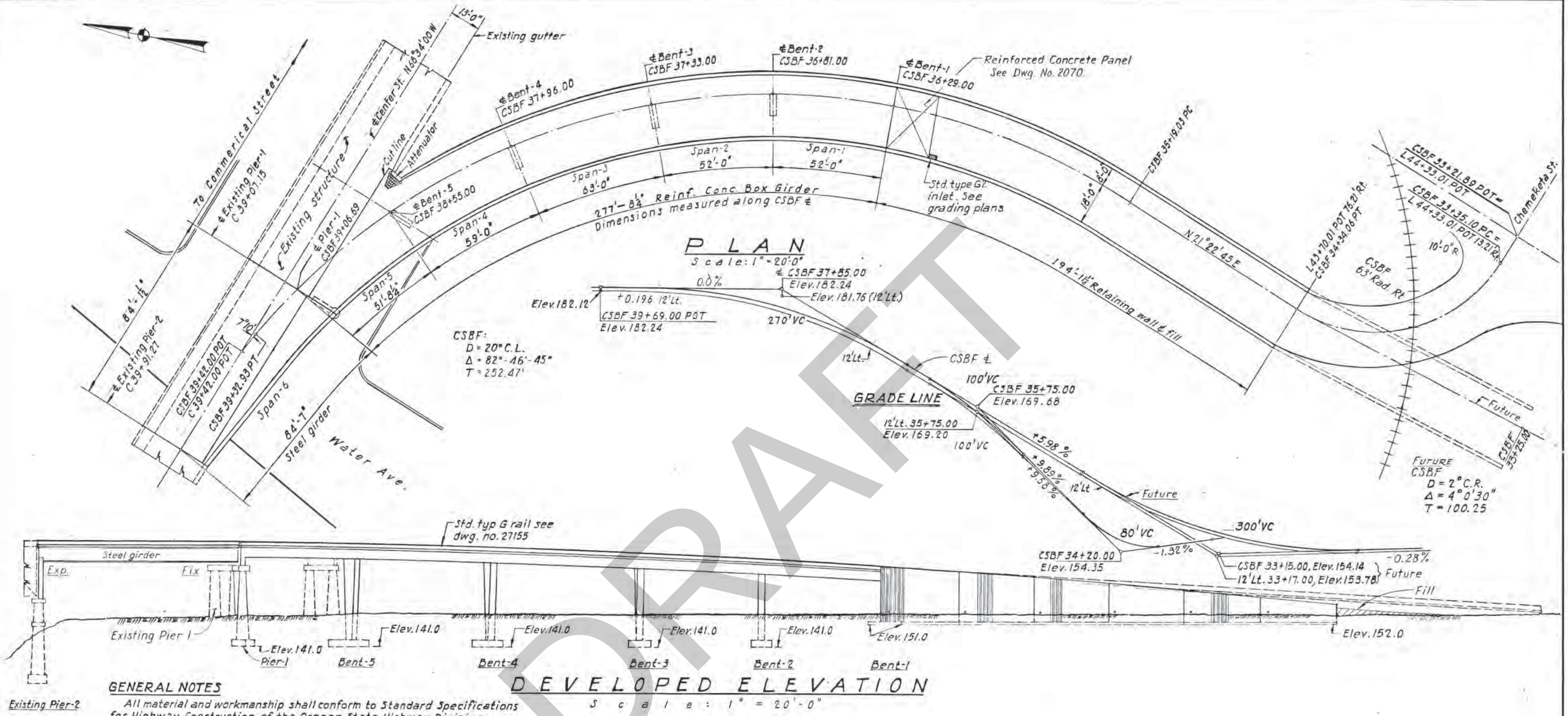
OREGON STATE HIGHWAY DIVISION
BRIDGE SECTION

CENTER TO FRONT SB OFF RAMP

FOUNDATION DATA

DATE Oct. 1975
BRIDGE NO. 123 G
SHEET 2 OF 24
DRAWING NO. 29878

Reviewed J.M.T. 11/17/75
L.L.W. 11/18/75



PLAN

Scale: 1" = 20'-0"

DEVELOPED ELEVATION

Scale: 1" = 20'-0"

GENERAL NOTES

All material and workmanship shall conform to Standard Specifications for Highway Construction of the Oregon State Highway Division.

Bridge designed for HS20-44 loading with an allowance of 15 psf for future wearing surface.

Concrete box girder spans designed by Load Factor Design Method. Concrete strengths shall be as shown in adjoing table.

All structural steel shall conform to ASTM Specifications in accordance with detail plans. All bolts at structural connections shall be ⅞" dia. high strength bolts conforming to ASTM Specifications A 325.

All reinforcing steel shall conform to ASTM Specification A 615. Bars no. 3 thru no. 5 shall be grade 40 (fs = 20,000 psi). Bars no. 6 thru 11 shall be grade 60 (fs = 24,000 psi). The following splice lengths shall be used unless shown otherwise. Bars no. 3 thru no. 5 shall be lapped 24 diameter or a minimum of 1'-0", bars no. 6 and 7 shall be lapped 38 dia. bars no. 8 and 9 shall be lapped 48 dia. bars no. 10 and 11 shall be lapped 60 dia. at all splices unless noted or shown otherwise. All bars shall be placed 2" clear of nearest face of concrete unless shown or noted otherwise. The top bends of stirrups extending from beam stems into the top slab may be shop or field bent, unless shown or noted otherwise.

Reinforcing steel for columns and walls shall not be fabricated until final footing elevations have been determined in the field.

Required footing bearings:
Bent 1: 1.2 ton/ft²
Bents 2 thru 5: 6.0 ton/ft².
Pier 1: 3.4 ton/ft²

CONCRETE STRENGTHS	
Box Stems and Bottom Slab	3300-1"
Deck	4000-1½"
All Other Concrete	3300-1½" (fc = 1320 psi)

AS CONSTRUCTED
8-30-77

* Accompanied by dwg. no's.
29878, 30743 thru 30757, 2070, 22970, 23836, 2127, 27155, 2126 & 2126A
Drawing Numbers 10086, 10090, 10091, 10094, 10095 & 10096 of existing structure to accompany this set of plans, for information only.

APPROVED:
Walter J. Hunt
BRIDGE ENGINEER
E.S. Hunter
ASS'T. STATE HIGHWAY ENGINEER

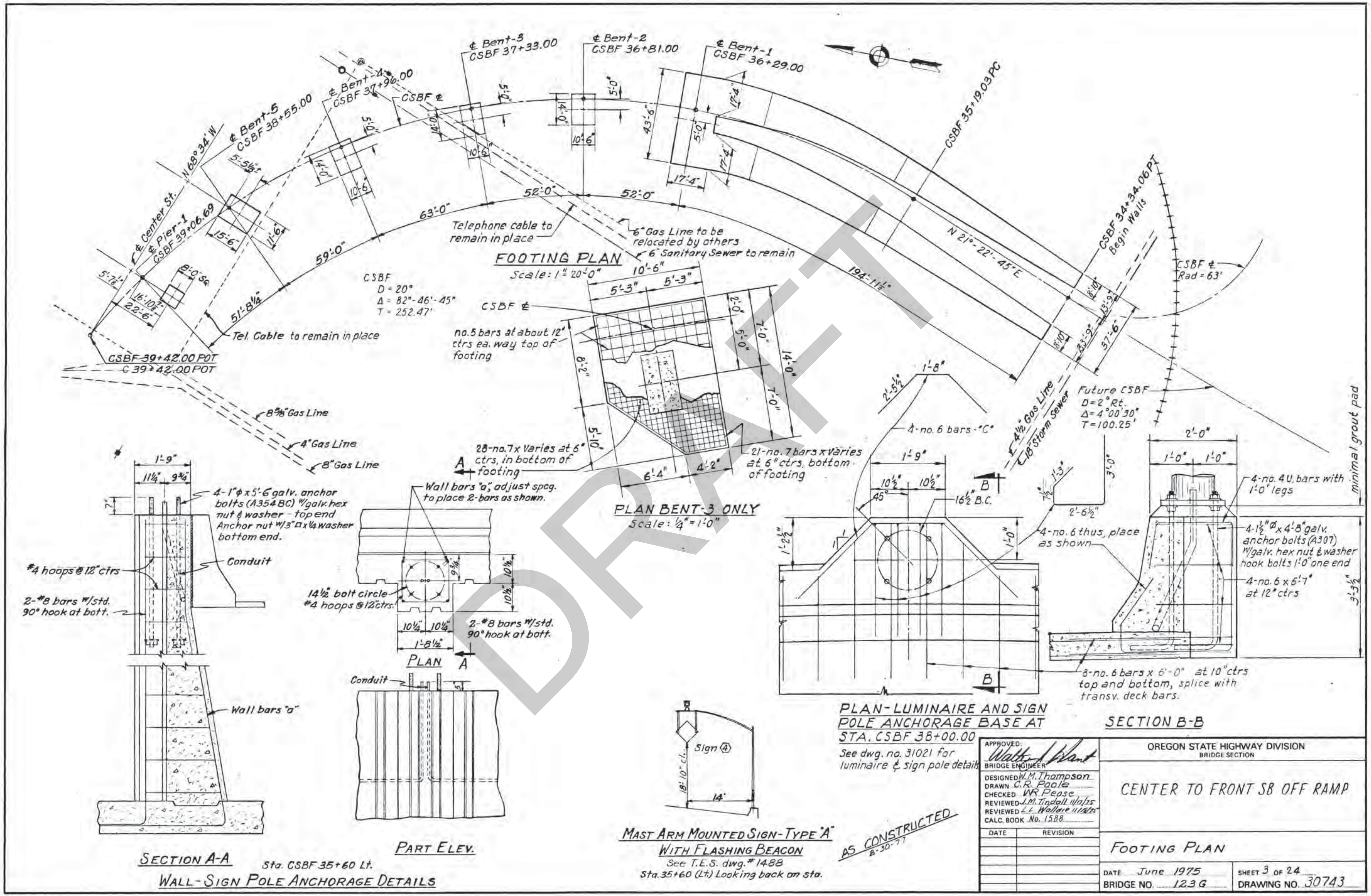
DESIGNED WMT
DRAWN JH
CHECKED WRP
CAL. BOOK 1588

OREGON STATE HIGHWAY DIVISION
BRIDGE SECTION

CENTER TO FRONT SO. BOUND OFF RAMP
CENTER ST. BRIDGE (FRONT ST. OFF-RAMP) SEC.
WILLAMINA-SALEM HWY. MARION. Co.

PLAN AND ELEVATION

DATE June 1975 BRIDGE NO. 123 G
ACCOMPANIED BY DWGS. * See above
SHEET 1 OF 24
DRAWING NO. 30742



FOOTING PLAN
Scale: 1" = 20'-0"

PLAN BENT-3 ONLY
Scale: ¼" = 1'-0"

PLAN-LUMINAIRE AND SIGN POLE ANCHORAGE BASE AT STA. CSBF 38+00.00

See dwg. no. 31021 for luminaire & sign pole details

SECTION B-B

SECTION A-A

PLAN

PART ELEV.

Sta. CSBF 35+60 Lt.

WALL-SIGN POLE ANCHORAGE DETAILS

MAST ARM MOUNTED SIGN-TYPE "A" WITH FLASHING BEACON

See T.E.S. dwg. # 1488

Sta. 35+60 (Lt) Looking back on sta.

AS CONSTRUCTED
8-30-77

APPROVED: Walter J. Plant
BRIDGE ENGINEER
DESIGNED W.M. Thompson
DRAWN C.R. Poole
CHECKED WR Pease
REVIEWED J.M. Tindall 11/17/75
REVIEWED L.L. Wallace 11/18/75
CALC. BOOK No. 1588

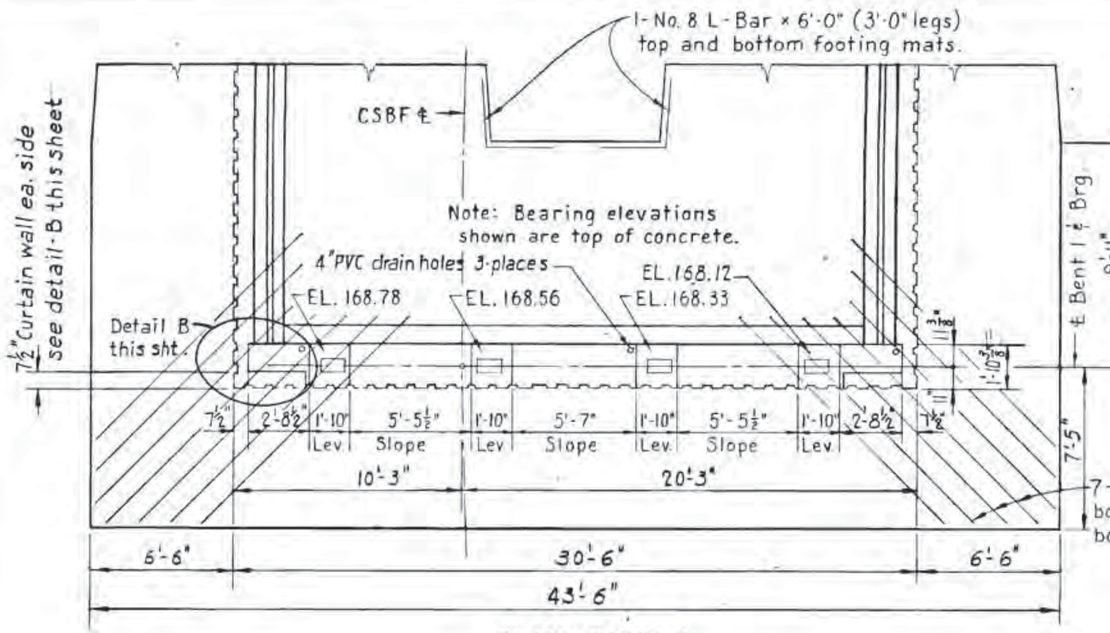
DATE	REVISION

OREGON STATE HIGHWAY DIVISION
BRIDGE SECTION

CENTER TO FRONT SB OFF RAMP

FOOTING PLAN

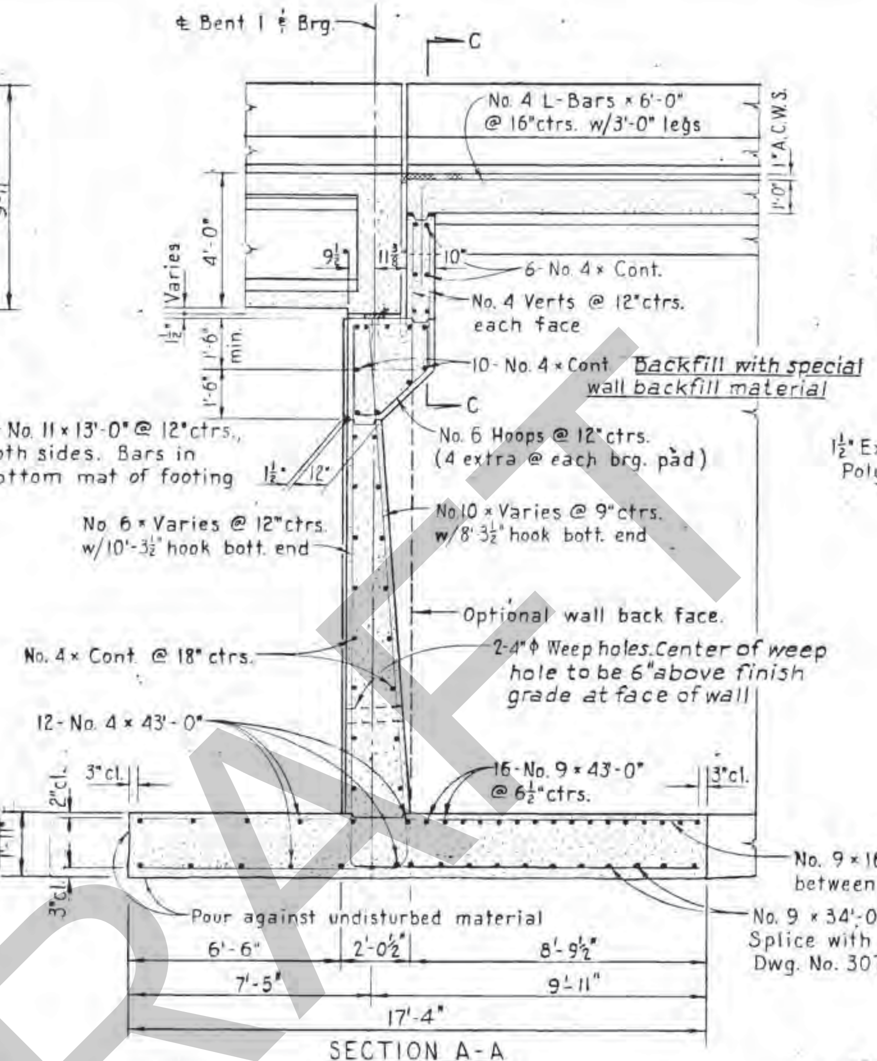
DATE June 1975
BRIDGE NO. 123 G
SHEET 3 OF 24
DRAWING NO. 30743



PLAN - BENT 1

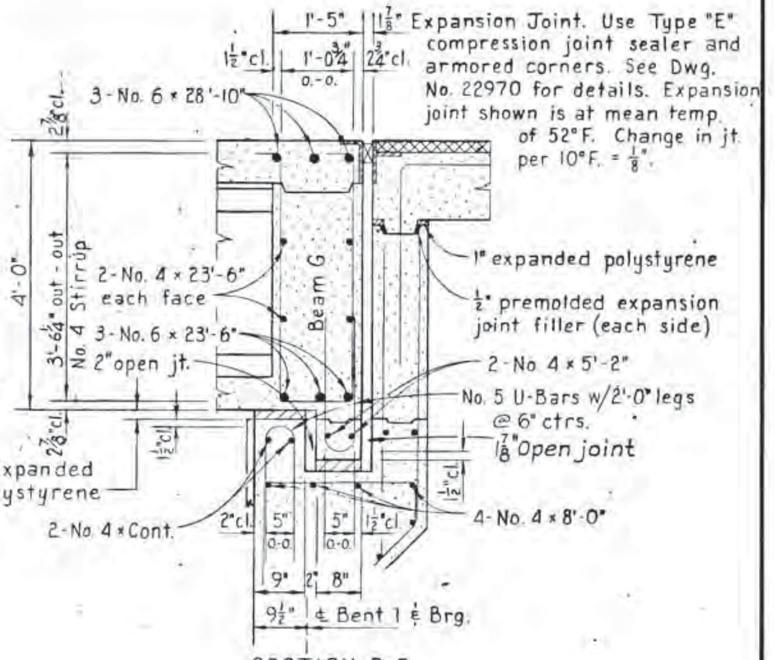
Scale: ¼" = 1'-0"

Box Girder and Paving Slab Omitted



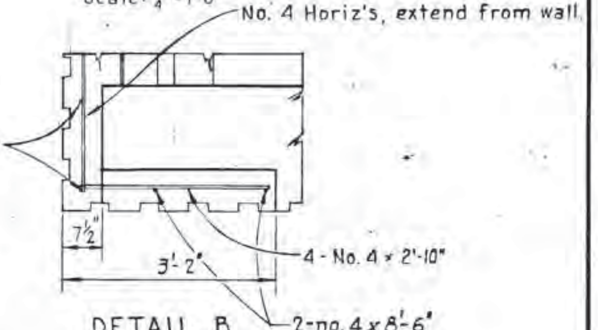
SECTION A-A

Scale: ⅜" = 1'-0"



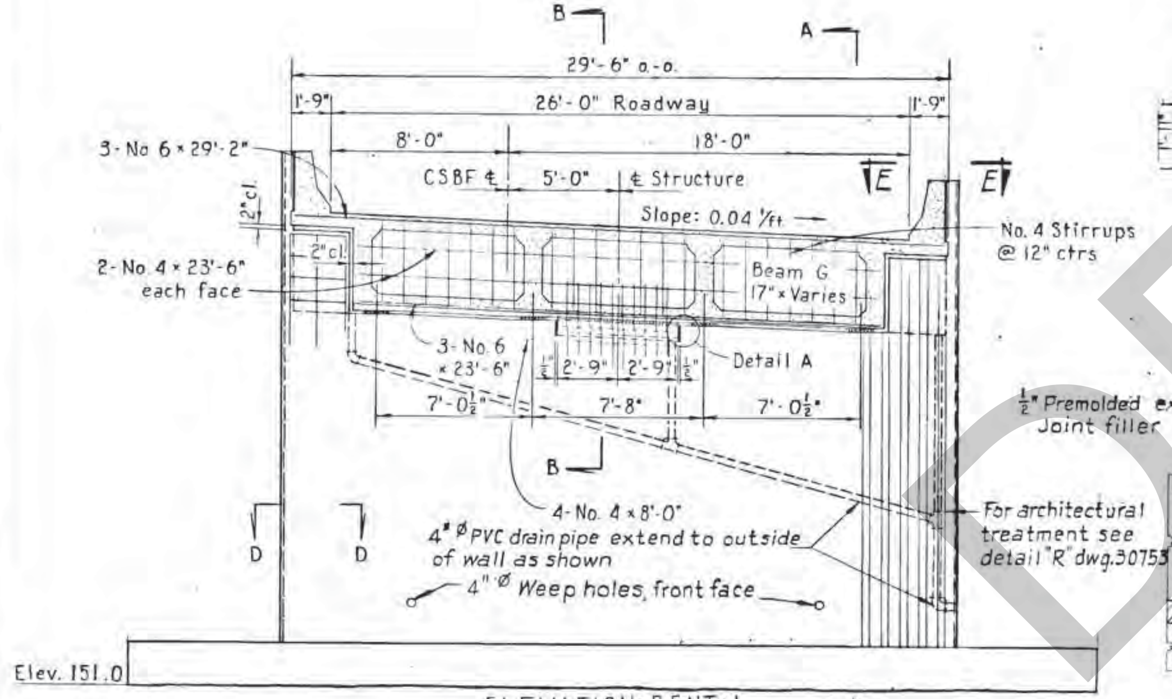
SECTION B-B

Scale: ¾" = 1'-0"



DETAIL B

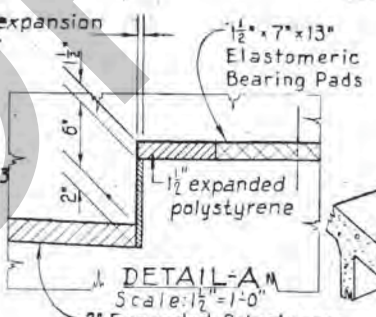
Scale: ¾" = 1'-0"



ELEVATION-BENT-1

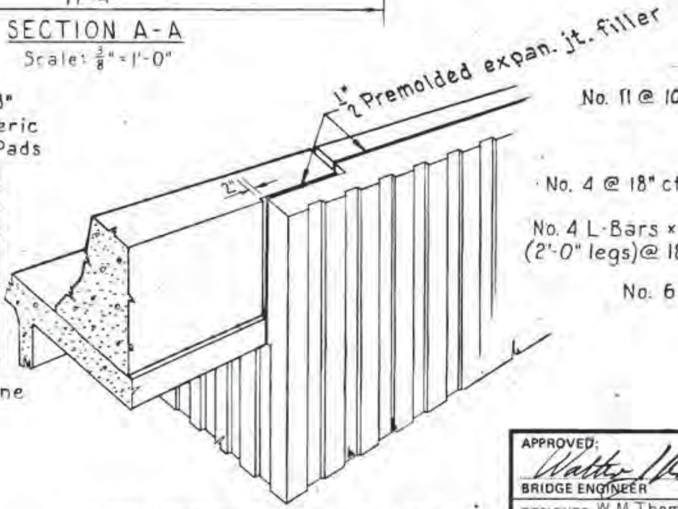
Scale : ¼" = 1'-0"

LOOKING BACK ON STATION

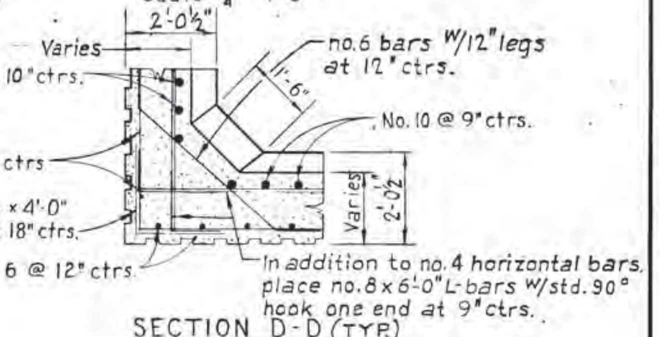


DETAIL-A

Scale: 1½" = 1'-0"



PERSPECTIVE-AT VIEW-EE



SECTION D-D (TYP.)

Scale: ½" = 1'-0"

AS CONSTRUCTED
8-30-77

APPROVED: Walter J. Ward
BRIDGE ENGINEER

DESIGNED W M Thompson
DRAWN W M Thompson
CHECKED WR Pease
REVIEWED J.M. Tindall 11/17/75
REVIEWED L.L. Wallace 11/18/75
CALC. BOOK No. 1588

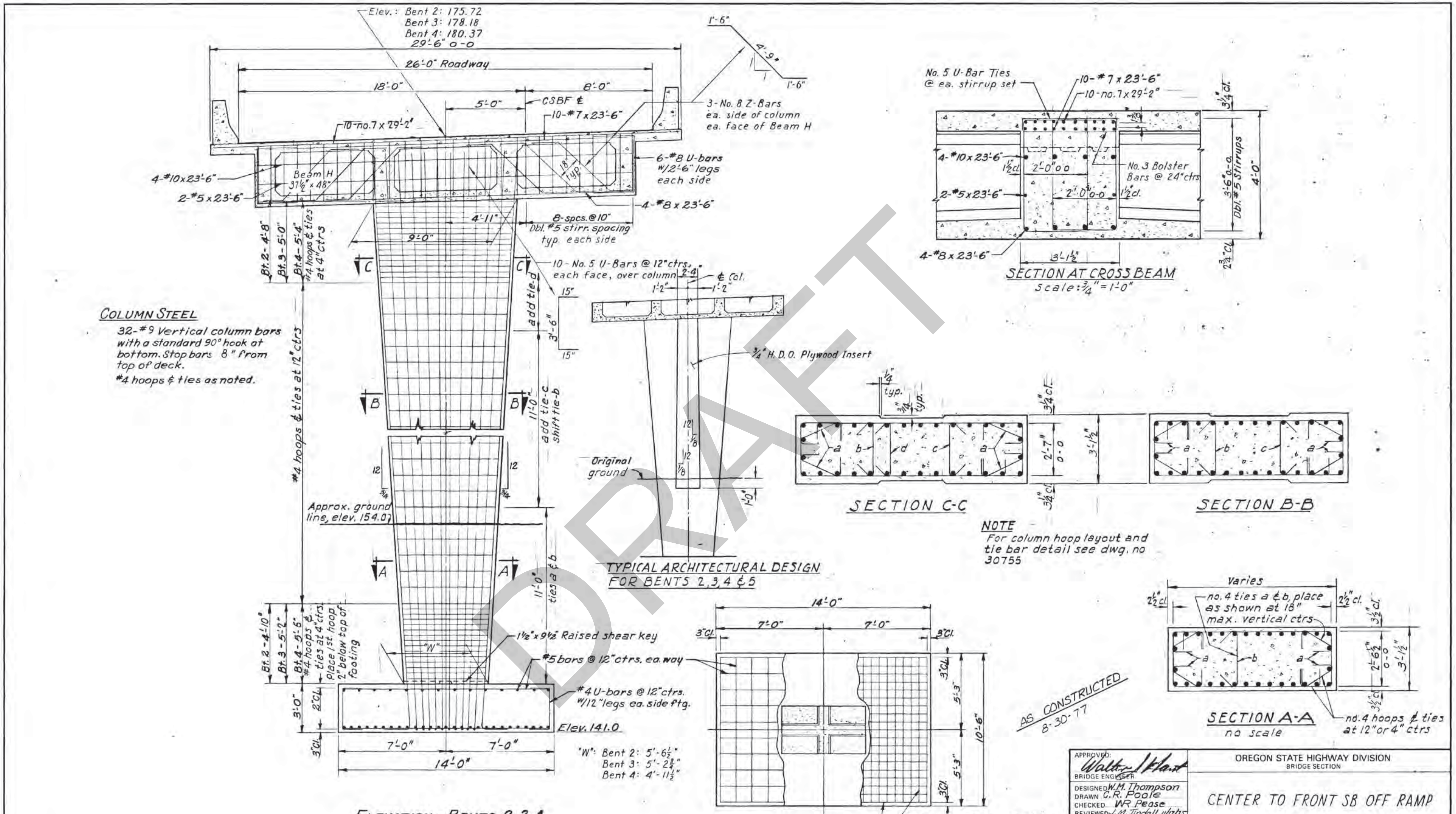
DATE	REVISION

OREGON STATE HIGHWAY DIVISION
BRIDGE SECTION

CENTER TO FRONT SB OFF RAMP

BENT-1 DETAILS

DATE June 1975
BRIDGE NO. 123G
SHEET 12 OF 24
DRAWING NO. 30752



COLUMN STEEL

32-#9 Vertical column bars with a standard 90° hook at bottom. Stop bars 8" from top of deck.

#4 hoops & ties as noted.

SECTION AT CROSS BEAM
Scale: ¾" = 1'-0"

TYPICAL ARCHITECTURAL DESIGN FOR BENTS 2, 3, 4 & 5

SECTION C-C

SECTION B-B

NOTE
For column hoop layout and tie bar detail see dwg. no 30755

SECTION A-A
no scale

ELEVATION - BENTS 2, 3, 4
Scale: ⅜" = 1'-0"

BENTS 2 & 4 FOOTING PLAN
Scale: ⅜" = 1'-0"

See dwg. no. 30743 for footing plan of bent-3

APPROVED: Walter J. Hart
BRIDGE ENGINEER
DESIGNED W.M. Thompson
DRAWN C.R. Poole
CHECKED W.R. Pease
REVIEWED J.M. Tindall 11/1/75
REVIEWED L.L. Wallace 11/1/75
CALC. BOOK No. 1588

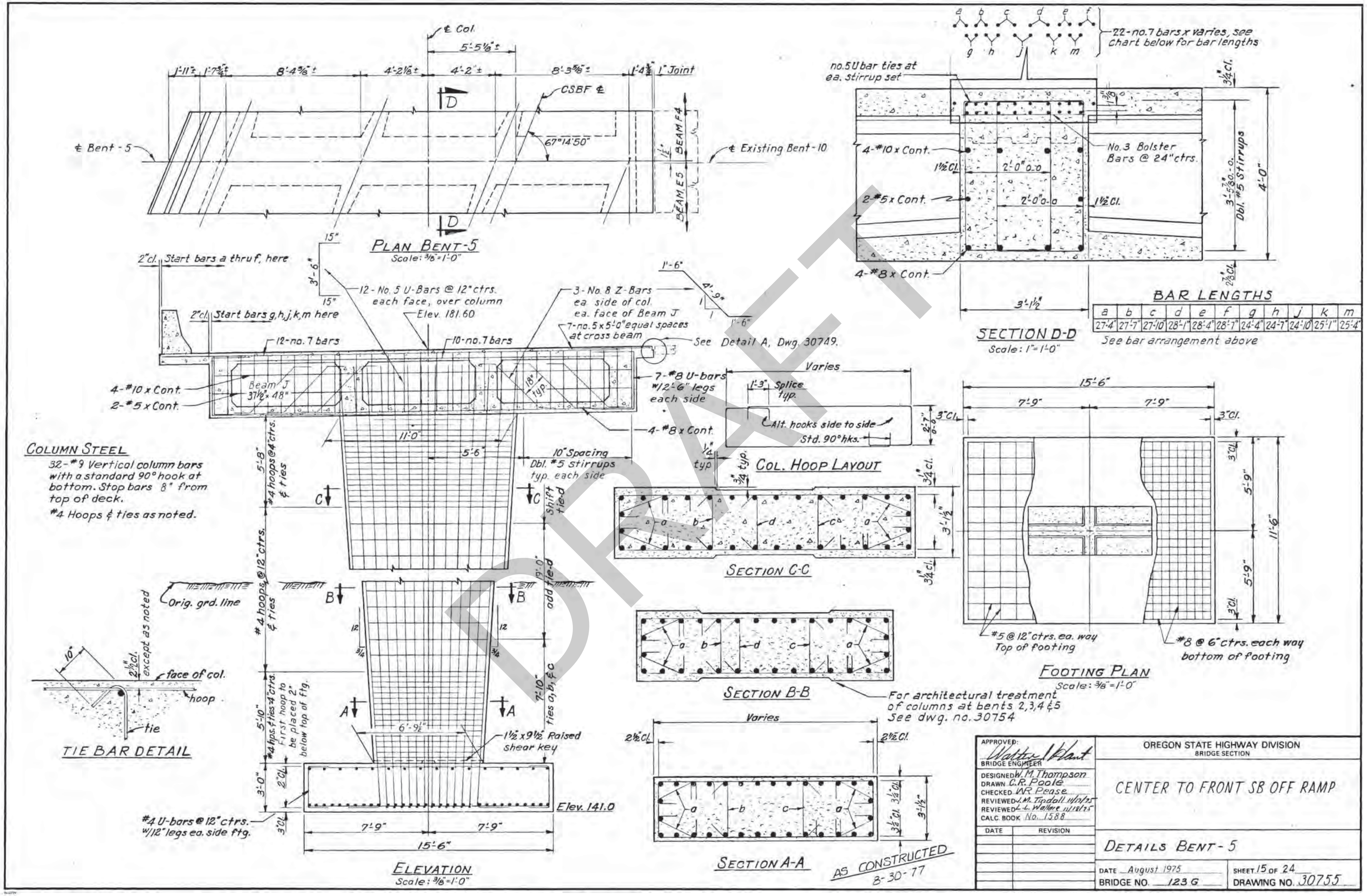
DATE	REVISION

OREGON STATE HIGHWAY DIVISION
BRIDGE SECTION

CENTER TO FRONT SB OFF RAMP

DETAILS BENT 2, 3, & 4

DATE June 1975
SHEET 14 OF 24
BRIDGE NO. 123G
DRAWING NO. 30754



BAR LENGTHS

a	b	c	d	e	f	g	h	j	k	m
27'-4"	27'-7"	27'-10"	28'-1"	28'-4"	28'-7"	24'-4"	24'-7"	24'-10"	25'-1"	25'-4"

See bar arrangement above

AS CONSTRUCTED
8-30-77

APPROVED: Walter J Plant
BRIDGE ENGINEER
DESIGNED W. M. Thompson
DRAWN C.R. Poole
CHECKED W.R. Pease
REVIEWED J.M. Tindall 11/17/75
REVIEWED L.L. Wallace 11/18/75
CALC. BOOK No. 1588

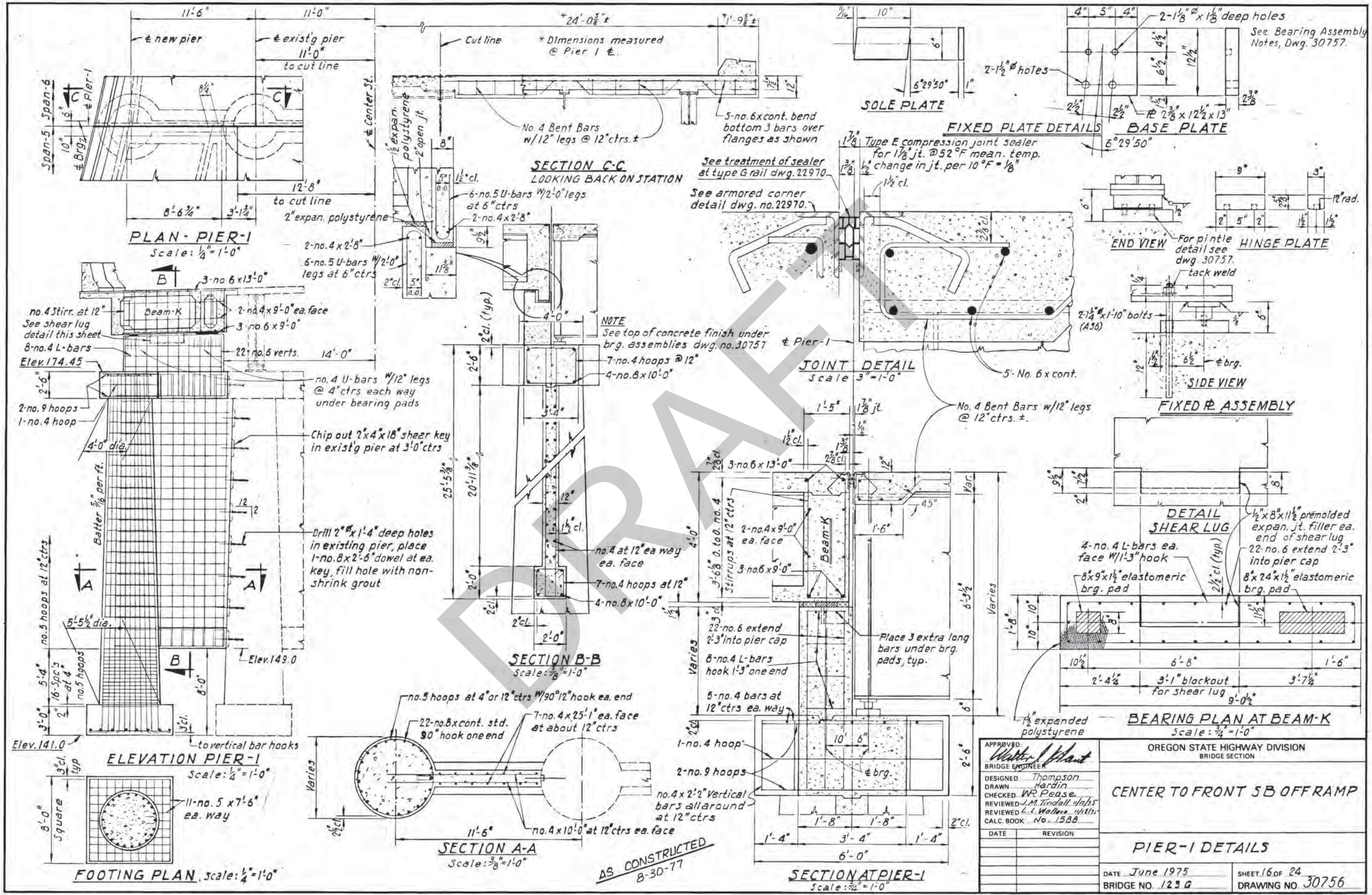
DATE	REVISION

OREGON STATE HIGHWAY DIVISION
BRIDGE SECTION

CENTER TO FRONT SB OFF RAMP

DETAILS BENT-5

DATE August 1975
BRIDGE NO. 123 G
SHEET 15 OF 24
DRAWING NO. 30755



APPROVED: Walter J. Hart
BRIDGE ENGINEER

DESIGNED Thompson
DRAWN Hardin
CHECKED W.R. Pease
REVIEWED J.M. Tindall 11/17/75
REVIEWED L.L. Wallace 11/18/75
CALC. BOOK No. 1588

DATE	REVISION

OREGON STATE HIGHWAY DIVISION
BRIDGE SECTION

CENTER TO FRONT SB OFF RAMP

PIER-1 DETAILS

DATE June 1975
BRIDGE NO. 123 G
SHEET 16 OF 24
DRAWING NO. 30756

BRIDGE	DRAWING NUMBER
EAST APPROACH	37002 - 37040
RIVER SPANS	37041 - 37070
RIVER SPANS (STEEL ALTERNATE)	37071 - 37082
WEST APPROACH	37083 - 37129
DETOUR STRUCTURE	37138 - 37142
ACCOMPANIED BY DRAWINGS	2070, 2105, 2117, 2127, 27155, 30937, 30938, 30939, 30940, 31117, 31275 31600, 31724, 31825, 34880 34903, 34949, 34959, 35010 37143, 37144
DRAWINGS FOR INFORMATION ONLY	459-463, 10086-10108 9233, 30031-30039, 30742-30757 35968, 35969, 35972

VICINITY MAP

GENERAL NOTES

All material and workmanship shall conform to the Standard Specifications for Highway Construction of the Oregon State Highway Division.

The river spans and bridge widening are designed for HS25 loading with an allowance of 25 psf for present and 25 psf for future wearing surface.

Concrete members, except prestressed members, designed by Load Factor Design Method.

Prestressing steel shall be in accordance with detail plans.

All other reinforcing steel shall conform to ASTM Specification A615(S1) Grade 60. The following splice lengths shall be used unless shown otherwise:

BAR SIZE	3	4	5	6	7	8	9	10	11	14 & 18
SPLICE LENGTH	1'-0"	1'-4"	1'-8"	2'-0"	2'-9"	3'-7"	4'-7"	5'-9"	7'-1"	Not Permitted

All bars shall be placed 2" clear of the nearest face of concrete, unless shown otherwise. The top bends of stirrups extending from beam stems into the top slab may be shop or field bent.

All reinforcing steel in the upper portion of the deck shall be epoxy coated. This includes all top longitudinal bars, all top transverse bars, all transverse bent bars, all bars extending from the deck into the parapet or curb, and all other bars noted on the detail plans. Bike W Spans 12 thru 19 shall have no epoxy coated reinforcing.

Reinforcing steel for columns and walls shall not be fabricated until final footing elevations have been determined in the field.

All structural steel shall conform to ASTM Specifications in accordance with detail plans. In place of ASTM A588, ASTM A572 grade 50 may be used for plates 2 inches thick and under.

All bolts at structural connections shall be 7/8" dia. high strength bolts conforming to ASTM Specification A325, unless shown otherwise.

Concrete in the post-tensioned box girder superstructure of Spans 2R-5R shall be as shown on the detail plans.

Concrete in the deck, except Spans 2R-5R conc. alternate, shall be Class 4500-3/4".

Concrete in crossbeams of bents 2 thru 10 and 11 thru 25 shall be Class 4000-3/4".

Concrete in columns of bents 2 thru 10 and 11 thru 25 shall be Class 4000-1-1/2".

Concrete in the bottom slab and stems of box girders of Spans 2 thru 10 and Span 1R, and the stems of Spans 11 thru 26 shall be Class 3300-3/4".

All other concrete shall be Class 3300-1 1/2".

Piling shall be as noted on the detail plans, in the Special Provisions and as called out below.

RIVER SPANS

All piling in Piers 2 thru 6 shall be HP14x73 steel piling driven to a minimum bearing of 110 tons per pile. An acceptable alternate is HP12x74 steel piling.

All piling at Piers 5 and 6 shall have reinforced tips.

WEST APPROACH

All piling in Bents 11 thru 25 shall be one of the following three options A, B, or C and shall be driven to the minimum bearing specified on the detail plans.

A. 12" nom. steel shells for cast in place concrete piling.
B. 12" square prestressed concrete piling, see dwg. 31825.
C. 14" octagonal prestressed concrete piling, see dwg. 31825.

Gradelines and elevations shown on these plans were determined from "As Constructed" plans of the existing bridge and are to be verified in the field.

Where the new bridge construction is dependent on existing bridge dimensions shown on these plans, the contractor shall verify their accuracy before starting construction or fabrication of materials.

For general notes relative to the Detour Structure, see dwg. no. 37138 .

APPROVED Walter J. Hart P.E.
STRUCTURAL DESIGN ENGINEER
Ed Hunter P.E.
TECHNICAL SERVICES ENGINEER
DESIGNED
DRAWN J. Hardin
CHECKED
REVIEWED J.M. Tindall 7/2/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK

DATE	REVISION

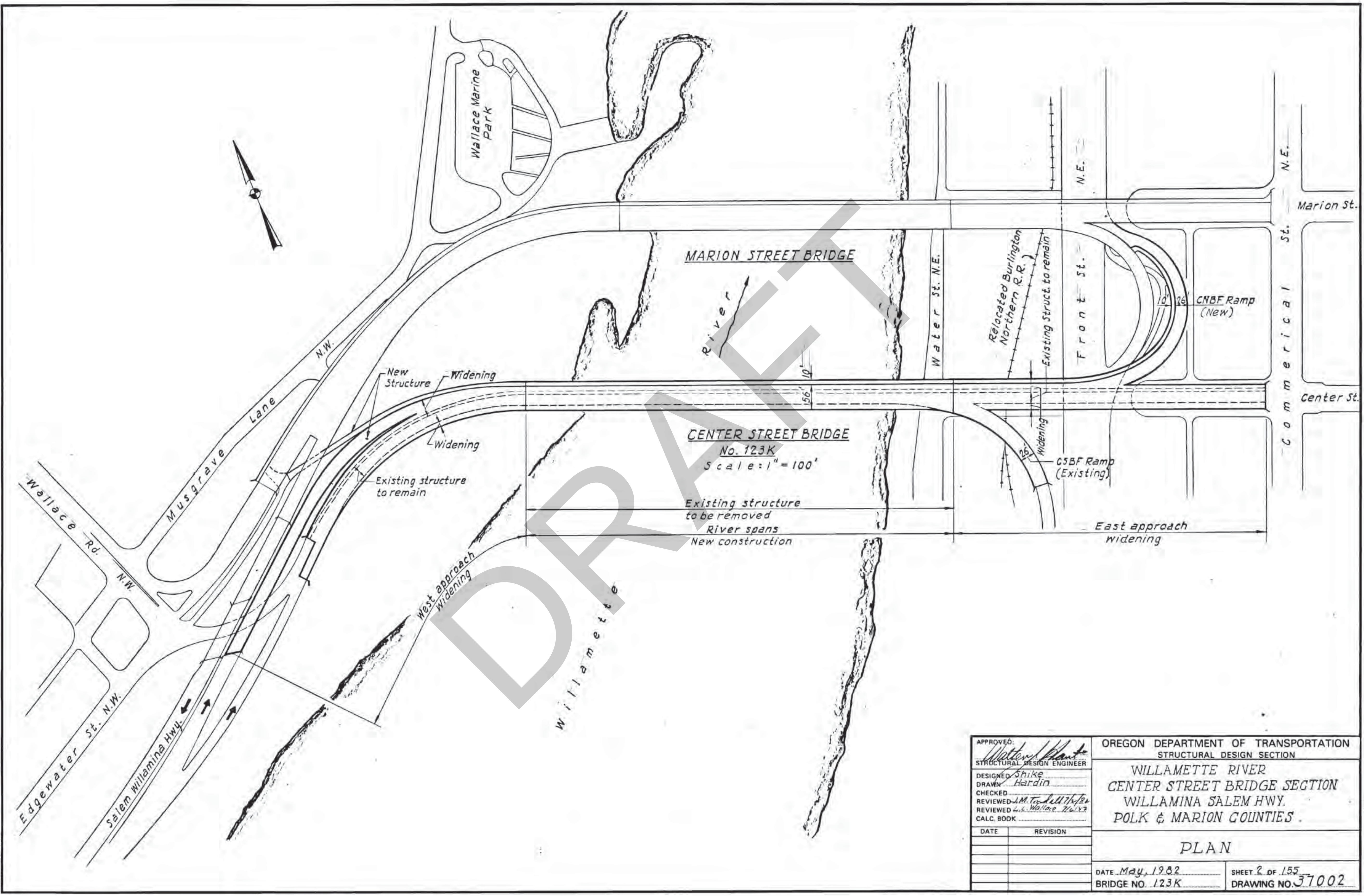
OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIV. CENTER ST. BRIDGE
WILLAMETTE RIV. (CENTER ST.) BRIDGE SEC.
WILLAMINA-SALEM HIGHWAY
POLK AND MARION COUNTIES

INDEX

ACCOMPANIED BY DWGS. See above

DATE May, 1982
BRIDGE NO. 123K
SHEET 1 OF 155
DRAWING NO. 37001



APPROVED:
STRUCTURAL DESIGN ENGINEER
DESIGNED Shike
DRAWN Hardin
CHECKED
REVIEWED J.M. Tindall 7/6/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK

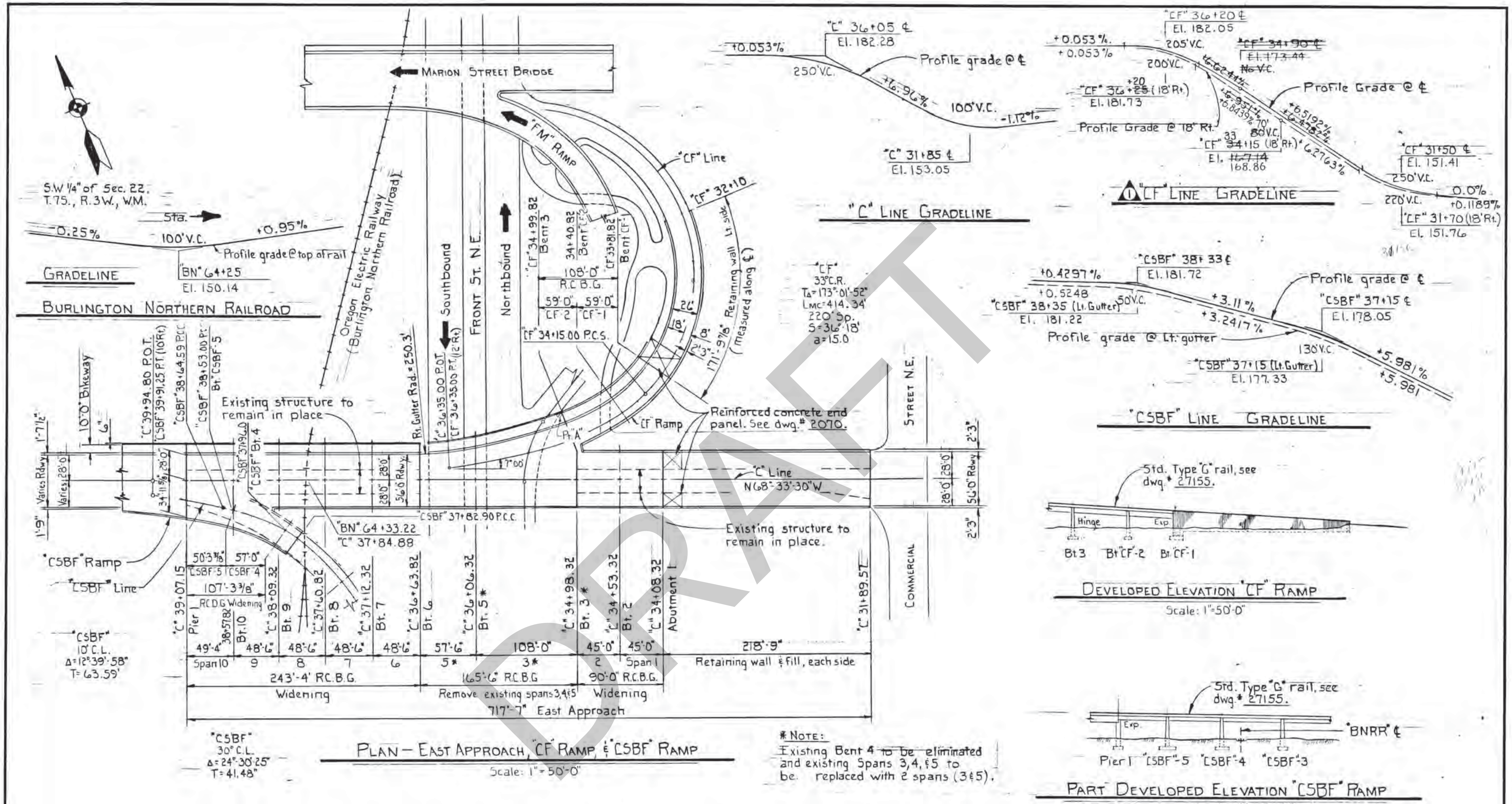
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
CENTER STREET BRIDGE SECTION
WILLAMINA SALEM HWY.
POLK & MARION COUNTIES.

PLAN

DATE May, 1982
BRIDGE NO. 123K
SHEET 2 OF 155
DRAWING NO. 37002



GRADELINE

BURLINGTON NORTHERN RAILROAD

"C" LINE GRADELINE

⚠① "CF" LINE GRADELINE

"CSBF" LINE GRADELINE

PLAN – EAST APPROACH, "CF" RAMP, & "CSBF" RAMP

Scale: 1"=50'-0"

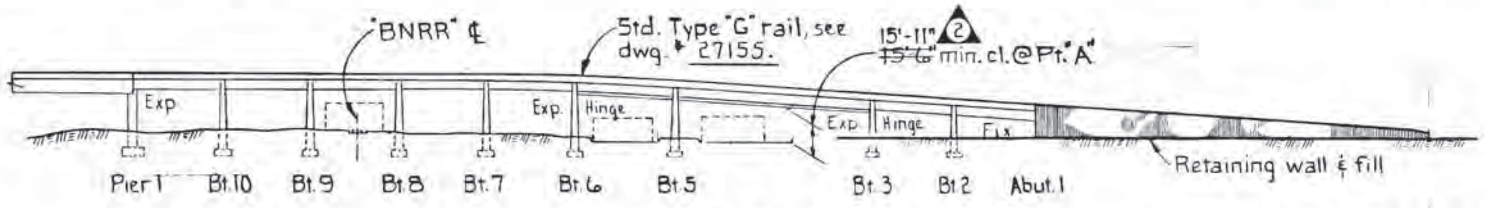
***NOTE:**
Existing Bent 4 ~~to be~~ eliminated and existing Spans 3,4, & 5 to be replaced with 2 spans (3 & 5).

DEVELOPED ELEVATION "CF" RAMP

Scale: 1"=50'-0"

PART DEVELOPED ELEVATION "CSBF" RAMP

Scale: 1"=50'-0"

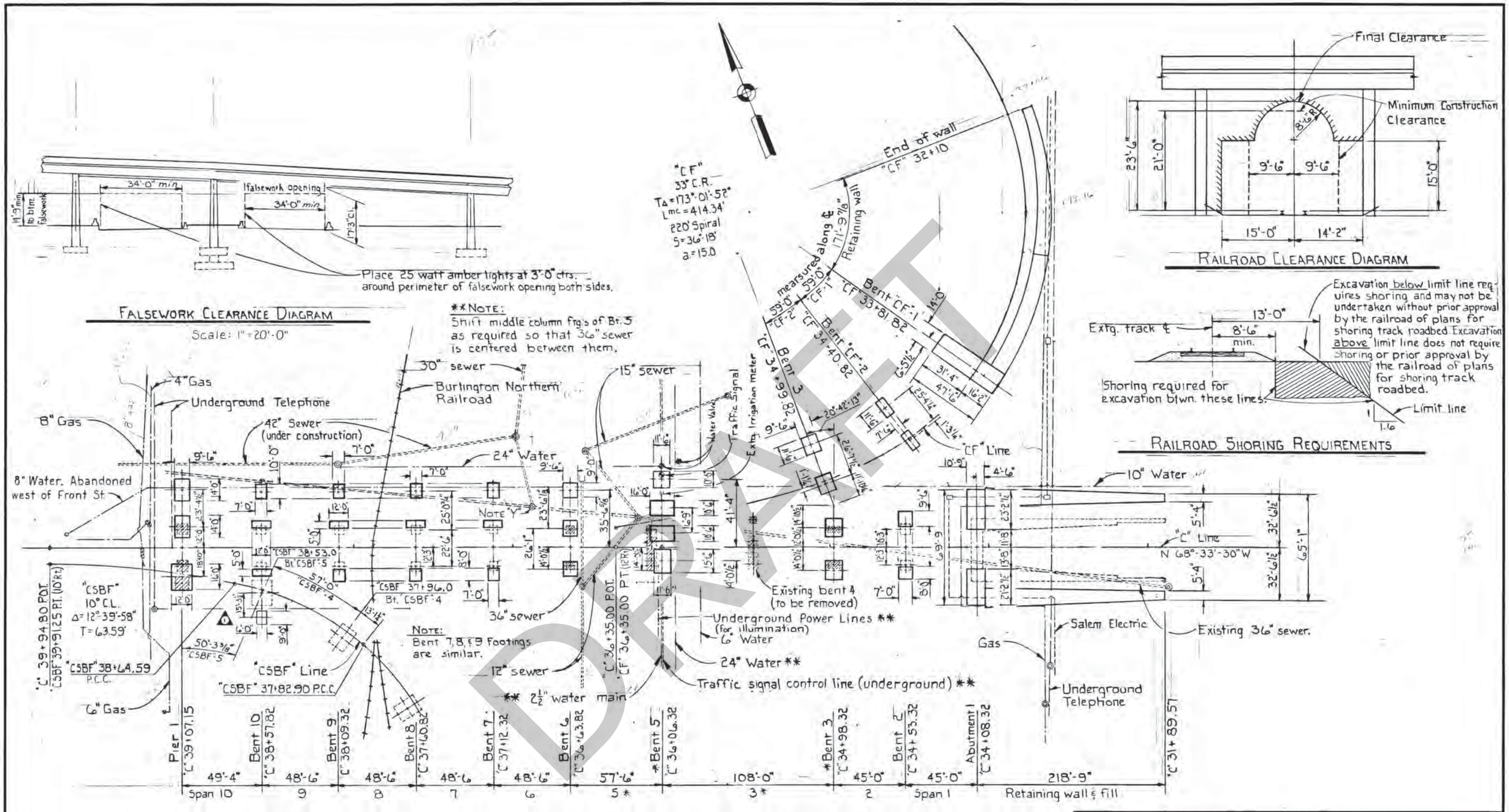


ELEVATION – EAST APPROACH

Scale: 1"=50'-0"

FOR INFORMATION ONLY

APPROVED: Walter J. Blunt STRUCTURAL DESIGN ENGINEER	OREGON DEPARTMENT OF TRANSPORTATION STRUCTURAL DESIGN SECTION
DESIGNED W.M. Thompson DRAWN J. Silbernagel CHECKED Page REVIEWED L.M. Tindall 7/6/82 REVIEWED L.L. Wallace 7/6/82 CALC. BOOK 1832	WILLAMETTE RIVER (CENTER STREET) BRIDGE
DATE | REVISION ① 9-7-82 "CF" Gradeline ② 9-7-82 Min. Vert. Clear.	PLAN & ELEVATION - EAST APPROACH
	DATE May, 1982 SHEET 3 OF 155 BRIDGE NO. 123K DRAWING NO. 37003



FALSEWORK CLEARANCE DIAGRAM

Scale: 1" = 20'-0"

RAILROAD CLEARANCE DIAGRAM

RAILROAD SHORING REQUIREMENTS

FOOTING PLAN

Scale: 1" = 30'-0"

**NOTE: Shift middle column ftg's of Bt. 5 as required so that 36" sewer is centered between them.

NOTE: Bent 7, 8, & 9 Footings are similar.

"CSBF"
30° C.L.
Δ=24°-30'-25"
T=41.48'

*NOTE: Bent 4 & Span 4 to be eliminated.

** To remain in service

NOTE 'Y': Abandon 36" Sewer line west of this point.

APPROVED: Walter J. Hart P.E.
STRUCTURAL DESIGN ENGINEER
DESIGNED W. M. Thompson
DRAWN J. Silbernagel
CHECKED Page
REVIEWED J. M. Tindall 7/6/82
REVIEWED L. L. Wallace 7/6/82
CALC. BOOK 1832

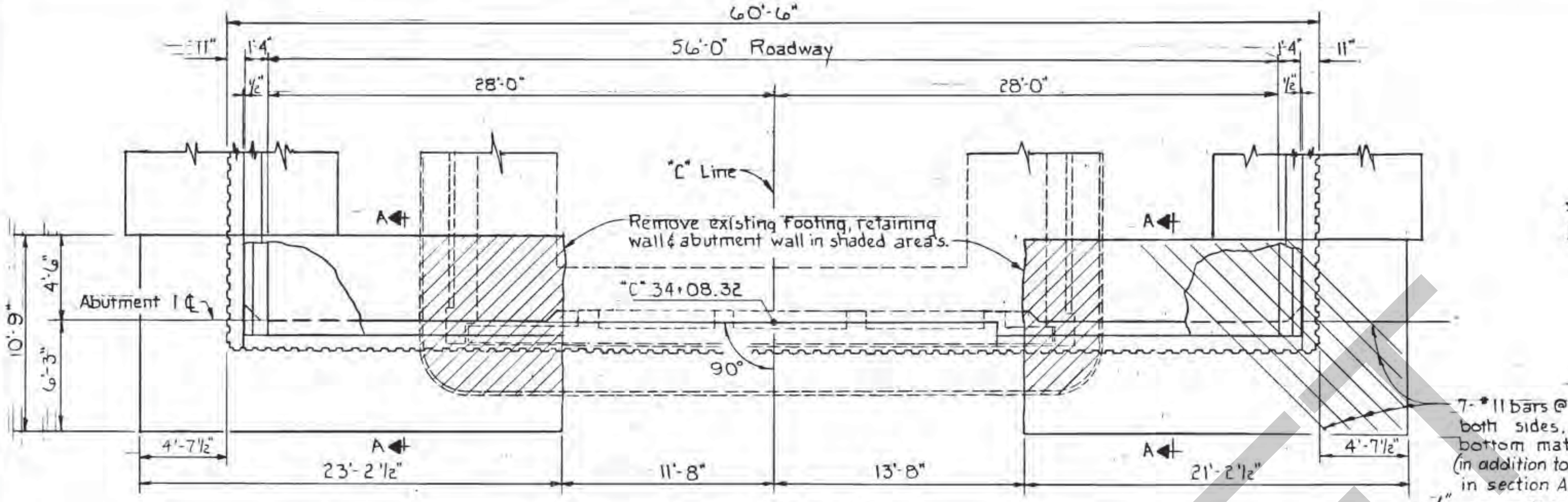
DATE	REVISION
4-26-82	Add column Bt. 10

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
-
(CENTER STREET) BRIDGE

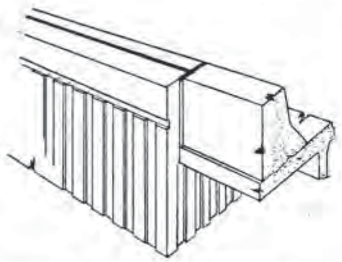
FOOTING PLAN

DATE May, 1982
BRIDGE NO. 123K
SHEET 4 OF 155
DRAWING NO. 37004

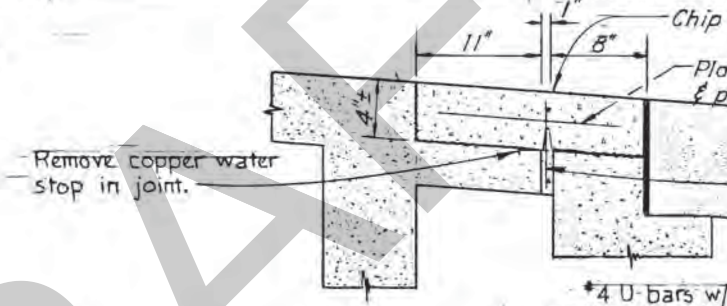


PLAN - ABUTMENT 1

Scale: ¼"=1'-0"
(Looking back on Station)
(Superstructure omitted for clarity)

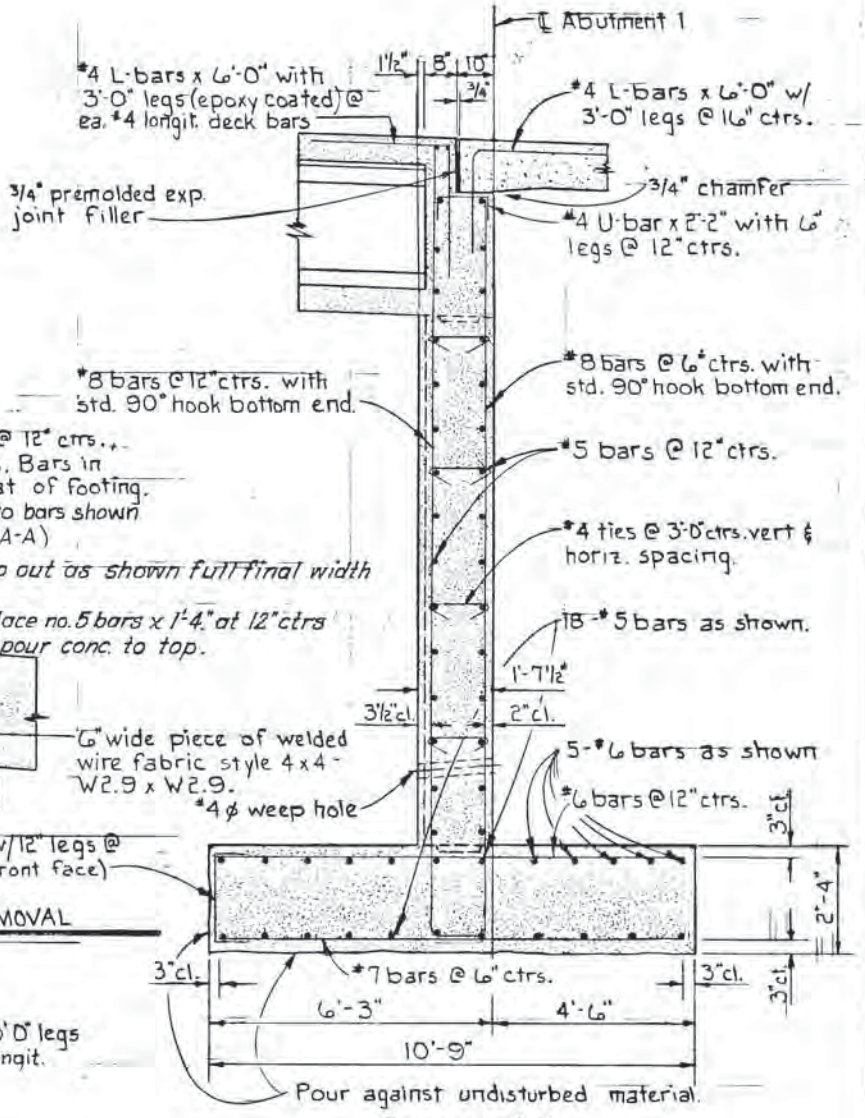


RAIL TRANSITION



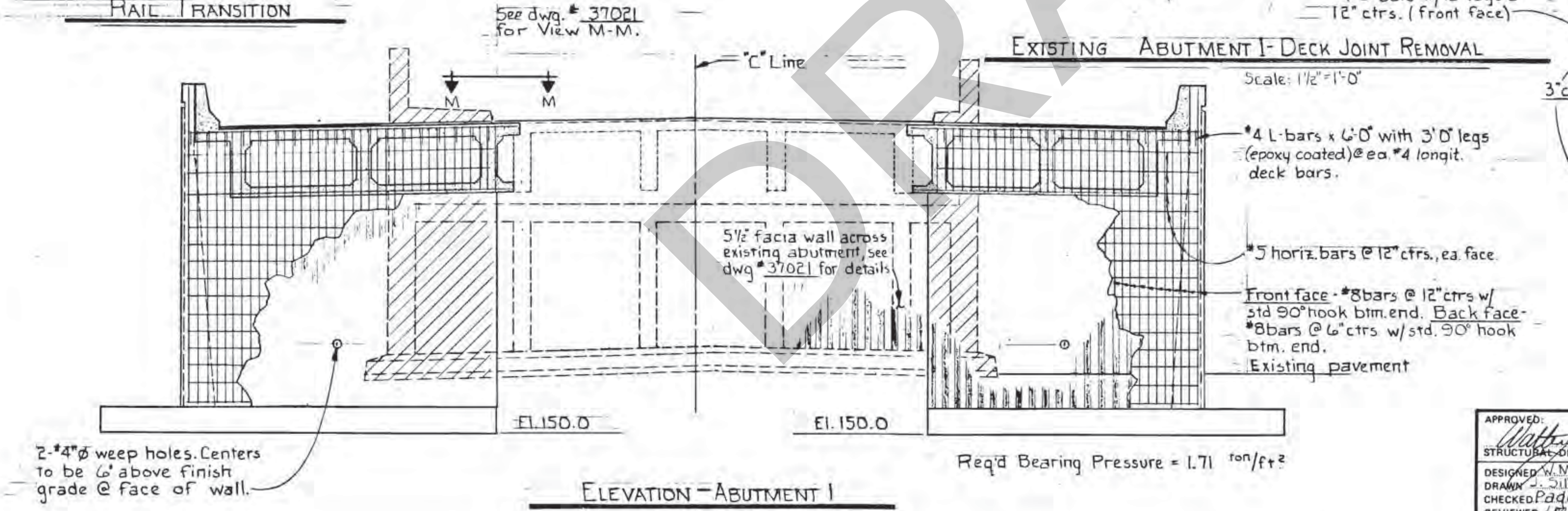
EXISTING ABUTMENT 1 - DECK JOINT REMOVAL

Scale: 1½"=1'-0"



SECTION A-A

Scale: ½"=1'-0"



ELEVATION - ABUTMENT 1

Scale: ¼"=1'-0"
(Looking back on Station)

APPROVED: Walter Wlandt
STRUCTURAL DESIGN ENGINEER
DESIGNED W. M. Thompson
DRAWN J. Silbernagel
CHECKED Page
REVIEWED J.M. Tindall 7/6/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

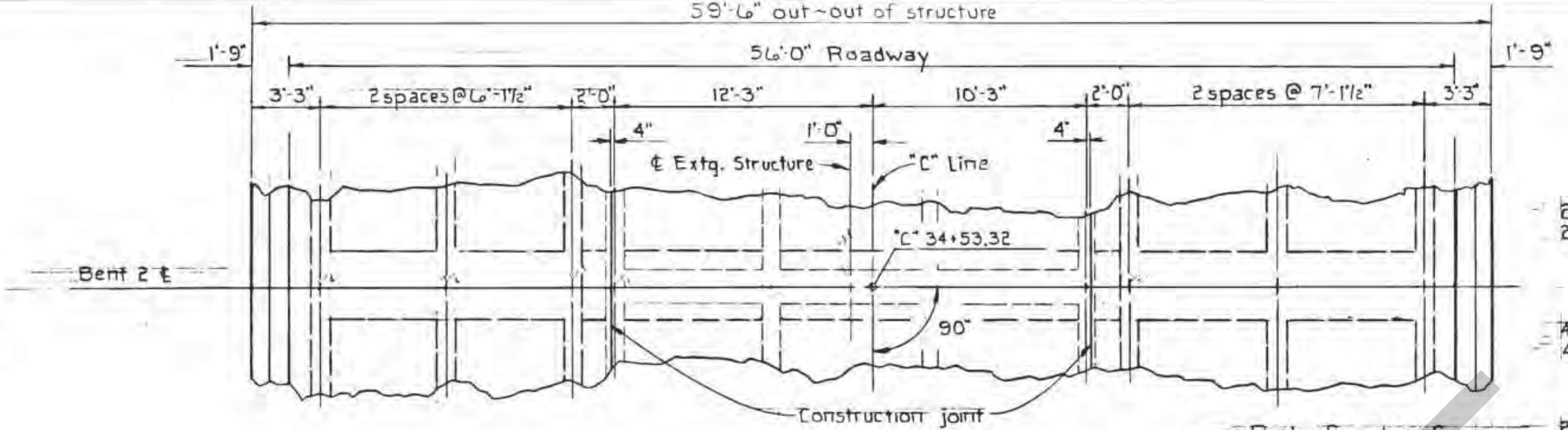
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

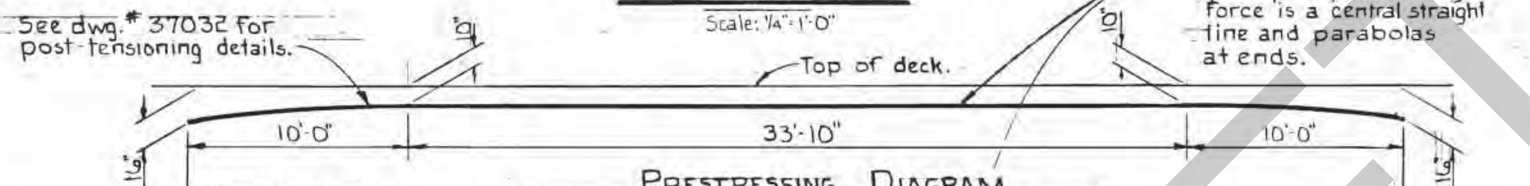
WILLAMETTE RIVER
(CENTER STREET) BRIDGE

PLAN & ELEVATION - ABUTMENT 1

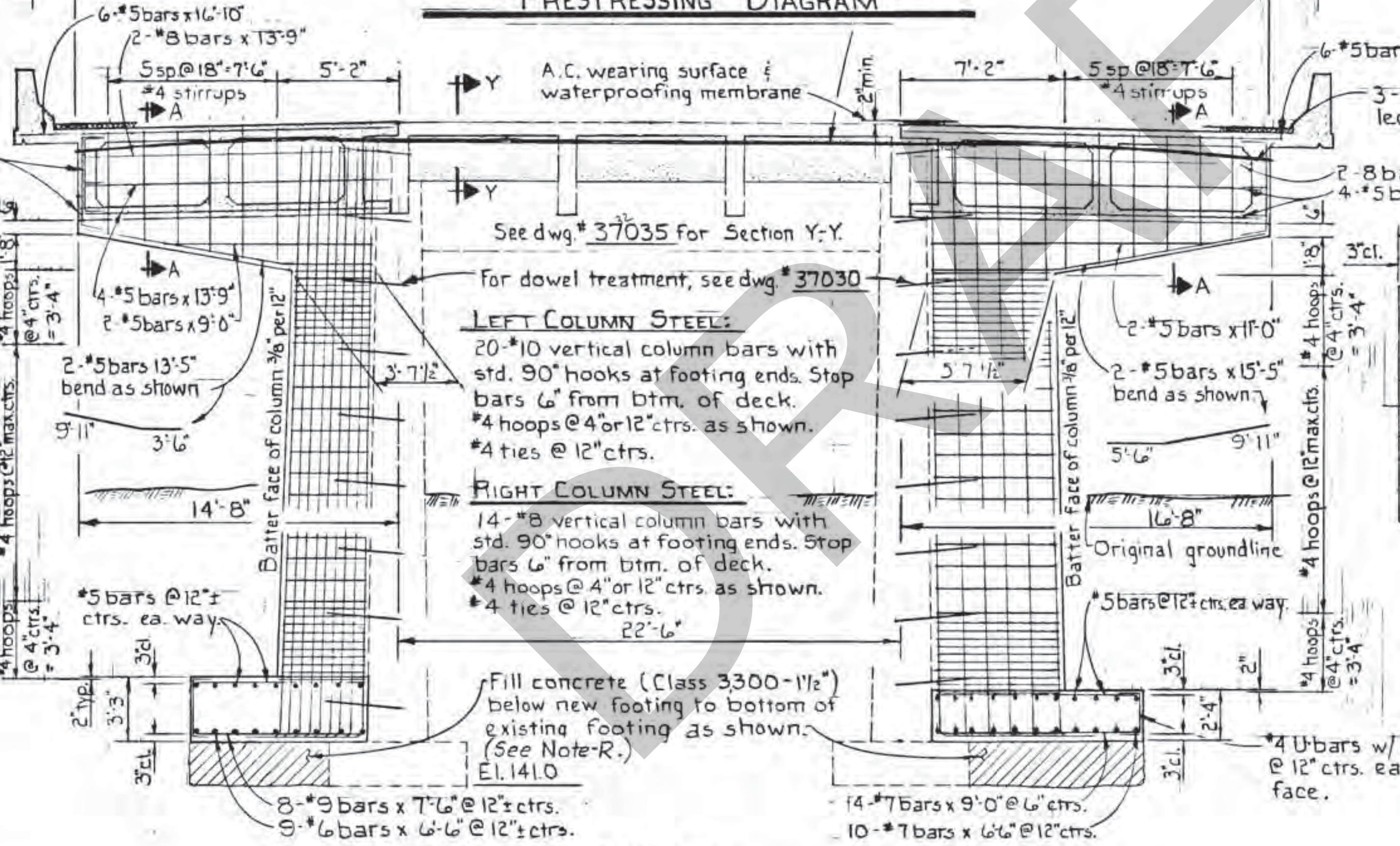
DATE May, 1982
SHEET 20 OF 155
BRIDGE NO. 123K
DRAWING NO. 37020

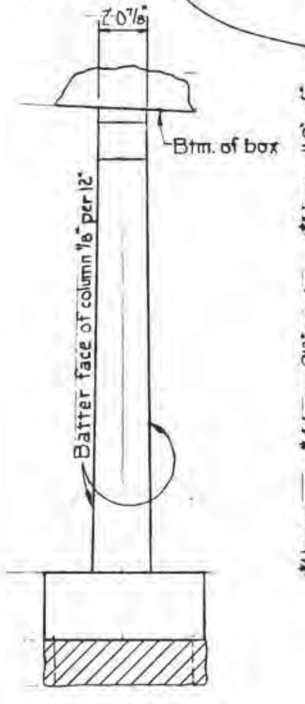


PLAN
Scale: ¼"=1'-0"



PRESTRESSING DIAGRAM



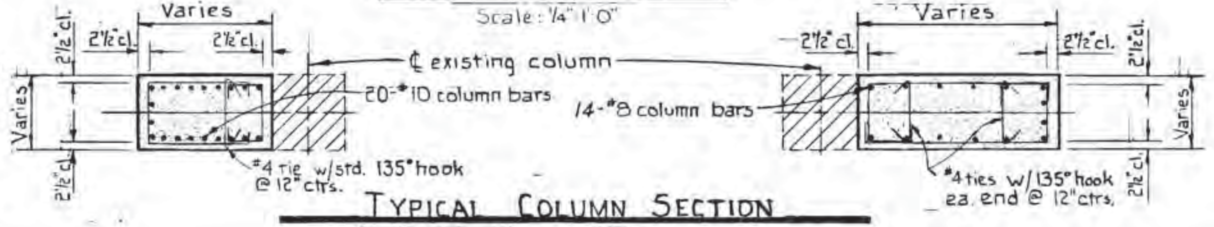


SIDE VIEW
Scale: ¼"=1'-0"

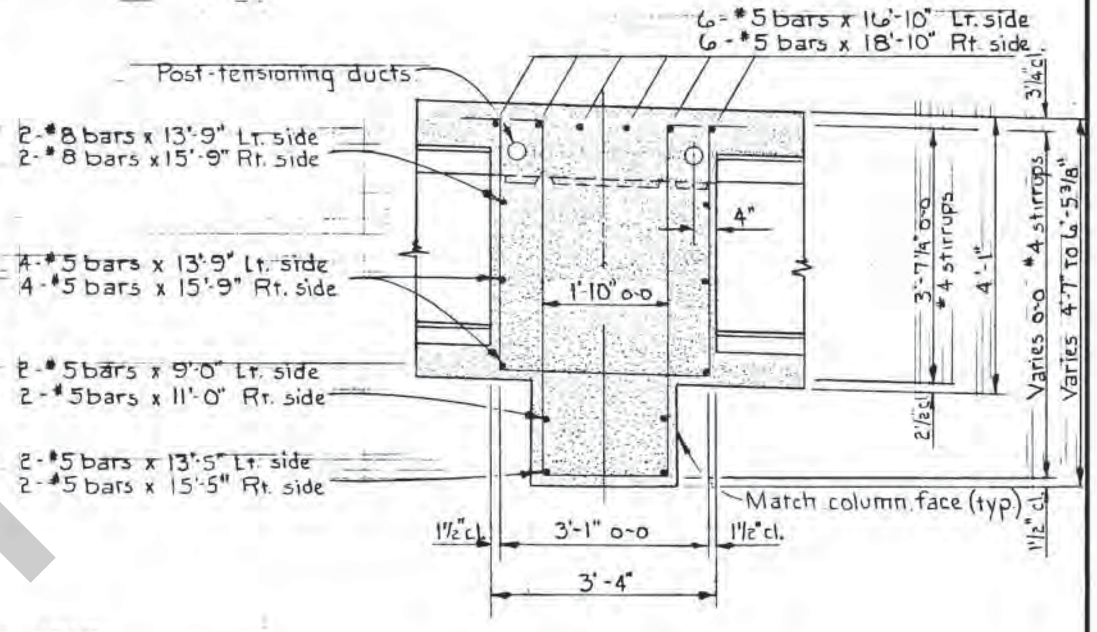
ELEVATION
Scale: ¼"=1'-0"

Req'd Bearing Pressures:
Left footing = 5.9 ton/ft.²
Right footing 5.3 ton/ft.²

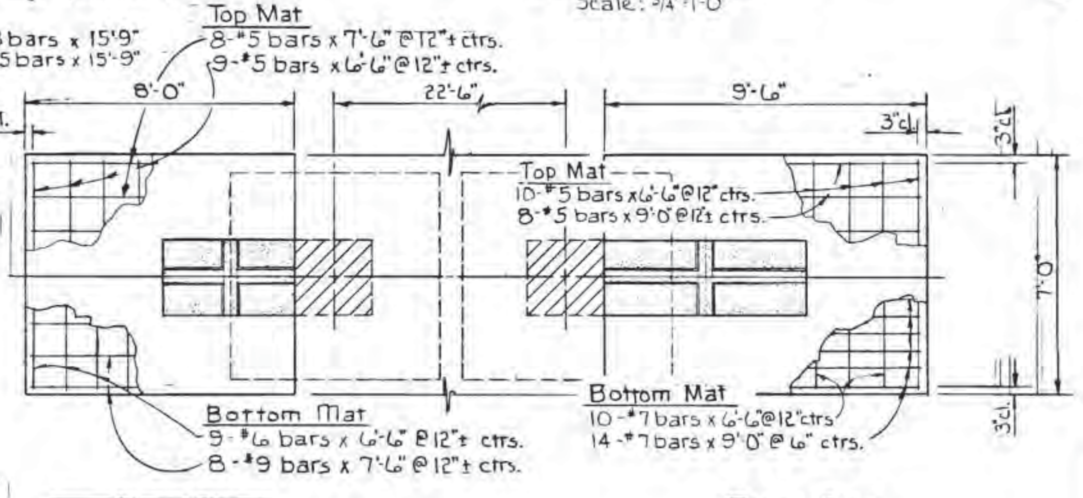
Note-R: Let fill concrete take initial set, then revibrate before pouring footings.



TYPICAL COLUMN SECTION
Scale: ¼"=1'-0"



SECTION A-A
Scale: ¾"=1'-0"



LEFT SIDE

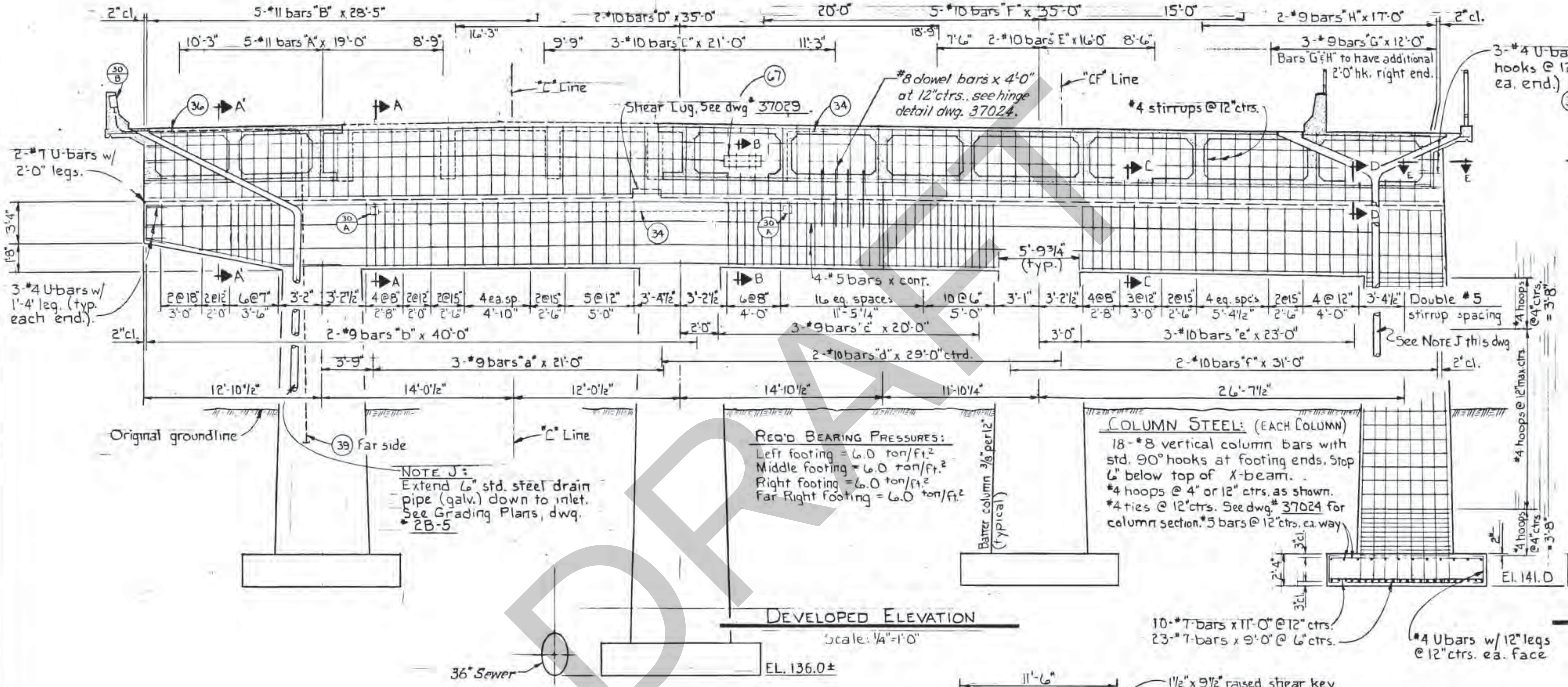
RIGHT SIDE

FOOTING PLAN
Scale: ⅜"=1'-0"

Note: See "Tie Bar Details" & "Column Hoop Layout" on dwg.# 37026.

APPROVED: Walter J. Hart, STRUCTURAL DESIGN ENGINEER	OREGON DEPARTMENT OF TRANSPORTATION STRUCTURAL DESIGN SECTION
DESIGNED W.M. Thompson DRAWN J. Silbernagel CHECKED Page REVIEWED J.M. Tindall 7/6/82 REVIEWED L.L. Wallace 7/6/82 CALC. BOOK 1832	WILLAMETTE RIVER (CENTER STREET) BRIDGE
DATE / REVISION	PLAN & ELEVATION - BENT 2
	DATE May, 1982 BRIDGE NO. 123K
	SHEET 23 OF 155 DRAWING NO. 37023

NOTE:
Remove existing crossbeam, columns & footings. See dwg. #37024 for required support capacity of existing Span 2.

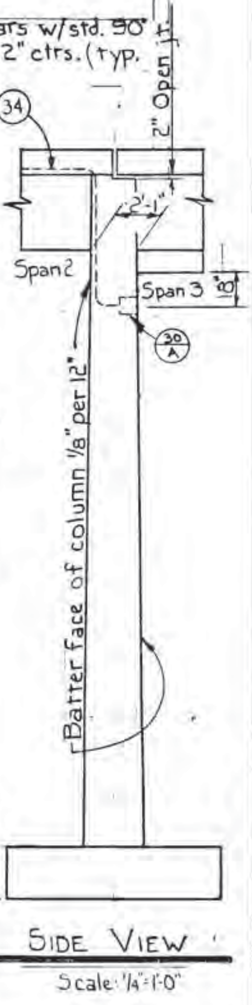


NOTE J:
Extend 6" std. steel drain pipe (galv.) down to inlet. See Grading Plans, dwg. #2B-5

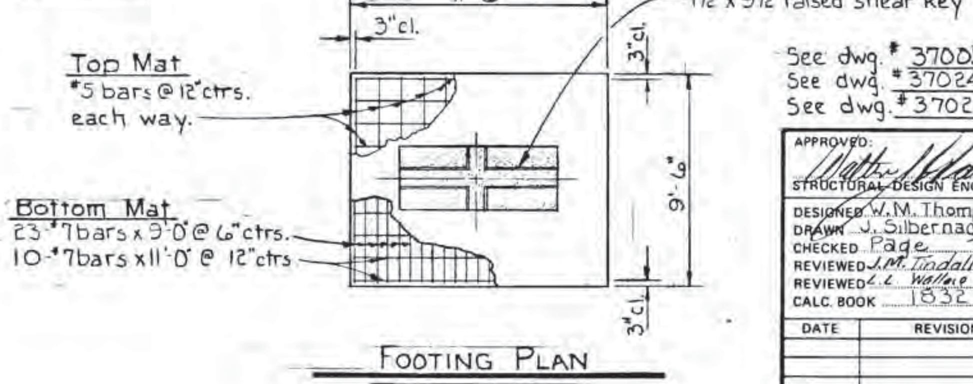
REQ'D BEARING PRESSURES:
Left footing = 6.0 ton/ft.²
Middle footing = 6.0 ton/ft.²
Right footing = 6.0 ton/ft.²
Far Right footing = 6.0 ton/ft.²

COLUMN STEEL: (EACH COLUMN)
18-#8 vertical column bars with std. 90° hooks at footing ends. Stop 6" below top of X-beam.
#4 hoops @ 4" or 12" ctrs. as shown.
#4 ties @ 12" ctrs. See dwg. #37024 for column section. #5 bars @ 12" ctrs. ea. way.

DEVELOPED ELEVATION
Scale: ¼"=1'-0"



SIDE VIEW
Scale: ¼"=1'-0"



FOOTING PLAN
Scale: ¼"=1'-0"

See dwg. #37005 for Detail Reference numbers.
See dwg. #37024 for Sections A-A, A'-A', B-B, C-C & D-D.
See dwg. #37029 for Section E-E.

APPROVED: Walter J. Hart
STRUCTURAL DESIGN ENGINEER
DESIGNED W. M. Thompson
DRAWN J. Silbernagel
CHECKED Page
REVIEWED J.M. Tindall 7/6/82
REVIEWED L.C. Wallace 7/6/82
CALC. BOOK 1832

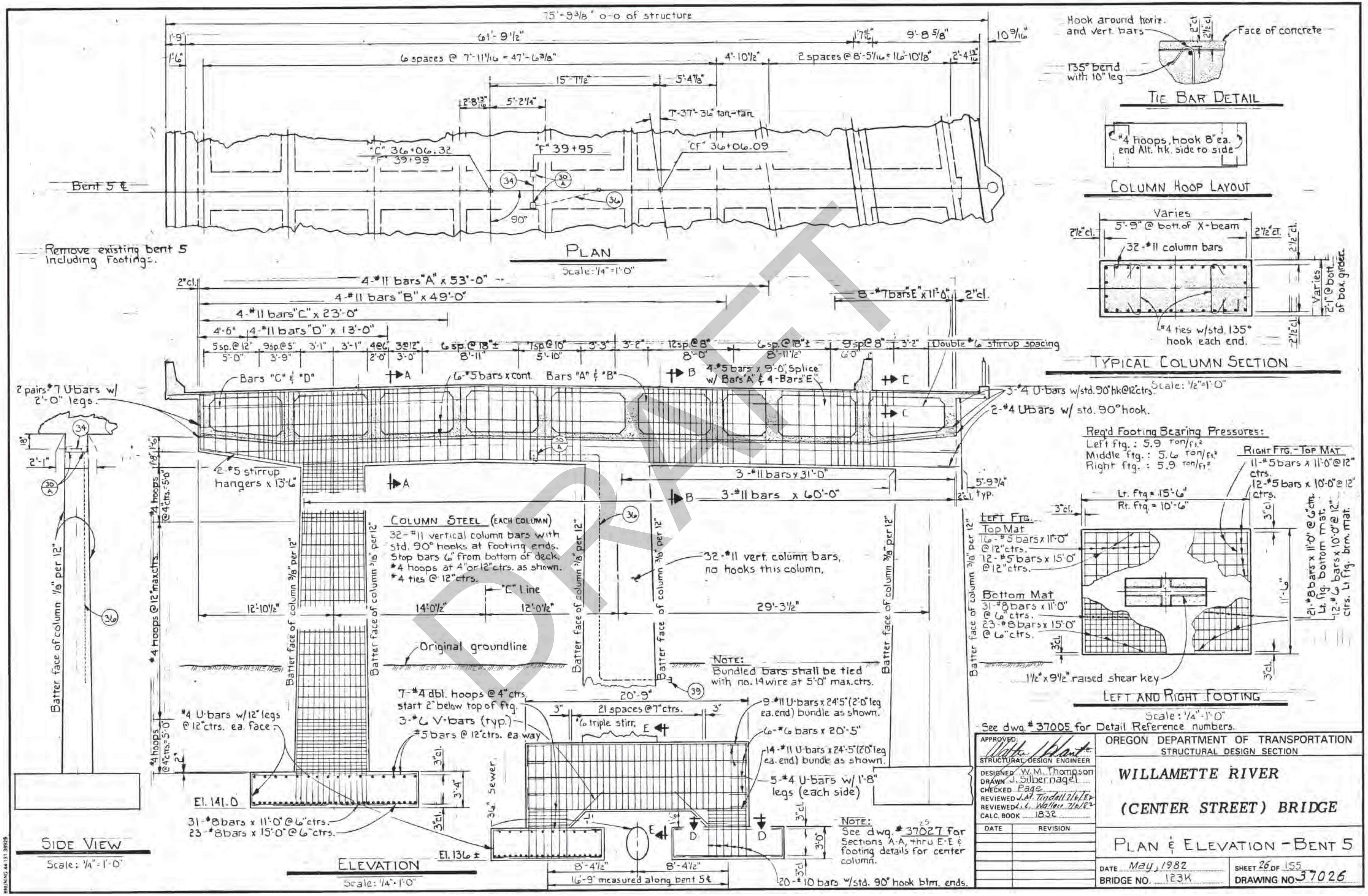
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

ELEVATION - BENT 3

DATE May, 1982
BRIDGE NO. 123K
SHEET 25 OF 155
DRAWING NO. 37025



PLAN
Scale: 1/4" = 1'-0"

TIE BAR DETAIL

COLUMN HOOP LAYOUT

TYPICAL COLUMN SECTION
Scale: 1/2" = 1'-0"

LEFT AND RIGHT FOOTING
Scale: 1/4" = 1'-0"

SIDE VIEW
Scale: 1/4" = 1'-0"

ELEVATION
Scale: 1/4" = 1'-0"

See dwg. #37005 for Detail Reference numbers.

APPROVED: Walter J. Hart
STRUCTURAL DESIGN ENGINEER
DESIGNED W.M. Thompson
DRAWN J. Silbernagel
CHECKED Page
REVIEWED J.M. Tyndall 7/6/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

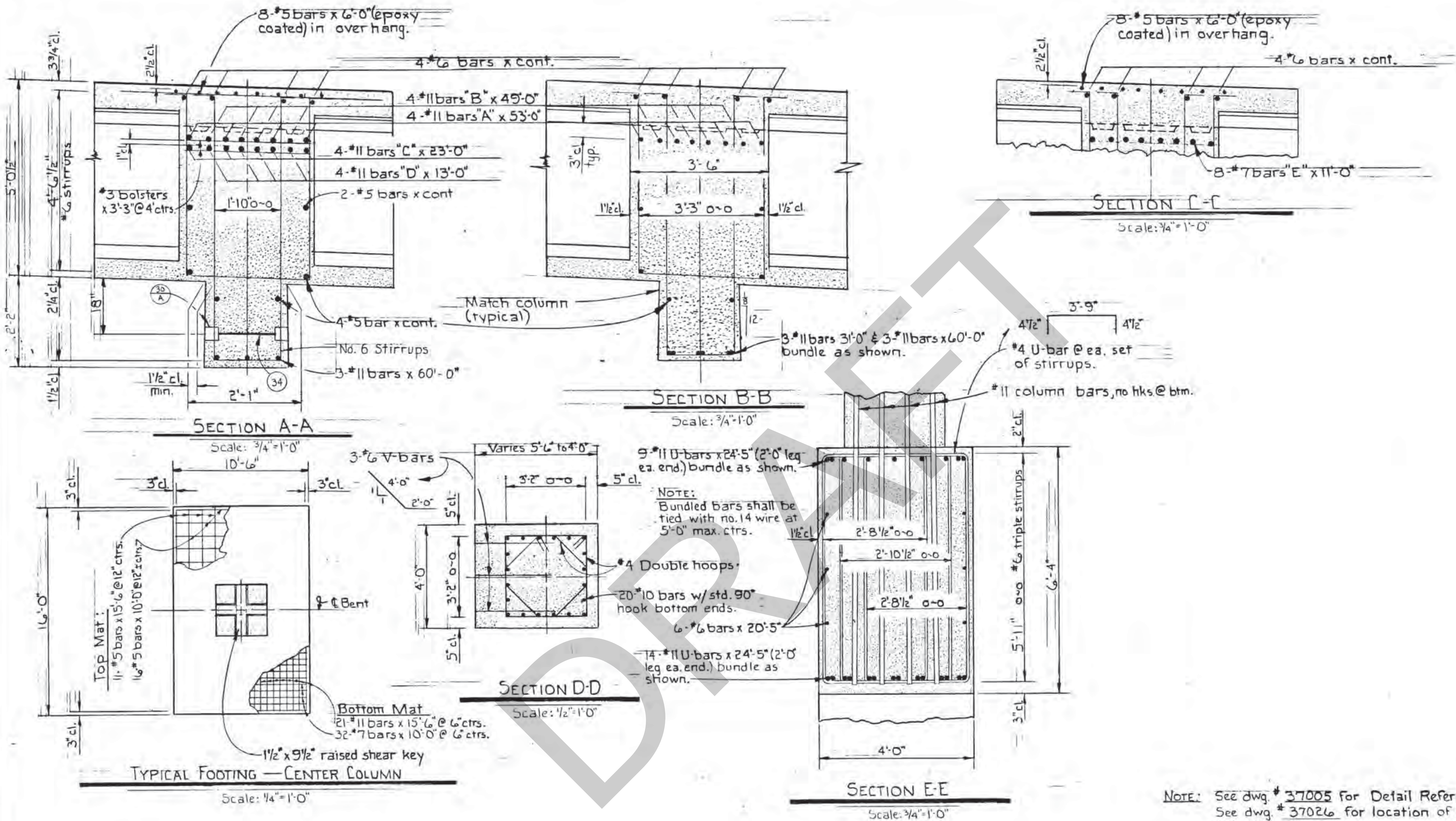
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER (CENTER STREET) BRIDGE

PLAN & ELEVATION - BENT 5

DATE May, 1982
SHEET 26 OF 155
BRIDGE NO. 123K
DRAWING NO 37026



SECTION A-A
Scale: ¾" = 1'-0"

SECTION B-B
Scale: ¾" - 1'-0"

SECTION C-C
Scale: ¾" = 1'-0"

TYPICAL FOOTING — CENTER COLUMN
Scale: ¼" = 1'-0"

SECTION D-D
Scale: ½" = 1'-0"

NOTE:
Bundled bars shall be tied with no. 14 wire at 5'-0" max. ctrs.

SECTION E-E
Scale: ¾" = 1'-0"

NOTE: See dwg. # 37005 for Detail Reference numbers
See dwg. # 37026 for location of Sections
A-A, thru E-E.

APPROVED: Walter Hart
STRUCTURAL DESIGN ENGINEER
DESIGNED W. M. Thompson
DRAWN J. Silbernagel
CHECKED Page
REVIEWED J. M. Tindall 7/6/82
REVIEWED L. C. Wallace 7/6/82
CALC. BOOK 1832

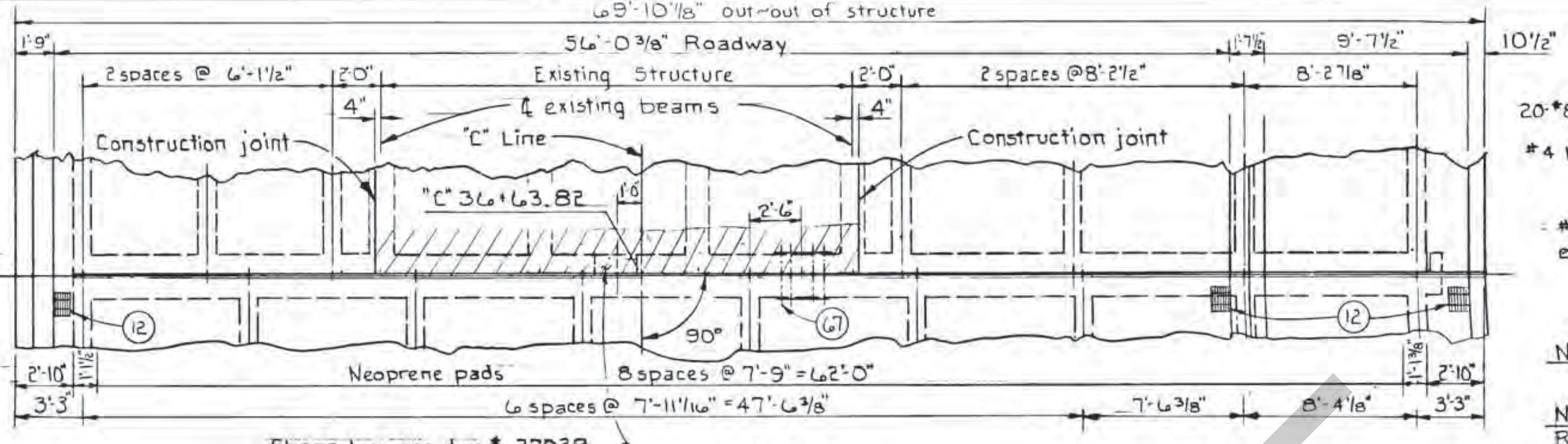
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

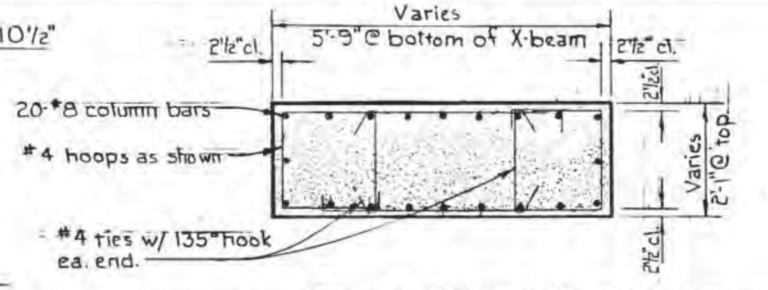
BENT-5 DETAILS

DATE May, 1982
BRIDGE NO. 123K
SHEET 27 OF 155
DRAWING NO. 37027



PLAN

Scale: ¼"=1'0"

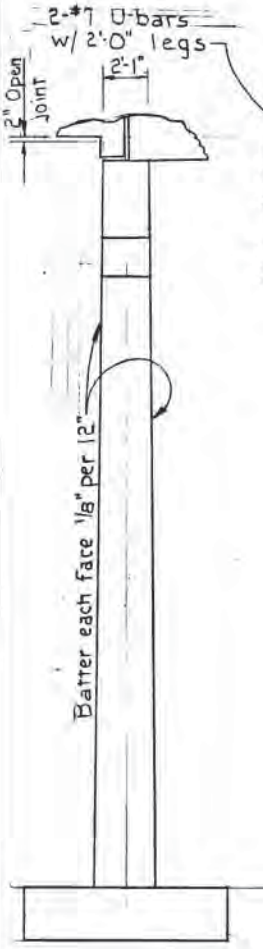


TYPICAL COLUMN SECTION

Scale: ½"=1'0"

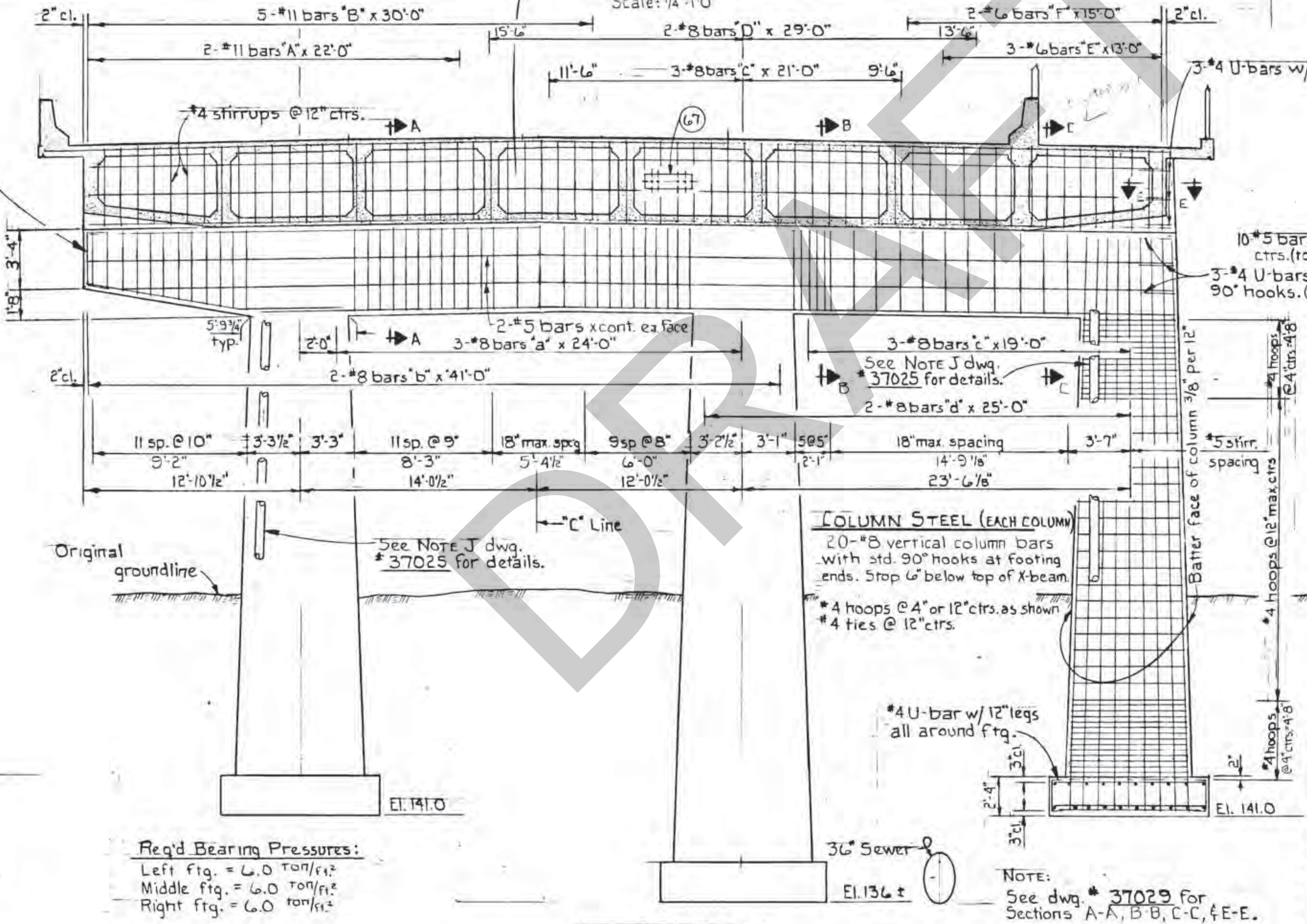
NOTE: See "Tie Bar Detail" & Column "Hoop Layout" on dwg.# 37026.

NOTE:
~~Remove existing crossbeam, columns, & footings.~~
See dwg. # 37029 for required support capacity
of existing Span 6.



SIDE VIEW

Scale: ¼"=1'0"



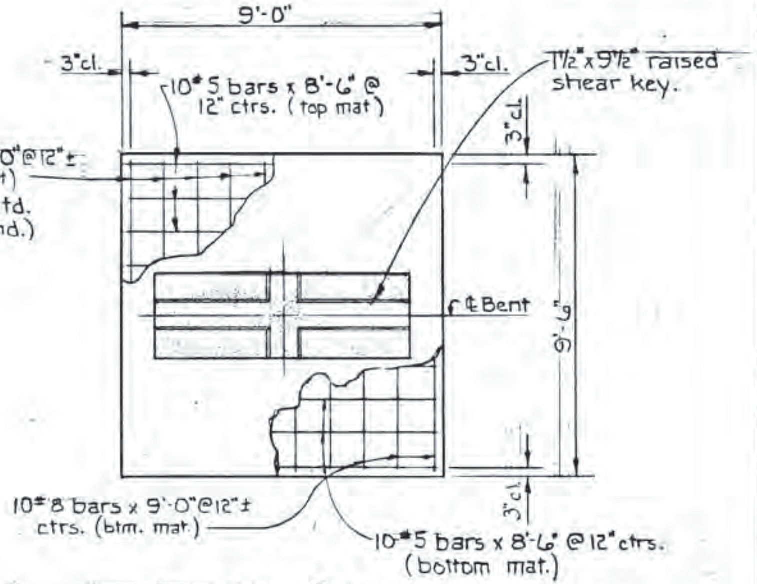
COLUMN STEEL (EACH COLUMN)
20-#8 vertical column bars with std. 90° hooks at footing ends. Stop 6" below top of X-beam.
#4 hoops @ 4" or 12" ctrs. as shown
#4 ties @ 12" ctrs.

Req'd Bearing Pressures:
Left ftg. = 6.0 Ton/ft²
Middle ftg. = 6.0 Ton/ft²
Right ftg. = 6.0 Ton/ft²

NOTE:
See dwg. # 37029 for Sections A-A, B-B, C-C, & E-E.

ELEVATION

Scale: ¼"=1'-0"



FOOTING PLAN

Scale: ⅜"=1'-0"

NOTE: See dwg. # 37005 for Detail Reference numbers

APPROVED: Walter M. Hart
STRUCTURAL DESIGN ENGINEER
DESIGNED W. M. Thompson
DRAWN J. Silbernagel
CHECKED Page
REVIEWED J.M. Tindall 7/6/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

DATE	REVISION

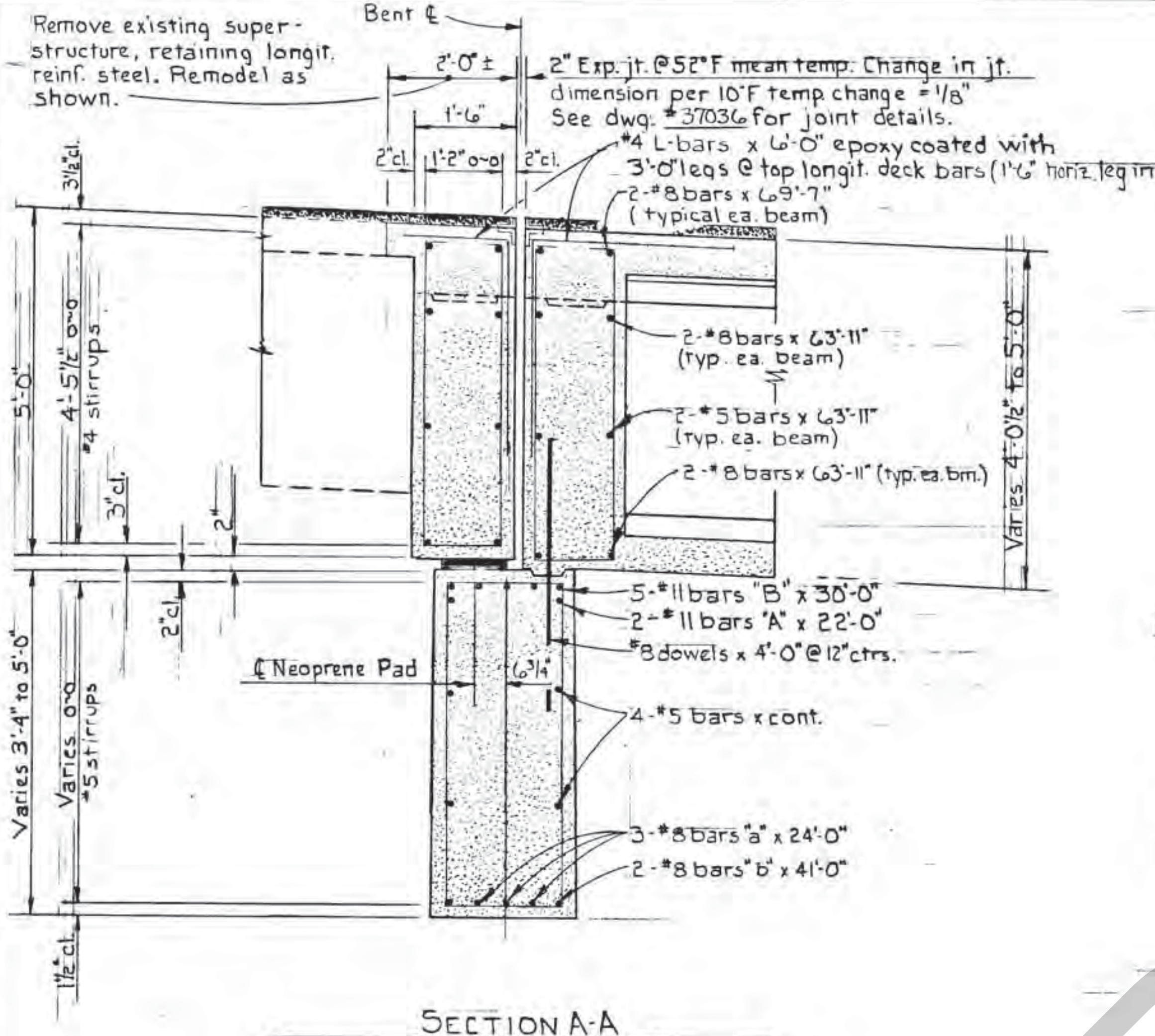
OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

PLAN & ELEVATION – BENT 6

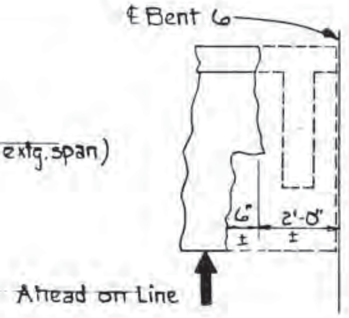
DATE May, 1982
BRIDGE NO. 123K
SHEET 28 OF 155
DRAWING NO. 37028

BRUNING 44-131 38925

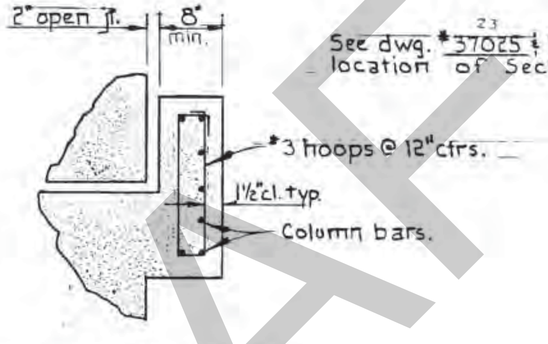


SECTION A-A

Scale: 3/4" = 1'-0"



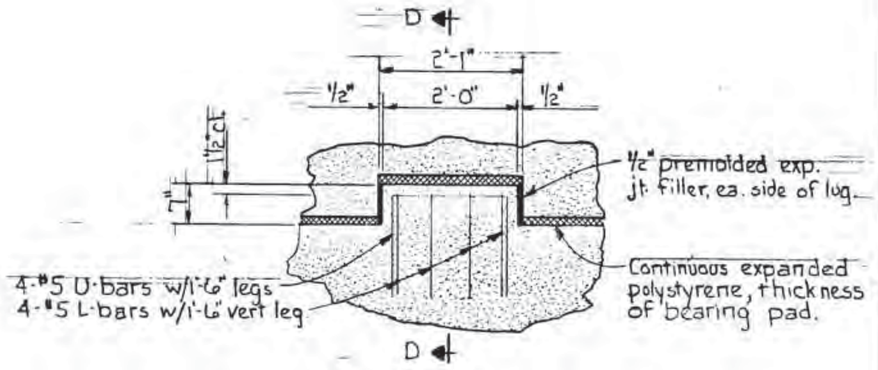
NOTE:
Support existing beams on falsework
with a minimum capacity of:
20 tons per beam (Bent 6)



See dwg. #37025 & #37028 for location of Section E-E.

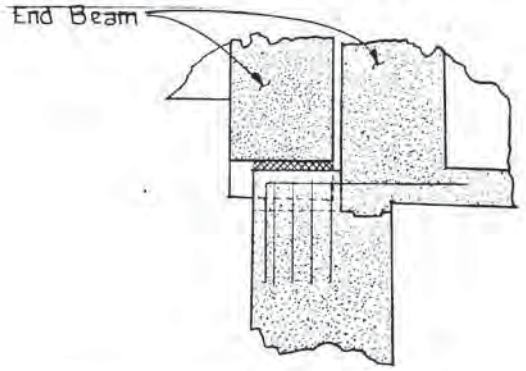
SECTION E-E

Scale: 1" = 1'-0"



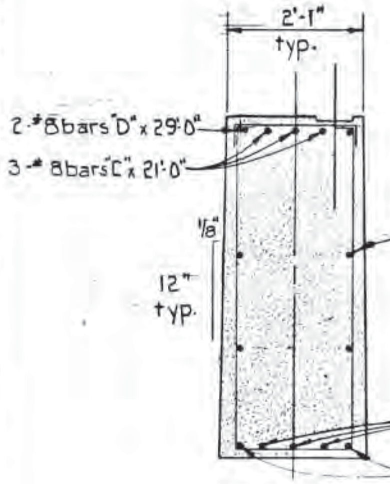
ELEVATION-SHEAR LUG

Scale: 3/4" = 1'-0"



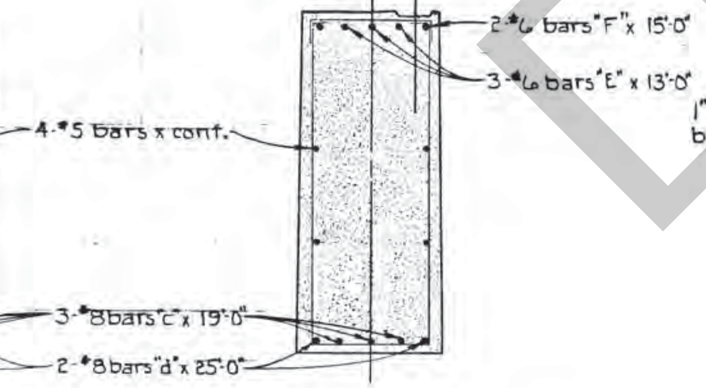
SECTION D-D

Scale: 3/4" = 1'-0"



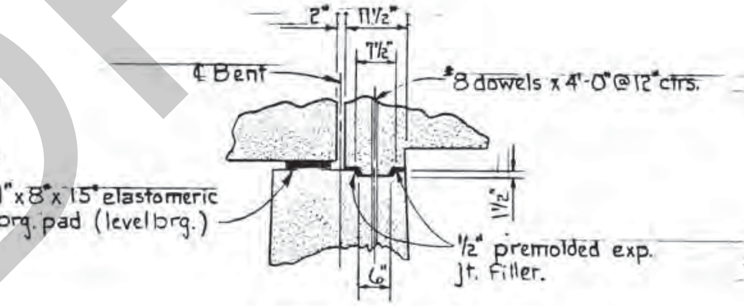
SECTION B-B

Scale: 3/4" = 1'-0"



SECTION C-C

Scale: 3/4" = 1'-0"

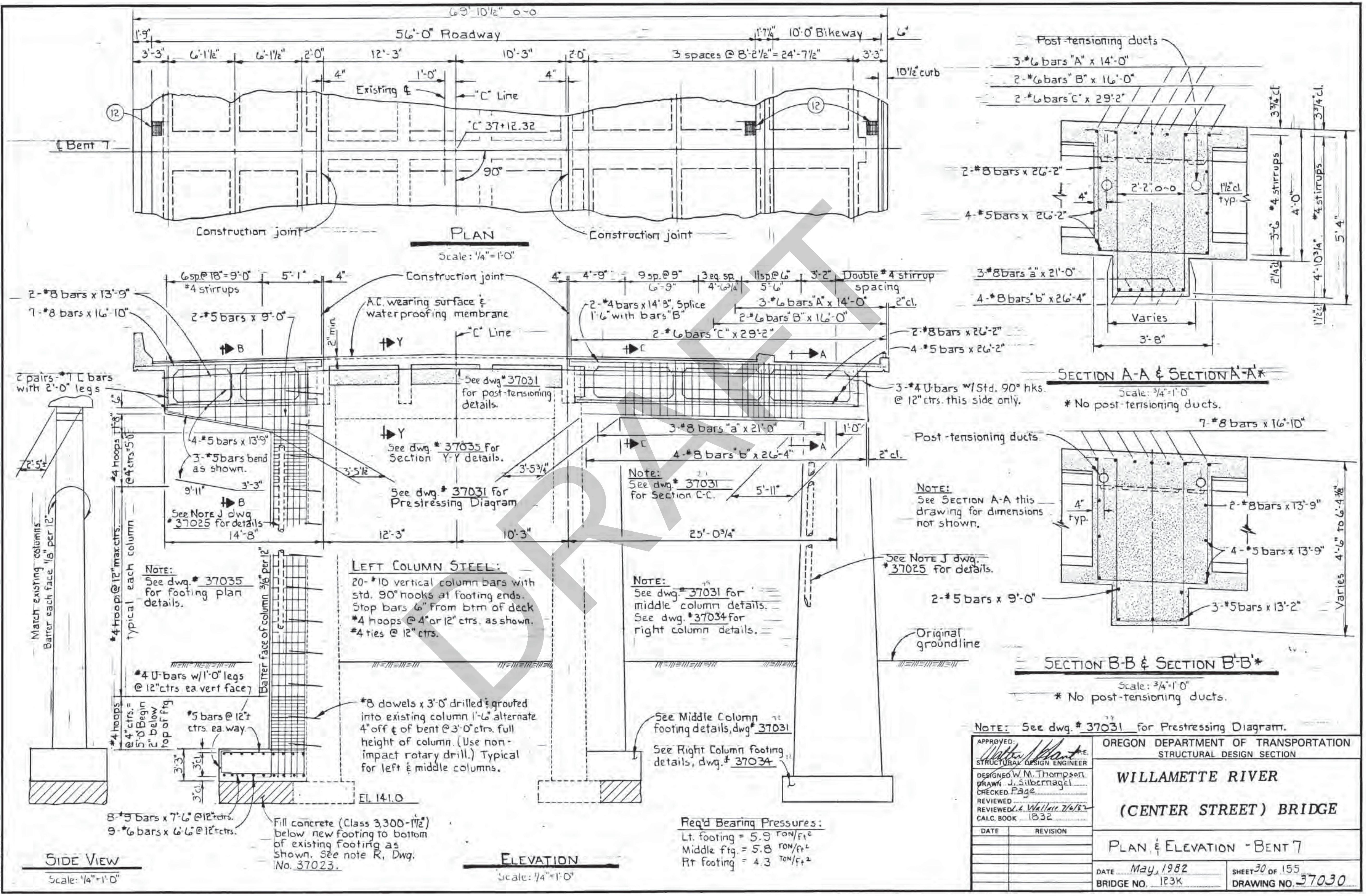


HINGE DETAILS

Scale: 3/4" = 1'-0"

NOTE:
See dwg #37028 for location
of Sections A-A, B-B, & C-C.

APPROVED: Walter J. Hart P.E. STRUCTURAL DESIGN ENGINEER	OREGON DEPARTMENT OF TRANSPORTATION STRUCTURAL DESIGN SECTION
DESIGNED W. M. Thompson DRAWN J. Silbernagel CHECKED Page REVIEWED J.M. Tindall 7/6/82 REVIEWED L.L. Wallace 7/6/82 CALC. BOOK 1832	WILLAMETTE RIVER (CENTER STREET) BRIDGE
DATE / REVISION	BENT 6 DETAILS
	DATE May, 1982 BRIDGE NO. 123K
	SHEET 29 OF 155 DRAWING NO. 37029



PLAN

Scale: ¼" = 1'-0"

SECTION A-A & SECTION A'-A'*

Scale: ¾" = 1'-0"

* No post-tensioning ducts.

NOTE: See dwg. #37035 for footing plan details.

LEFT COLUMN STEEL:
20-#10 vertical column bars with std. 90° hooks at footing ends. Stop bars 6" from btm of deck. #4 hoops @ 4" or 12" ctrs. as shown. #4 ties @ 12" ctrs.

#8 dowels x 3'-0" drilled & grouted into existing column 1'-6" alternate 4" off ℄ of bent @ 3'-0" ctrs. full height of column. (Use non-impact rotary drill.) Typical for left & middle columns.

Fill concrete (Class 3,300-1½") below new footing to bottom of existing footing as shown. See note R, Dwg. No. 37023.

NOTE: See dwg. #37031 for middle column details. See dwg. #37034 for right column details.

NOTE: See SECTION A-A this drawing for dimensions not shown.

SECTION B-B & SECTION B'-B'*

Scale: ¾" = 1'-0"

* No post-tensioning ducts.

Req'd Bearing Pressures:
Lt. footing = 5.9 TON/FT²
Middle ftg. = 5.8 TON/FT²
Rt footing = 4.3 TON/FT²

SIDE VIEW

Scale: ¼" = 1'-0"

ELEVATION

Scale: ¼" = 1'-0"

NOTE: See dwg. #37031 for Prestressing Diagram.

APPROVED: P.E.
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DESIGNED W. M. Thompson
DRAWN J. Silbernagel
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CALC. BOOK 1832

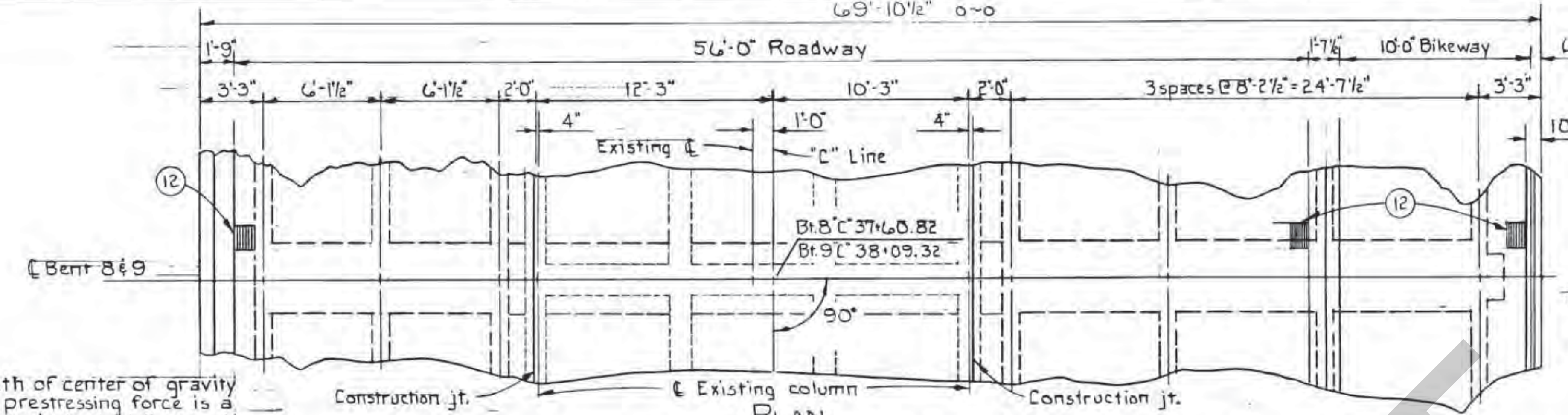
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

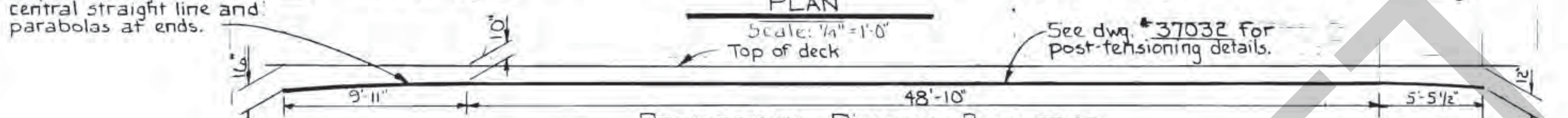
PLAN & ELEVATION - BENT 7

DATE May, 1982
BRIDGE NO. 123K
SHEET 30 OF 155
DRAWING NO. 37030

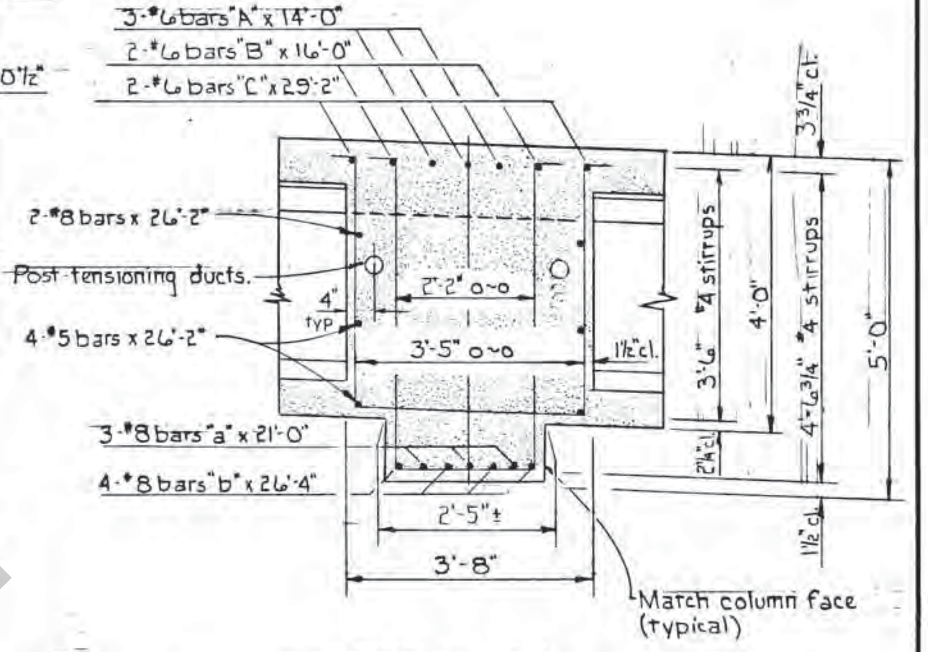


PLAN

Scale: ¼" = 1'-0"

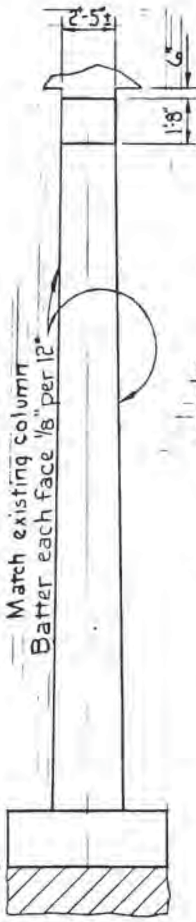


PRESTRESSING DIAGRAM BENTS 7, 8, & 9



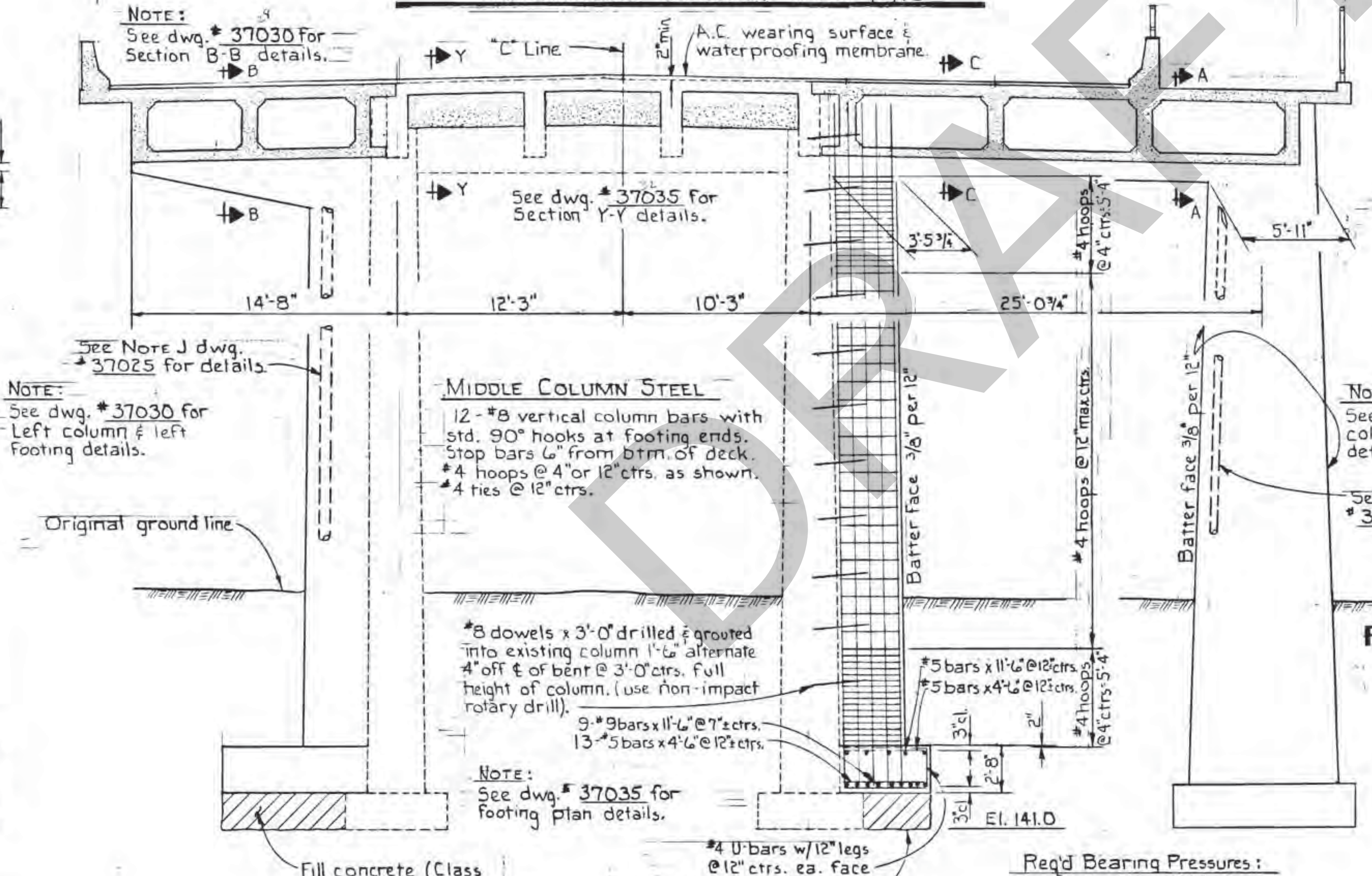
SECTION A-A

Scale: ¾" = 1'-0"



SIDE VIEW

Scale: ¼" = 1'-0"



MIDDLE COLUMN STEEL

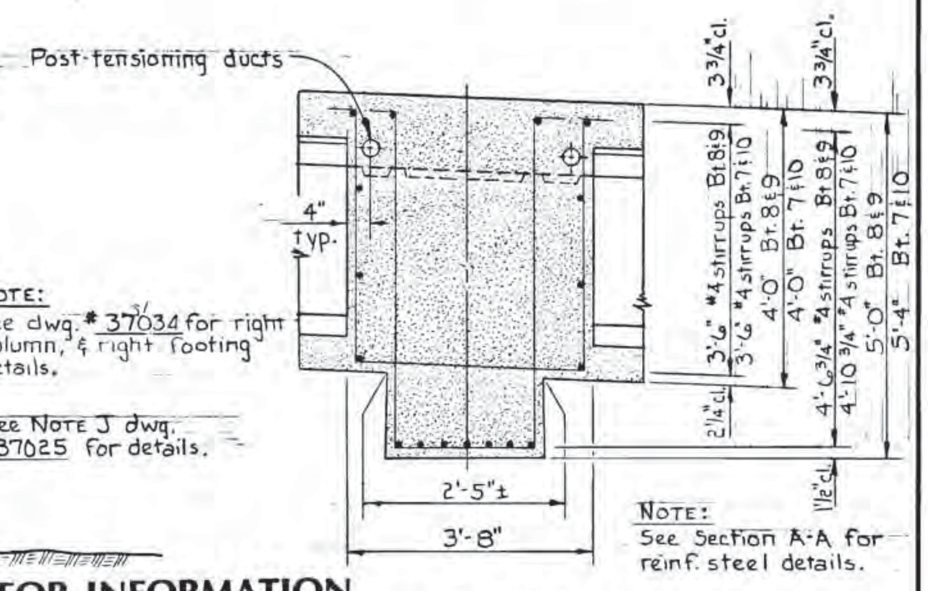
12-#8 vertical column bars with std. 90° hooks at footing ends. Stop bars 6" from btm. of deck.
#4 hoops @ 4" or 12" ctrs. as shown.
#4 ties @ 12" ctrs.

ELEVATION

Scale: ¼" = 1'-0"

Req'd Bearing Pressures:

Lt. footing = 5.9 TON/Ft²
Middle Ftg. = 5.7 TON/Ft²
Rt. Footing = 4.5 TON/Ft²



NOTE: See Section A-A for reinf. steel details.

SECTION C-C

Scale: ¾" = 1'-0"

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DESIGNED W.M. Thompson
DRAWN J. Silbernagel
CHECKED Page
REVIEWED J.M. Tindall 7/6/82
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CALC. BOOK 1832

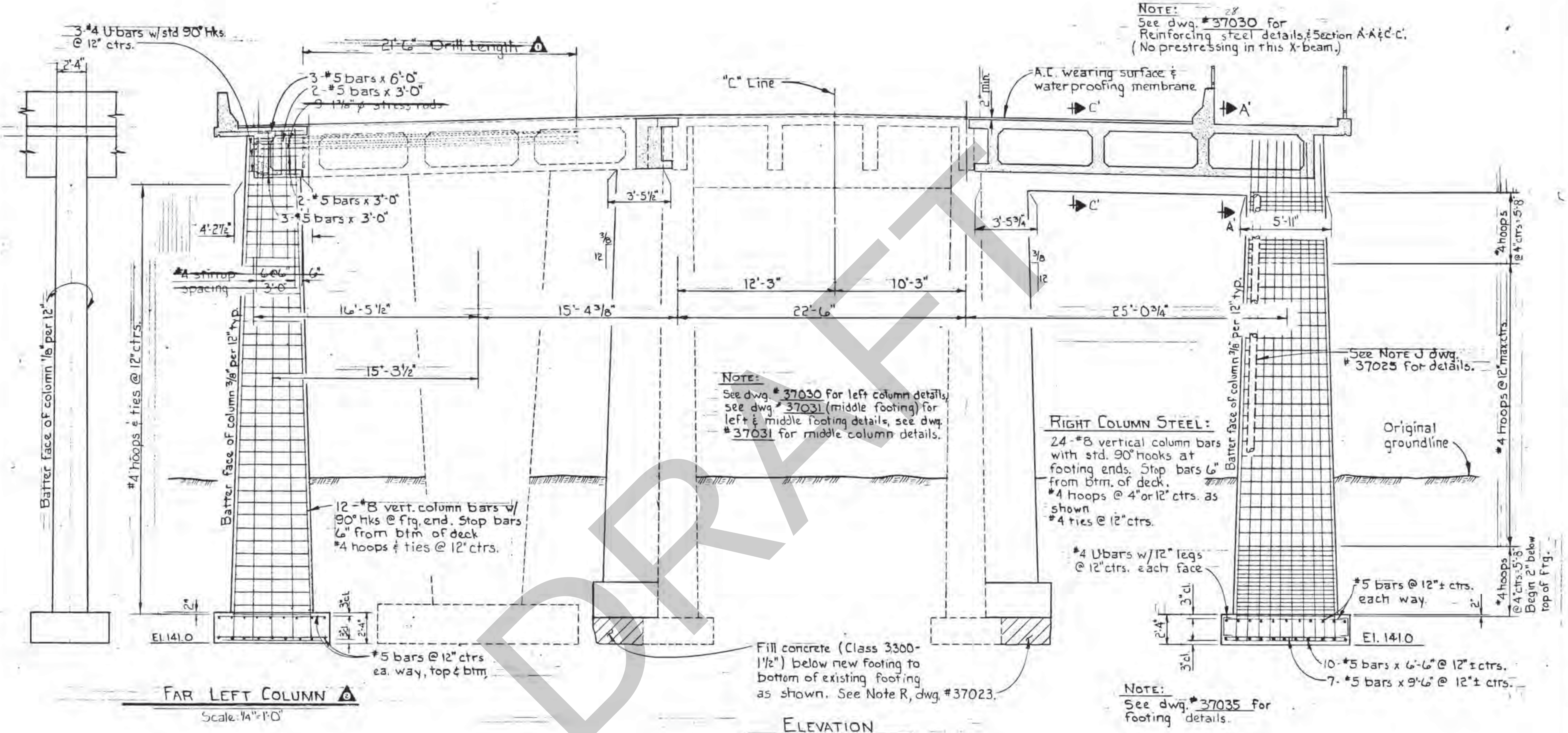
DATE	REVISION
1-12-88	AS CONSTRUCTED

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER (CENTER STREET) BRIDGE

PLAN & ELEVATION - BENTS 8 & 9

DATE May, 1982
BRIDGE NO. 123K
SHEET 31 OF 155
DRAWING NO. 37031



NOTE:
See dwg. #37030 for Reinforcing steel details, & Section A-A & C-C.
(No prestressing in this X-beam.)

NOTE:
See dwg. #37030 for left column details, see dwg. #37031 (middle footing) for left & middle footing details, see dwg. #37031 for middle column details.

RIGHT COLUMN STEEL:
24-#8 vertical column bars with std. 90° hooks at footing ends. Stop bars 6" from btm. of deck.
#4 hoops @ 4" or 12" ctrs. as shown
#4 ties @ 12" ctrs.

NOTE:
See dwg. #37035 for footing details.

FAR LEFT COLUMN ②
Scale: ¼" = 1'-0"

ELEVATION
Scale: ¼" = 1'-0"

FOR INFORMATION ONLY

APPROVED: Walter J. Hart P.E.
STRUCTURAL DESIGN ENGINEER
DESIGNED W.M. Thompson
DRAWN J. Silbernagel
CHECKED Page
REVIEWED J.M. Tindall 7/6/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

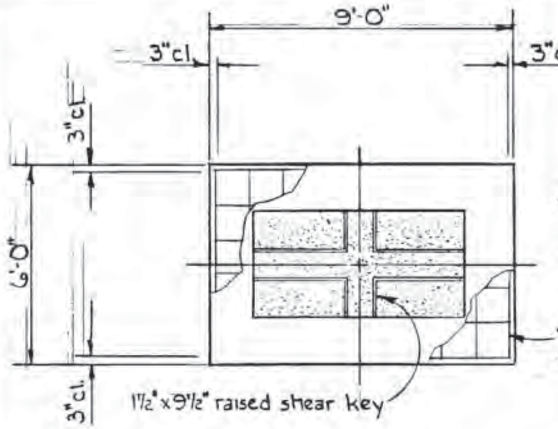
DATE	REVISION
① 4-26-83	Stress Rods removed
② 4-26-83	Col. & Ftg. added

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

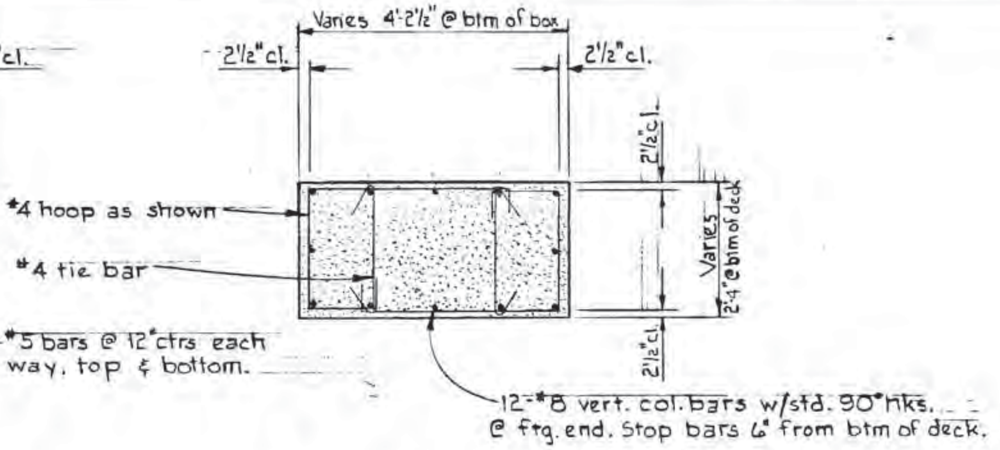
ELEVATION - BENT 10

DATE May, 1982
BRIDGE NO. 123K
SHEET 34 OF 155
DRAWING NO. 37034



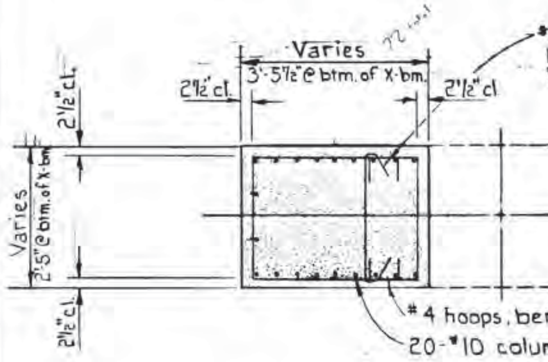
① PLAN - FAR LEFT FOOTING

Scale: 3/8"=1'-0"



① SECTION - FAR LEFT COLUMN

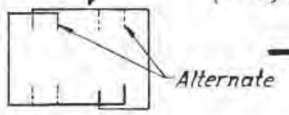
Scale: ½"=1'-0"



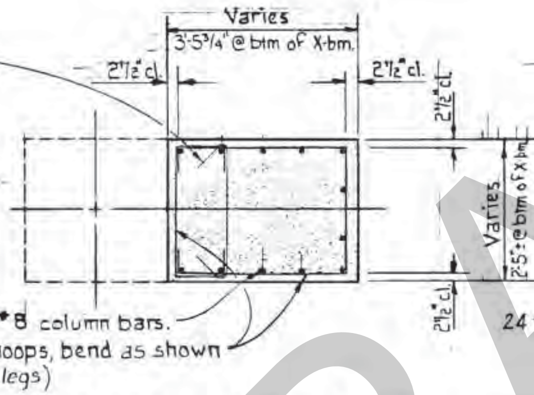
NOTE:
All sections shown typical for Bents 7, 8, 9, & 10.

SECTION - LEFT COLUMN

Scale: ½"=1'-0"

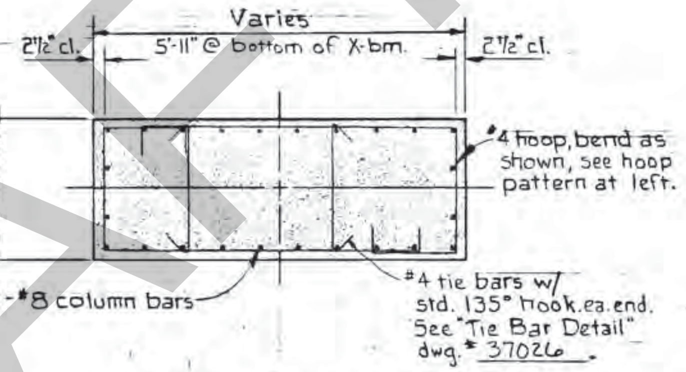


HOOP PATTERN



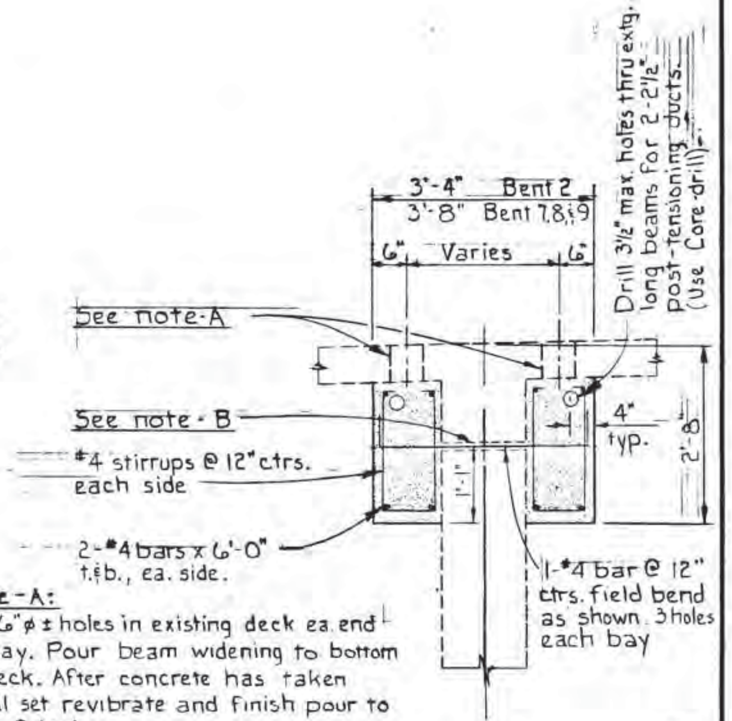
SECTION - MIDDLE COLUMN

Scale: ½"=1'-0"



SECTION - RIGHT COLUMN

Scale: ½"=1'-0"



NOTE-A:
Cut 6"ø± holes in existing deck ea. end ea. bay. Pour beam widening to bottom of deck. After concrete has taken initial set revibrate and finish pour to top of deck.

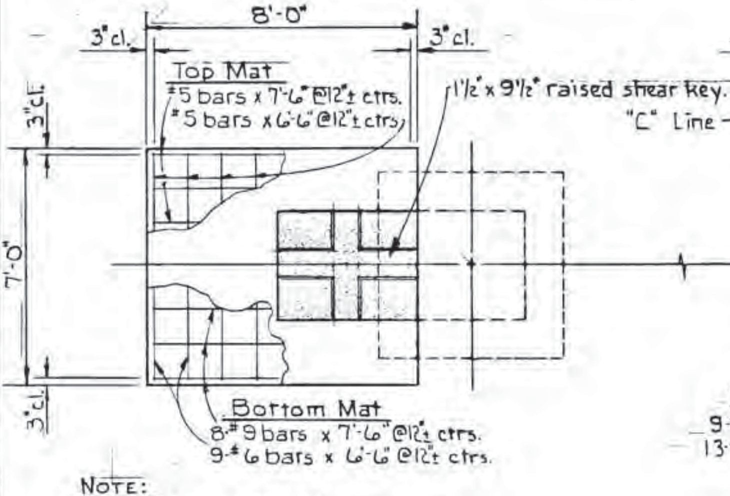
NOTE-B:
Drill 1"ø holes for #4 bars @ 12"ctrs. btwn. beams, ctr. Use non-impact type drill.

NOTE:
See dwg. #37023, #37030 & #37031 for location Section Y-Y.

FOR INFORMATION ONLY

SECTION Y-Y

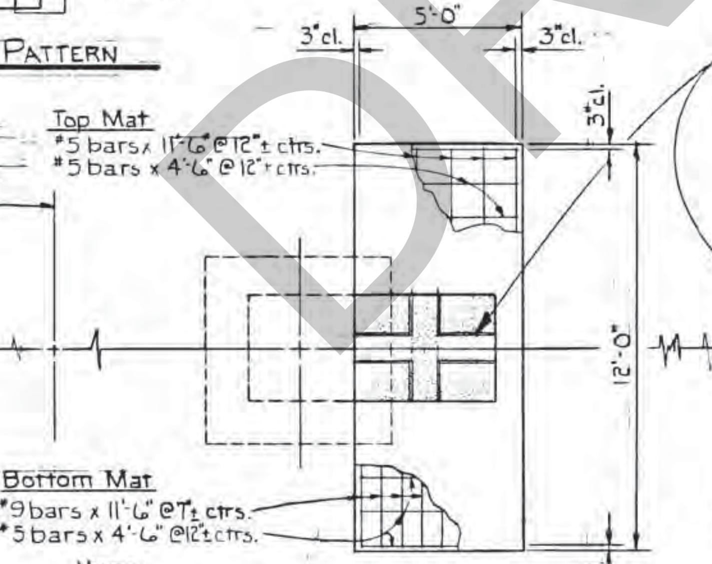
Scale: ¾"=1'-0"



NOTE:
Footing shown typical for bents 7, 8, & 9. See Middle Ftg. (opp. hand.) for Bent 10 left Ftg.

PLAN - LEFT FOOTING

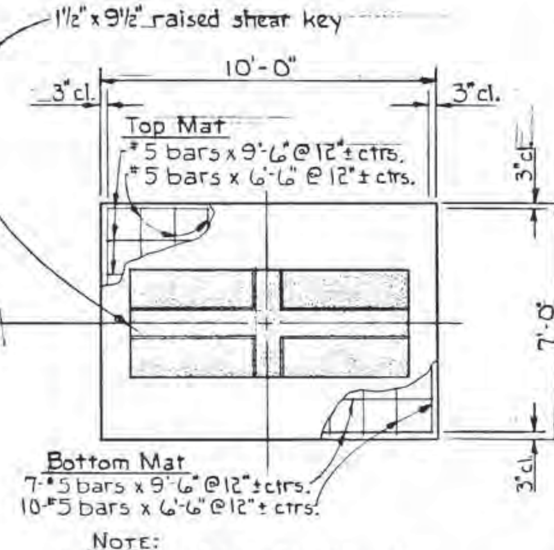
Scale: 3/8"=1'-0"



NOTE:
Footing shown typical for bents 7, 8, 9, & 10.

PLAN - MIDDLE FOOTING

Scale: 3/8"=1'-0"



NOTE:
Footing shown typical for bents 7, 8, 9, & 10.

PLAN - RIGHT FOOTING

Scale: 3/8"=1'-0"

APPROVED: Walter J. Hart
STRUCTURAL DESIGN ENGINEER
DESIGNED W.M. Thompson
DRAWN J. Silbernagel
CHECKED Page
REVIEWED J.M. Tindall 7/6/82
REVIEWED C.L. Wallace 7/6/82
CALC. BOOK 1832

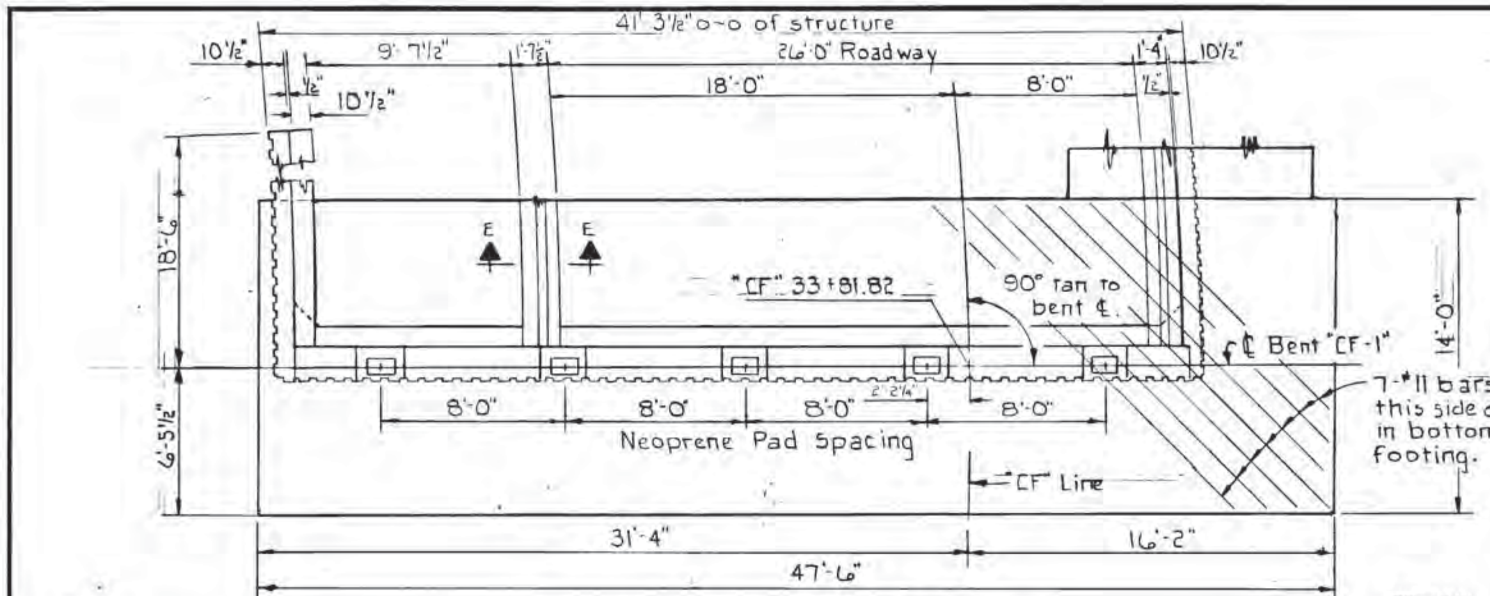
DATE	REVISION
① 4-26-83	Added Far Lt. col. & Ftg.

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER (CENTER STREET) BRIDGE

BENT 7, 8, 9, & 10 DETAILS

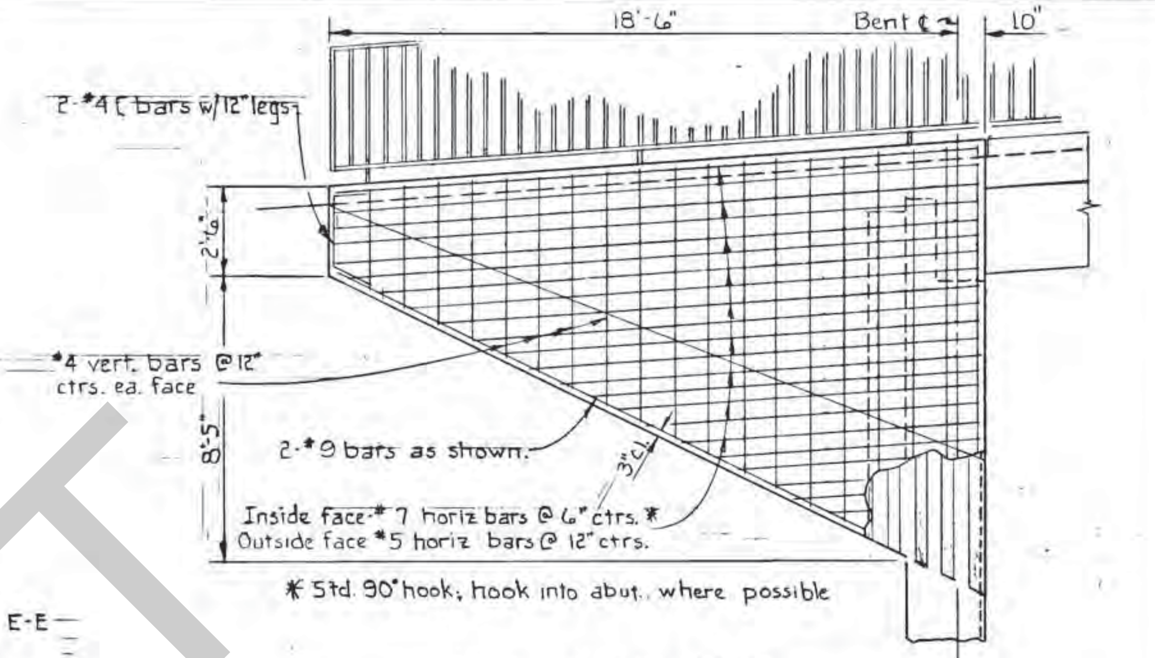
DATE May, 1982
SHEET 35 OF 155
BRIDGE NO. 123K
DRAWING NO. 37035



PLAN - BENT "CF"-1

Scale: ¼"=1'-0"

NOTE:
Looking back on Station.
Box girder, Reinf. conc. end panel omitted.

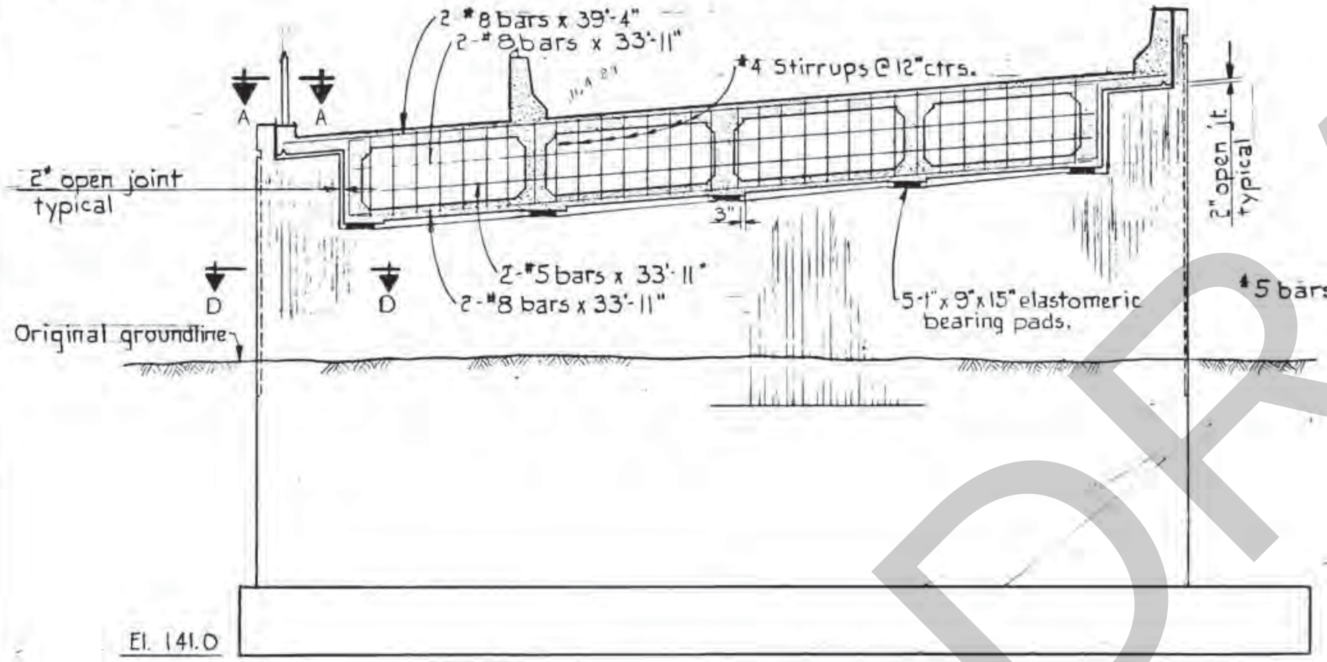


* Std. 90° hook; hook into abut. where possible

WINGWALL - BENT "CF"-1

Scale: ⅜"=1'-0"

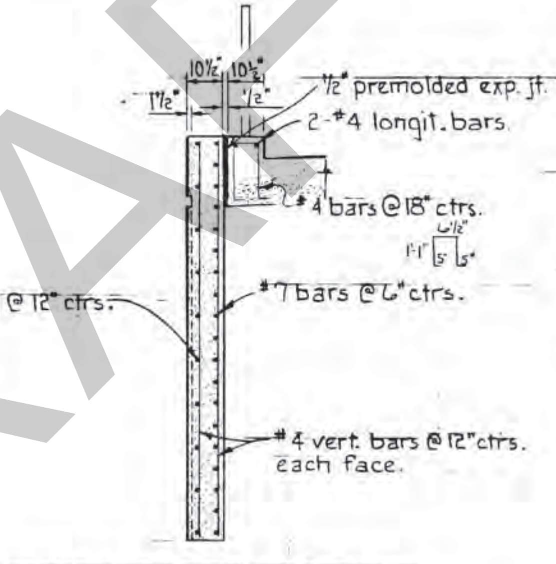
NOTE:
See Sections C-C thru E-E dwg.# 37039.



ELEVATION - BENT "CF"-1

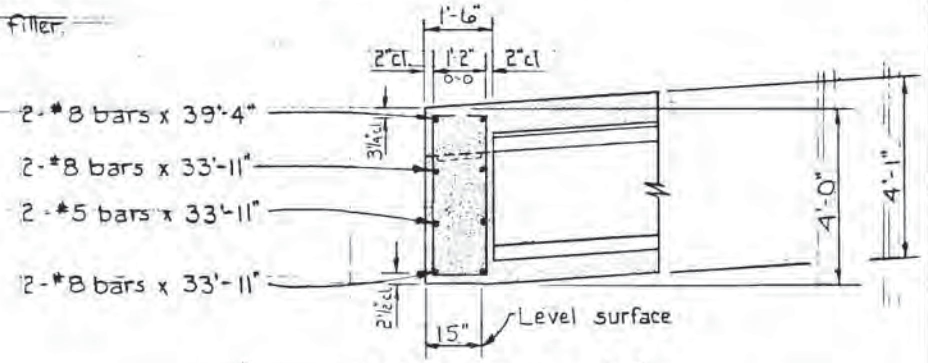
Scale: ¼"=1'-0"
(Looking back on Station)

NOTE:
See "Cap Detail" dwg. #37021 for details.



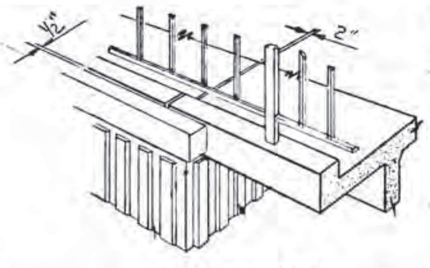
TYPICAL SECTION-WINGWALL

Scale: ½"=1'-0"

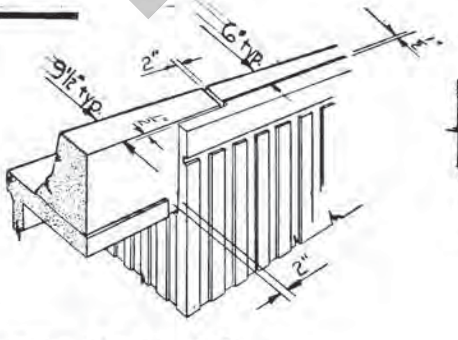


X-BEAM - BENT "CF"-1

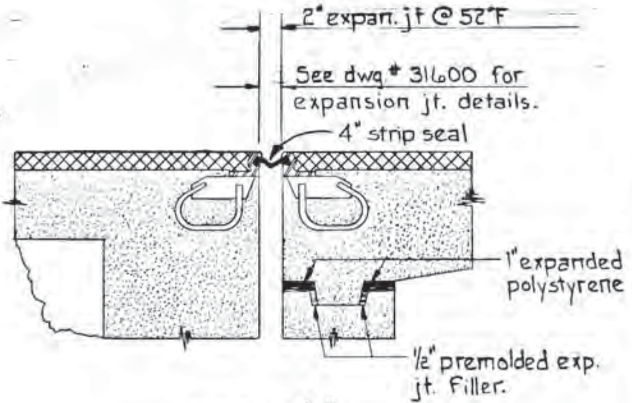
Scale: ½"=1'-0"



VIEW A-A



VIEW B-B



DETAIL "A"

APPROVED: Walter J. Hart
STRUCTURAL DESIGN ENGINEER
DESIGNED W. M. Thompson
DRAWN J. Silbernagel
CHECKED Page
REVIEWED J. M. Tisdall 7/6/82
REVIEWED L. L. Wallace 7/6/82
CALC. BOOK 1832

DATE	REVISION

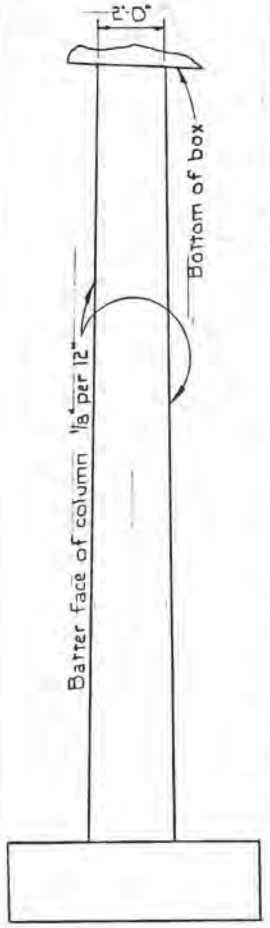
OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

PLAN & ELEVATION - BENT "CF"-1

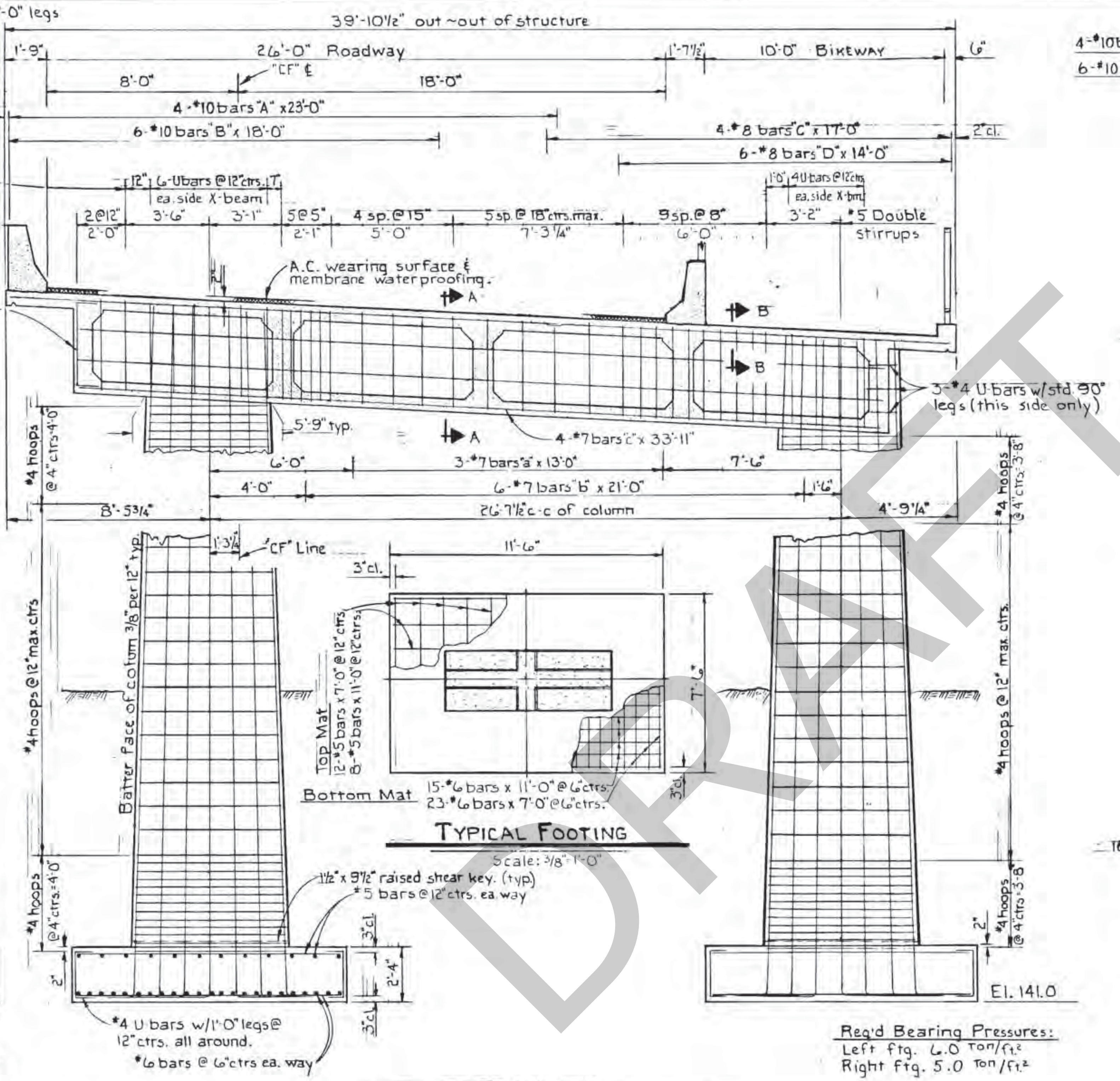
DATE May, 1982
SHEET 36 OF 155
BRIDGE NO. 123K
DRAWING NO. 37036

BRUNING 44-131 36925



SIDE VIEW

Scale: 1/4"=1'0"



TYPICAL FOOTING

Scale: 3/8"=1'-0"

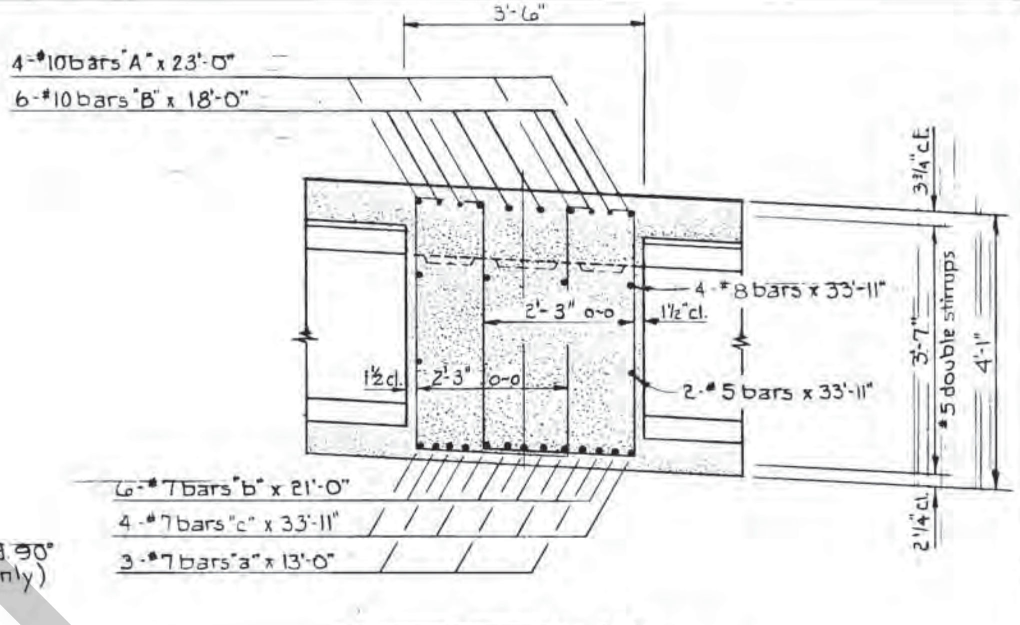
ELEVATION

Scale: 3/8"=1'-0"

Req'd Bearing Pressures:
Left ftg. 6.0 Ton/ft.²
Right ftg. 5.0 Ton/ft.²

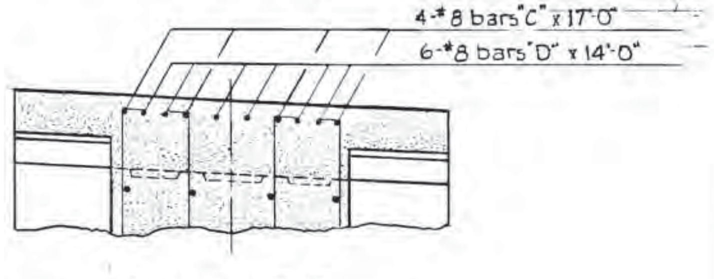
Column Steel (ea. column)
16-#8 vertical column bars with std. 90° hooks at footing ends. Stop bars 6" from bottom of deck.

#4 hoops @ 4" or 12" ctrs. as shown.
#4 ties @ 12" ctrs.



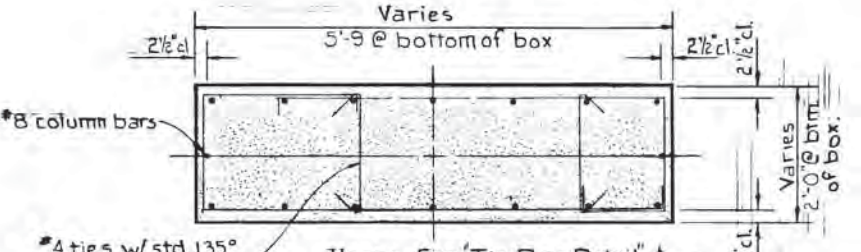
SECTION A-A

Scale: 3/4"=1'-0"



SECTION B-B

Scale: 3/4"=1'-0"



NOTE: See "Tie Bar Detail" & "Column Hoop Layout" on dwg.# 37026.

TYPICAL COLUMN SECTION

Scale: 3/4"=1'-0"

APPROVED: Walter J Hart
STRUCTURAL DESIGN ENGINEER
DESIGNED W.M. Thompson
DRAWN J. Silbernagel
CHECKED Page
REVIEWED J.M. Tisdall 7/6/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

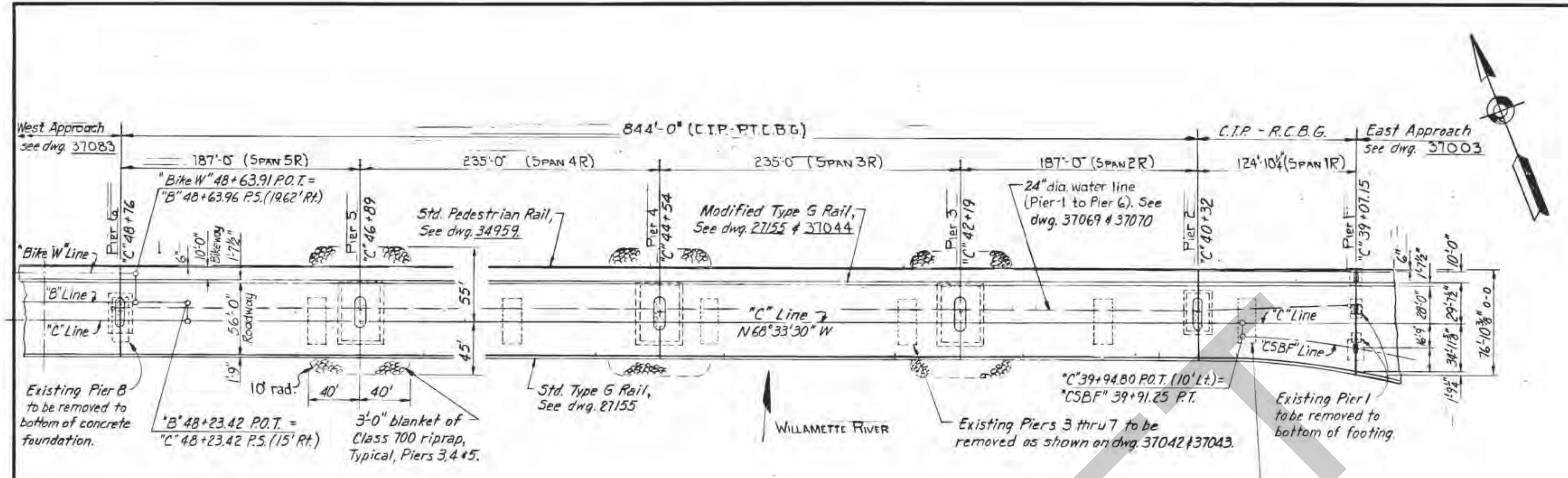
WILLAMETTE RIVER
(CENTER STREET) BRIDGE

ELEVATION - BENT "CF"-2

DATE May, 1982
BRIDGE NO. 123K
SHEET 40 OF 155
DRAWING NO. 37040

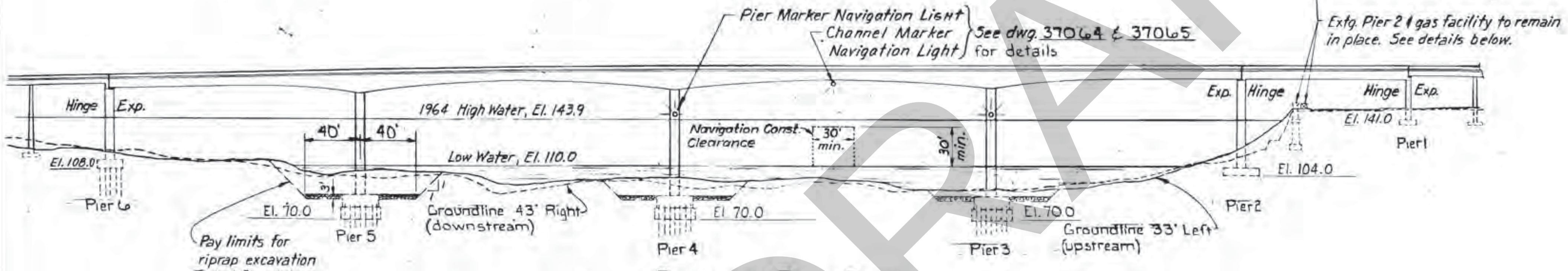
HYDRAULIC DATA

		Design Flood	Intermediate Regional Flood	Max. Regulated Flood Year (1964)
Discharge -	cfs	244,000	279,000	308,000
	m³/s			
Frequency - Years		50	100	100+
HW. Elevation Natural Channel	ft.	140.4	142.8	143.9
	m			



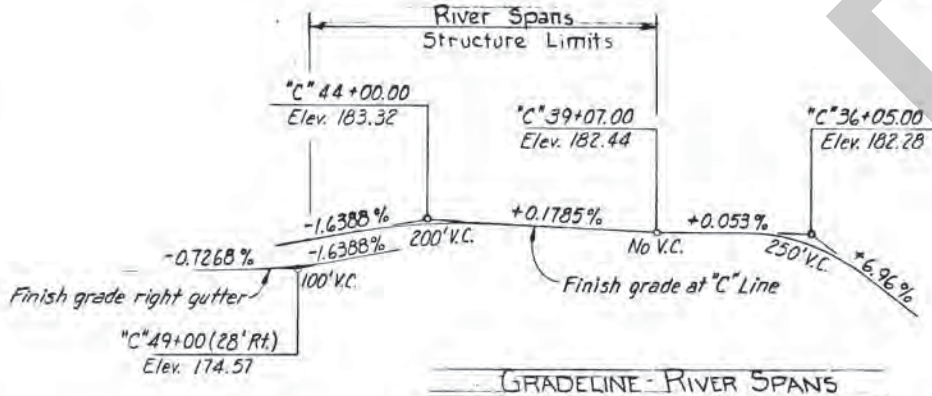
PLAN RIVER SPANS

Scale: 1"=50'



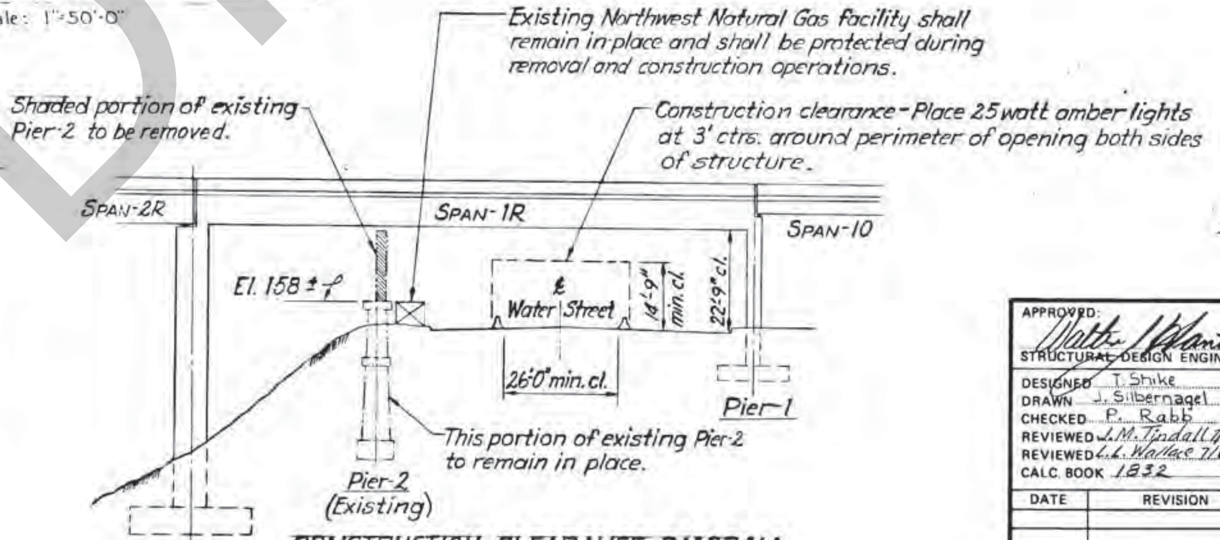
ELEVATION - RIVER SPANS

Scale: 1"=50'-0"



GRADELINE - RIVER SPANS

No Scale



CONSTRUCTION CLEARANCE DIAGRAM AND PARTIAL REMOVAL OF PIER-2

APPROVED: Walter J. Hart P.E.
STRUCTURAL DESIGN ENGINEER
DESIGNED T. Shike
DRAWN J. Silbernagel
CHECKED P. Rabb
REVIEWED J.M. Tindall 7/2/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

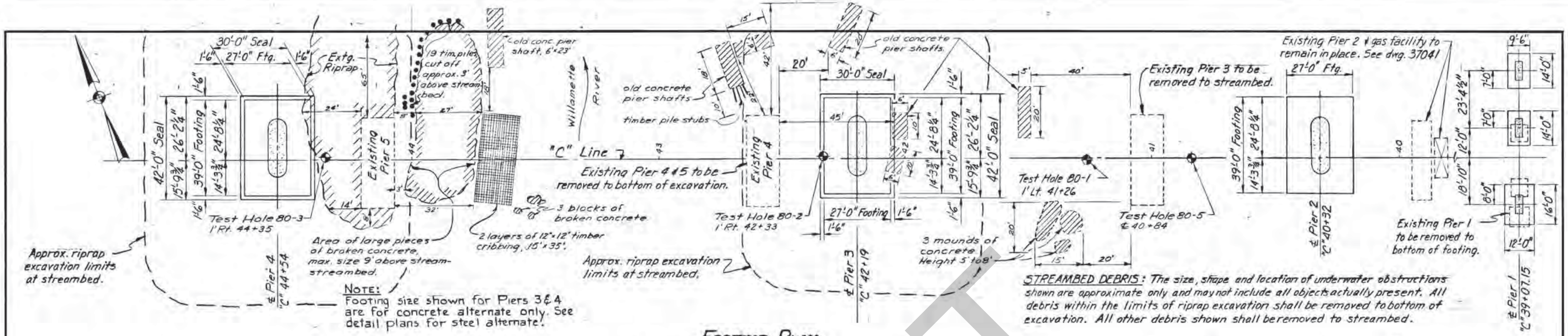
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

PLAN AND ELEVATION - RIVER SPANS

DATE May, 1982
BRIDGE NO. 123K
SHEET 41 OF 155
DRAWING NO. 37041

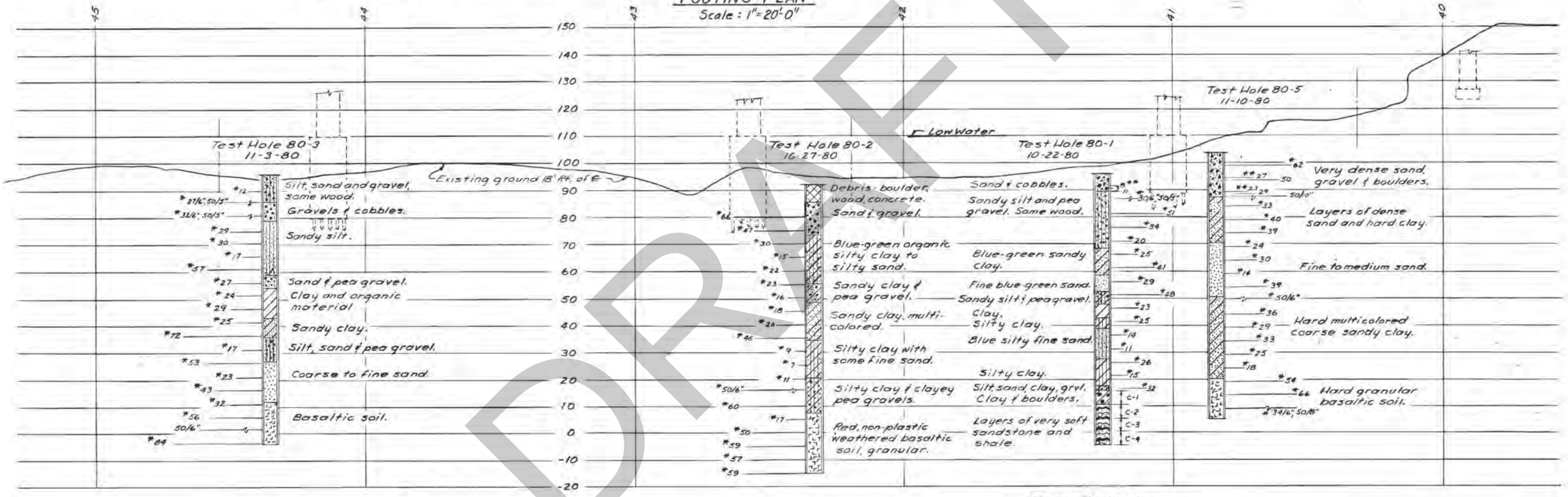


NOTE: Footing size shown for Piers 3 & 4 are for concrete alternate only. See detail plans for steel alternate.

STREAMBED DEBRIS: The size, shape and location of underwater obstructions shown are approximate only and may not include all objects actually present. All debris within the limits of riprap excavation shall be removed to bottom of excavation. All other debris shown shall be removed to streambed.

FOOTING PLAN

Scale: 1" = 20'-0"



ELEVATION

scale: 1" = 20'

Core	Recovery
C-1	80%
C-2	20%
C-3	17%
C-4	0

Elevations of bottom of existing piers shown are approximate only.

Foundation data shown on this drawing, in some instances, is a consolidation of and a revision in terminology from the original field drilling logs. The original field drilling logs are available for review in the office of the Bridge Engineer in Salem.

LEGEND OF MATERIALS

①	Gravels, cobbles and boulders.
②	Sand.
③	Silts.
④	Clays.
⑤	Silty gravels, gravel-silt mixture.
⑥	Silty sand, sand-silt mixture.
⑦	Clayey gravel, gravel-clay mixture.
⑧	Clayey sand, sand-clay mixture.
⑨	Silty clay.
⑩	Sand-silt-clay mixture, near equal proportions.
⑪	Sandstone.
⑫	Shale.
⑬	Basalt rock, broken, solid, weathered, lava.
⑭	

APPROVED: Walter J Hart
BRIDGE ENGINEER
DESIGNED
DRAWN JD
CHECKED
REVIEWED
REVIEWED
CALC. BOOK

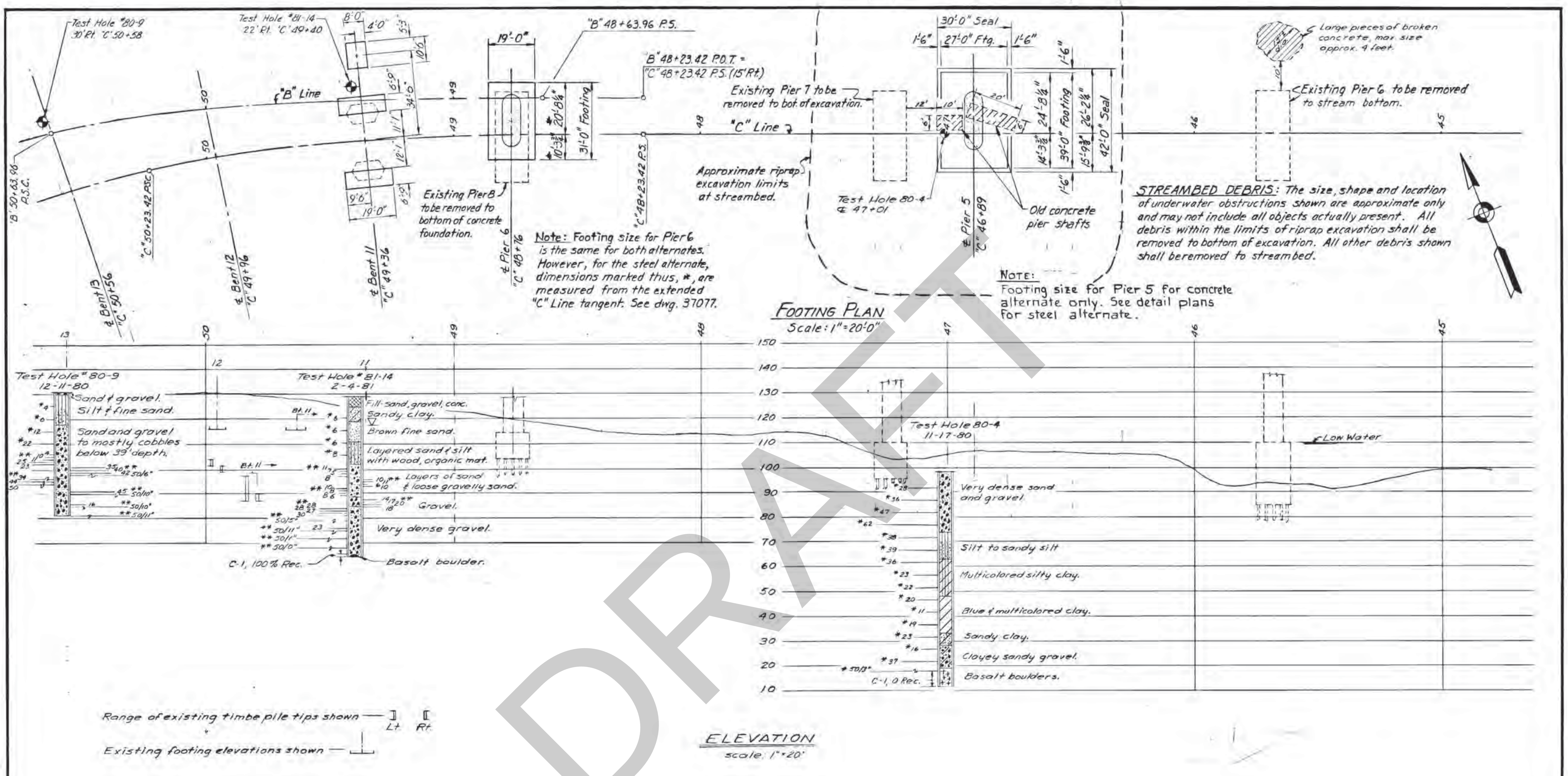
DATE	REVISION

OREGON STATE HIGHWAY DIVISION
BRIDGE SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

FOUNDATION DATA

DATE 12-4-80
BRIDGE NO. 123K
SHEET 42 OF 155
DRAWING NO. 37042



Note: Footing size for Pier 6 is the same for both alternates. However, for the steel alternate, dimensions marked thus, *, are measured from the extended "C" Line tangent. See dwg. 37077.

STREAMBED DEBRIS: The size, shape and location of underwater obstructions shown are approximate only and may not include all objects actually present. All debris within the limits of riprap excavation shall be removed to bottom of excavation. All other debris shown shall be removed to streambed.

NOTE: Footing size for Pier 5 for concrete alternate only. See detail plans for steel alternate.

FOOTING PLAN

Scale: 1"=20'-0"

Range of existing timbe pile tips shown — Lt. Rt.

Existing footing elevations shown —

ELEVATION

scale: 1"=20'

LEGEND OF MATERIALS

▒	Gravels, cobbles and boulders	▥	Silts.	▦	Silty gravels, gravel-silt mixture.	▨	Clayey gravel, gravel-clay mixture.	▨	Silty clay.	▤	Shale	▒	Basalt rock, broken, solid, weathered, lava.
░	Sand.	▨	Clays.	▥	Silty sand, sand-silt mixture.	▨	Clayey sand, sand-clay mixture.	▦	Sand-silt-clay mixture, near equal proportions.	▦	Sandstone.	☐	

∇ Elevation ground water encountered.
* Standard Penetration test.
** Oregon Miniature Pile test.
RQD = Rock Quality Designation
U = Undisturbed Sample
C = Core Sample

Foundation data shown on this drawing, in some instances, is a consolidation of and a revision in terminology from the original field drilling logs. The original field drilling logs are available for review through the office of the Structural Design Engineer in Salem.

APPROVED: Walter J Hant
STRUCTURAL DESIGN ENGINEER
DESIGNED
DRAWN JD
CHECKED
REVIEWED
REVIEWED
CALC. BOOK

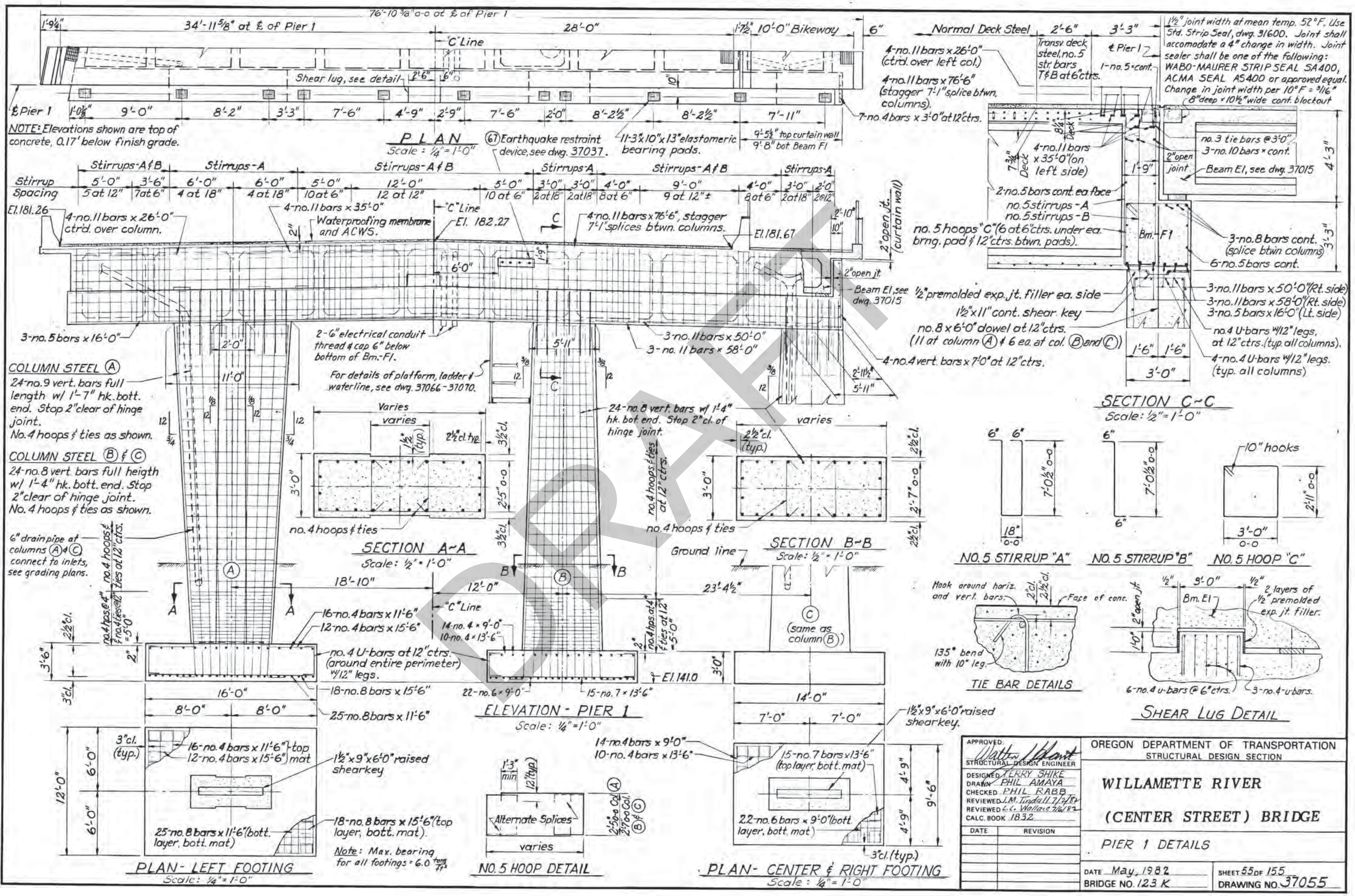
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

FOUNDATION DATA

DATE Jan. 1981
BRIDGE NO. 123K
SHEET 43 OF 155
DRAWING NO. 37043



APPROVED: Walter J. Hart
STRUCTURAL DESIGN ENGINEER
DESIGNED TERRY SHIKE
DRAWN PHIL AMAYA
CHECKED PHIL RABB
REVIEWED J.M. Tindall 7/2/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

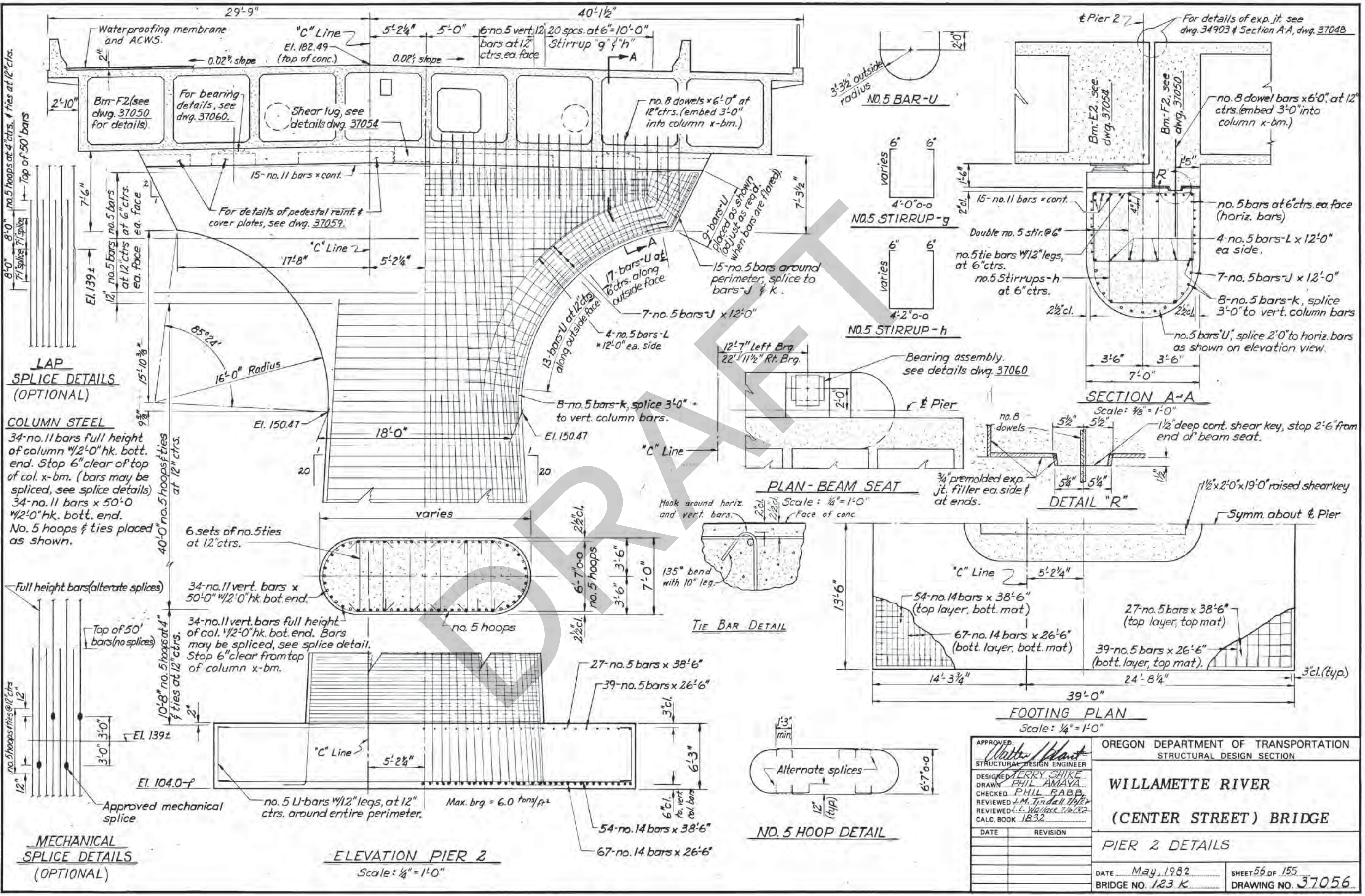
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

PIER 1 DETAILS

DATE May, 1982
BRIDGE NO. 123 K
SHEET 55 OF 155
DRAWING NO. 37055



LAP SPLICE DETAILS (OPTIONAL)

COLUMN STEEL
34-no. 11 bars full height of column w/2'-0" hk. bott. end. Stop 6" clear of top of col. x-bm. (bars may be spliced, see splice details)
34-no. 11 bars × 50'-0 w/2'-0" hk. bott. end.
No. 5 hoops & ties placed as shown.

NO. 5 BAR-U

NO. 5 STIRRUP-g

NO. 5 STIRRUP-h

SECTION A-A
Scale: ⅜" = 1'-0"

PLAN-BEAM SEAT
Scale: ¼" = 1'-0"

DETAIL "R"

TIE BAR DETAIL

FOOTING PLAN
Scale: ¼" = 1'-0"

MECHANICAL SPLICE DETAILS (OPTIONAL)

ELEVATION PIER 2
Scale: ¼" = 1'-0"

NO. 5 HOOP DETAIL

APPROVED:
STRUCTURAL DESIGN ENGINEER
DESIGNED TERRY SHIKE
DRAWN PHIL AMAYA
CHECKED PHIL RABB
REVIEWED J.M. Tindall 7/1/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

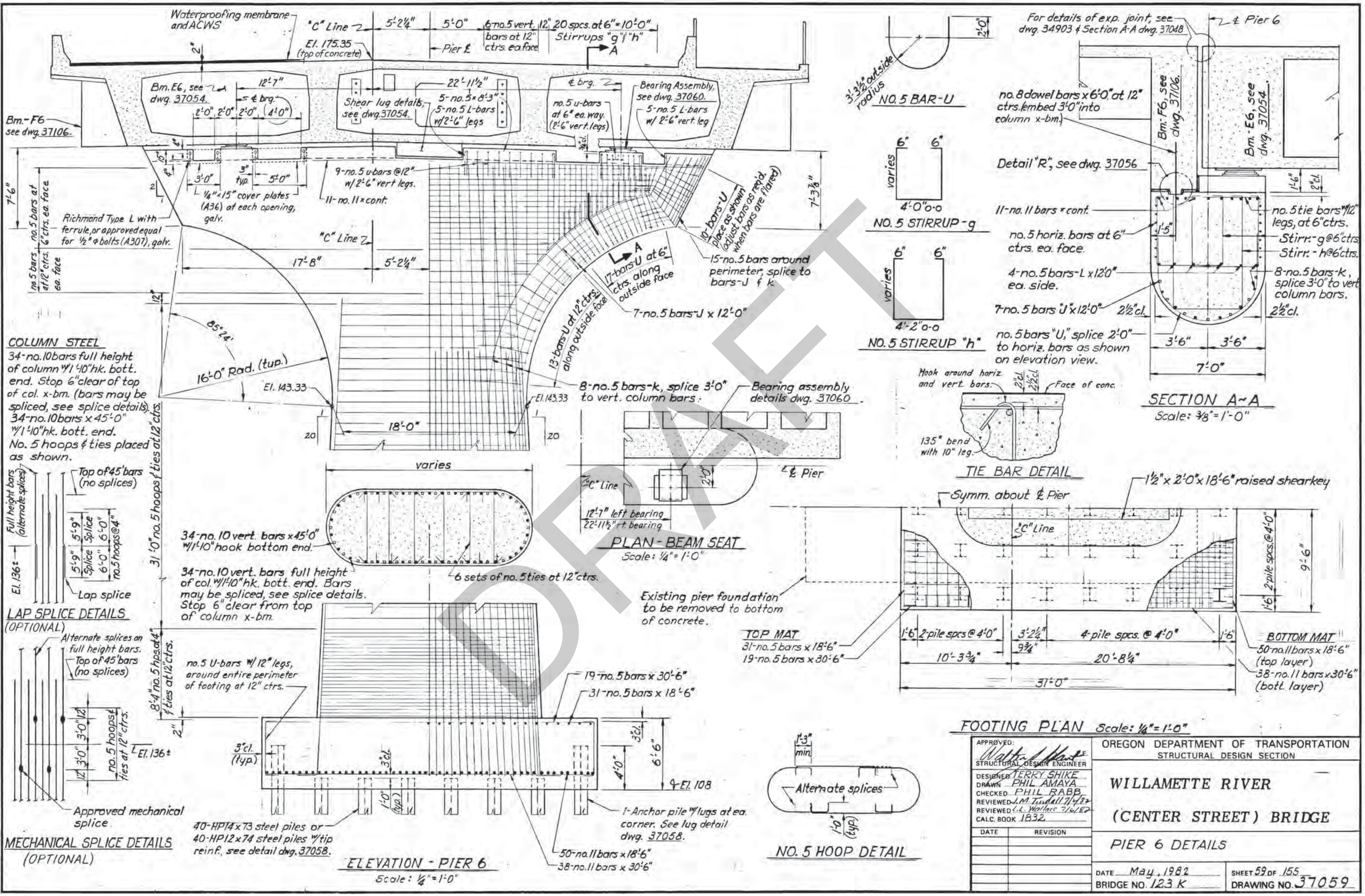
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

PIER 2 DETAILS

DATE May, 1982
SHEET 56 OF 155
BRIDGE NO. 123 K
DRAWING NO. 37056



APPROVED: Walter ... Hart P.E.
STRUCTURAL DESIGN ENGINEER
DESIGNED TERRY SHIKE
DRAWN PHIL AMAYA
CHECKED PHIL RABB
REVIEWED J.M. Tindall 7/1/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

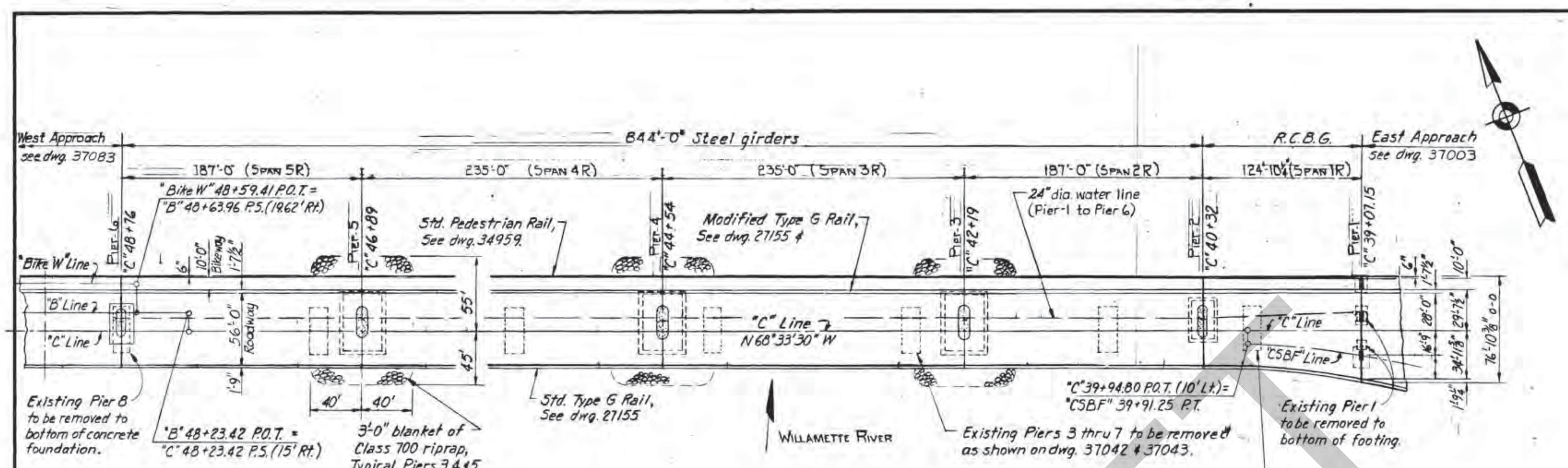
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

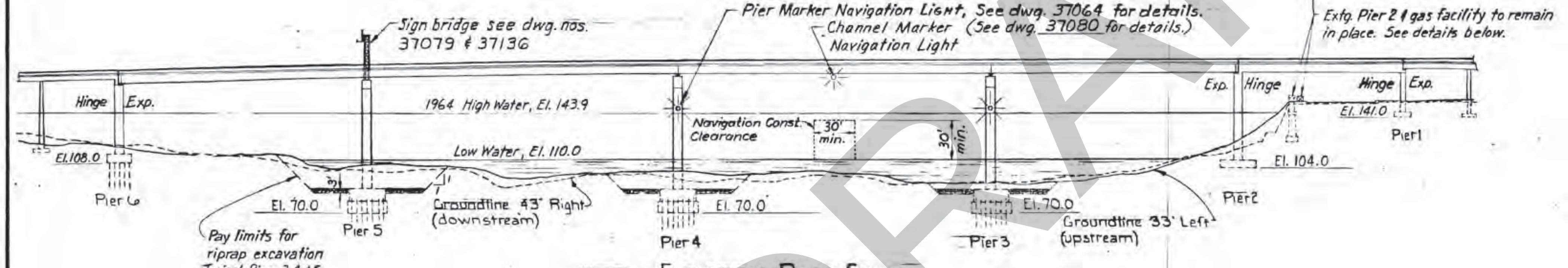
PIER 6 DETAILS

DATE May, 1982
BRIDGE NO. 123 K
SHEET 59 OF 155
DRAWING NO. 37059



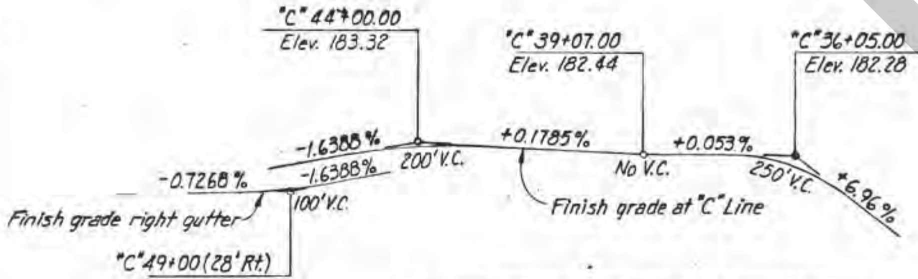
PLAN RIVER SPANS

Scale: 1"=50'



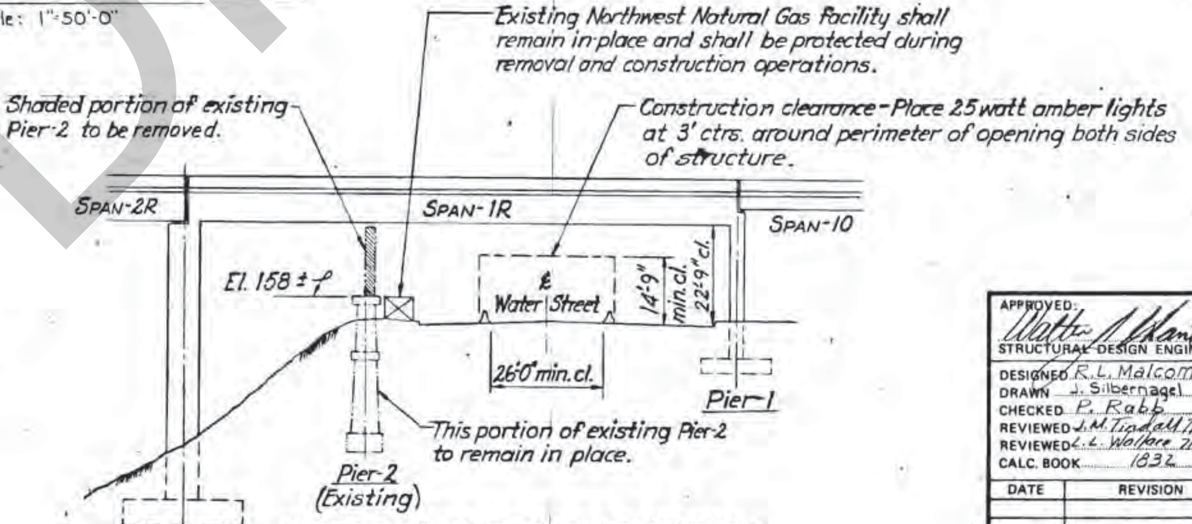
ELEVATION - RIVER SPANS

Scale: 1"=50'-0"



GRADELINE - RIVER SPANS

No Scale



CONSTRUCTION CLEARANCE DIAGRAM AND PARTIAL REMOVAL OF PIER-2

HYDRAULIC DATA

		Design Flood	Intermediate Regional Flood	Max. Regulated Flood Year (1964)
Discharge -	cfs	244,000	279,000	308,000
	m³/s			
Frequency - Years		50	100	100+
H.W. Elevation Natural Channel	ft.	140.4	142.8	143.9
	m			

APPROVED: Walter [illegible] P.E.
STRUCTURAL DESIGN ENGINEER
DESIGNED R. L. Malcom
DRAWN J. Silbernagel
CHECKED P. Rabb
REVIEWED J. M. Tindall 7/2/82
REVIEWED L. L. Wallace 7/8/82
CALC. BOOK 1832

DATE	REVISION

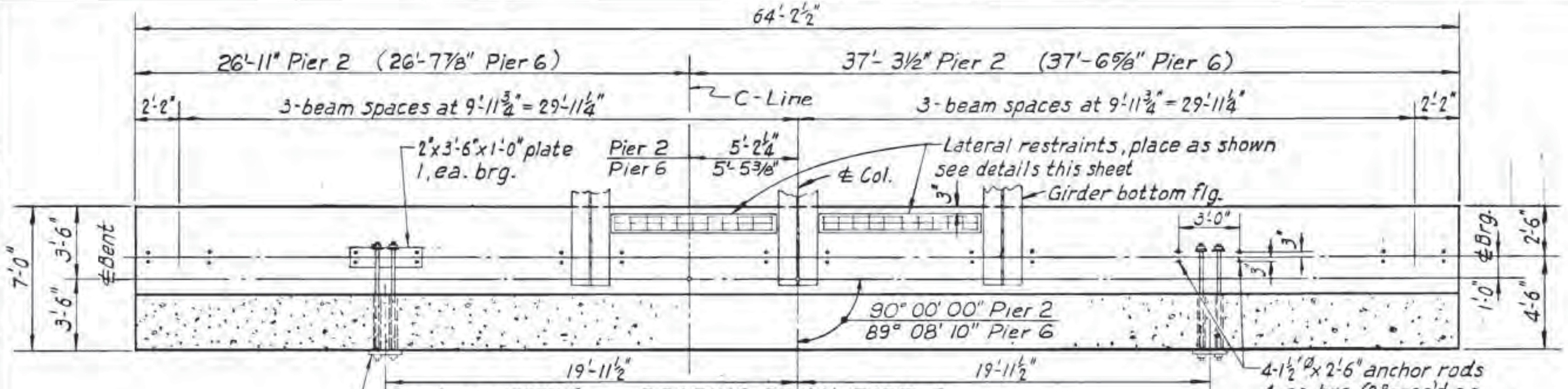
OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

STEEL ALTERNATE

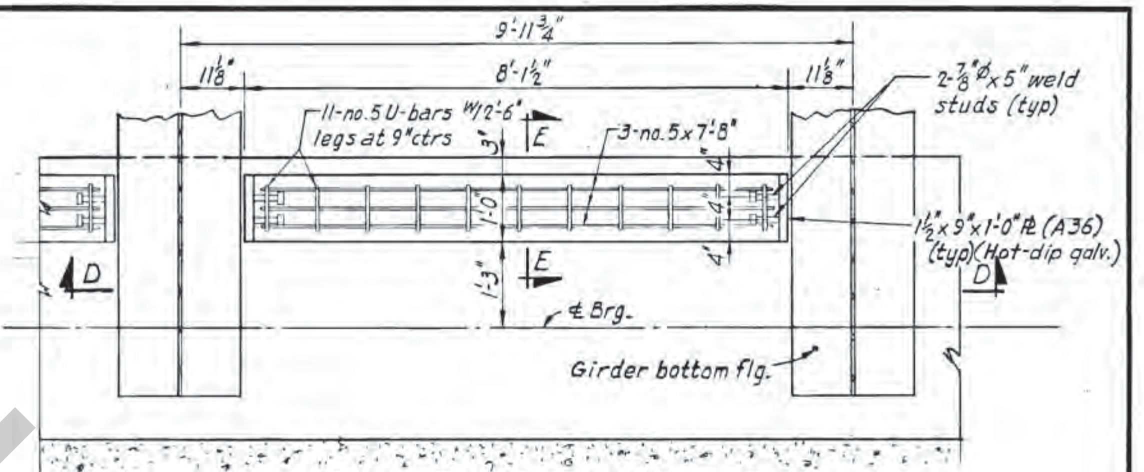
PLAN AND ELEVATION

DATE MAY, 1982
BRIDGE NO. 123K
SHEET 71 OF 155
DRAWING NO. 37071



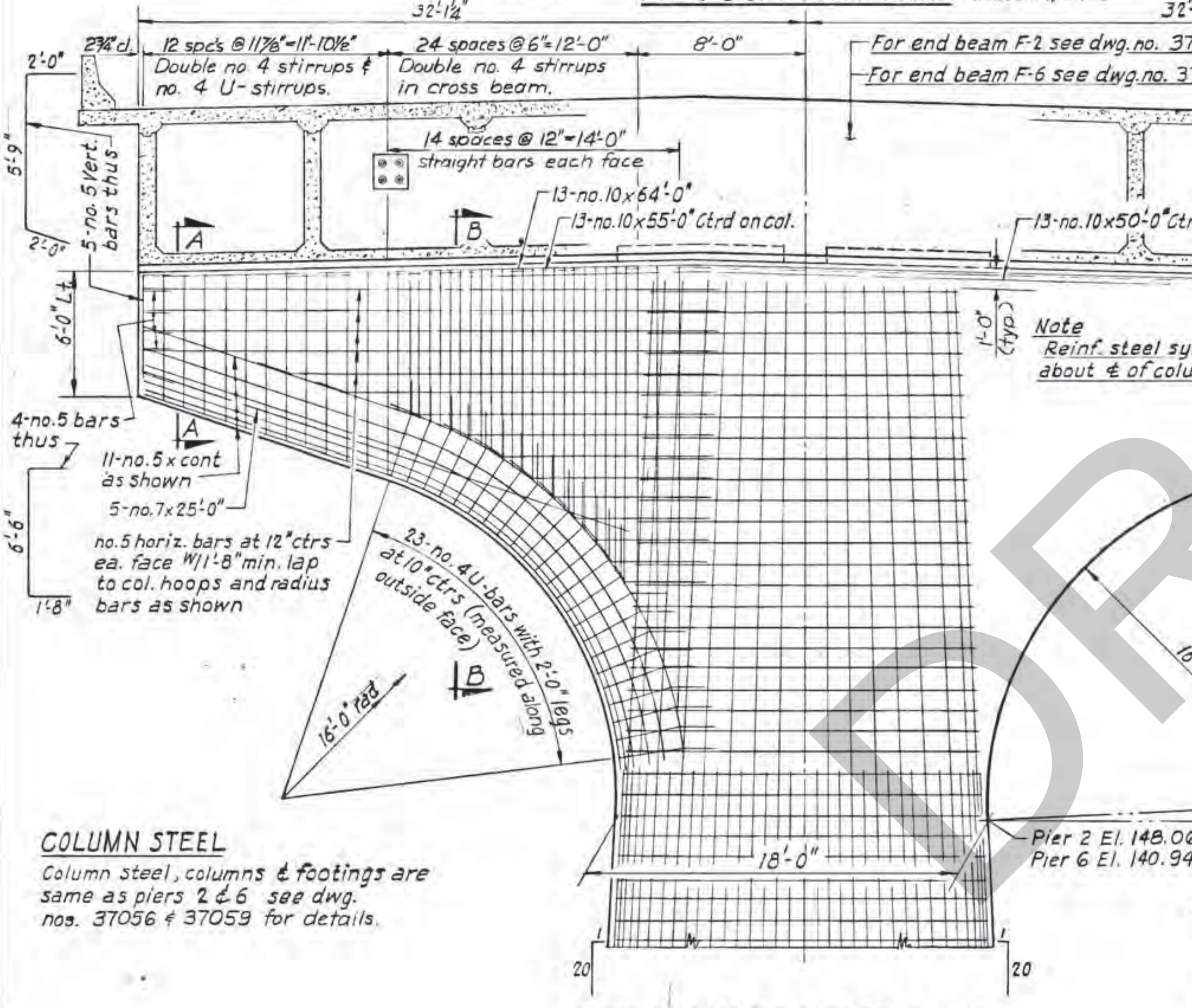
BEARING PLAN PIER-2
PIER-6 OPPOSITE HAND

Scale: ¼" = 1'-0"



PLAN AT LATERAL RESTRAINT
PIERS 2 & 6

Scale: ¾" = 1'-0"



COLUMN STEEL

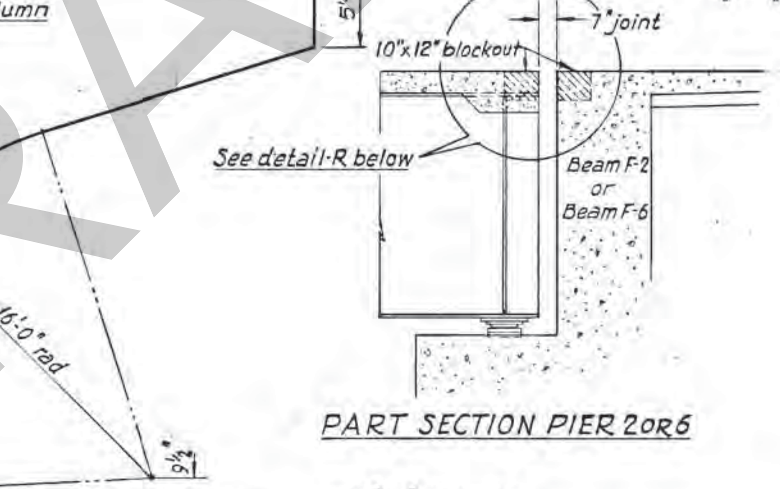
Column steel, columns & footings are same as piers 2 & 6 see dwg. nos. 37056 & 37059 for details.

PART ELEVATION PIER 2 OR 6

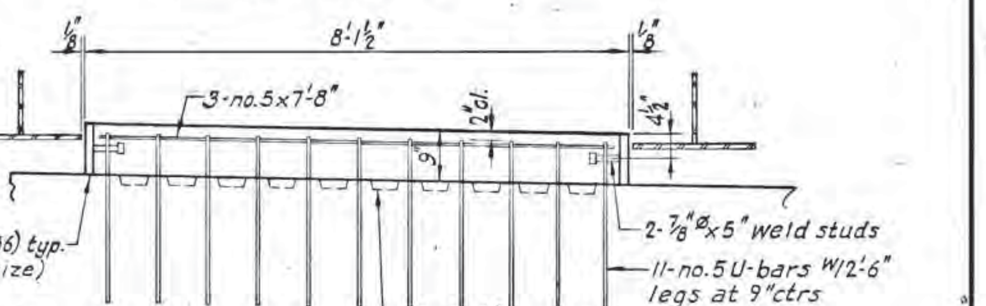
Scale: ¼" = 1'-0"

NOTE

Section AA & BB are the same as those shown on dwg. no. 37078 except note the difference in stirrup & bar sizes for piers 2 & 6

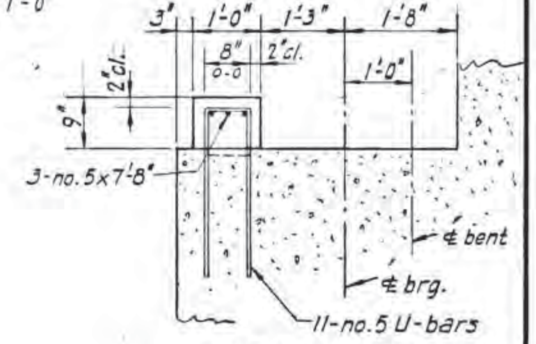


PART SECTION PIER 2 OR 6



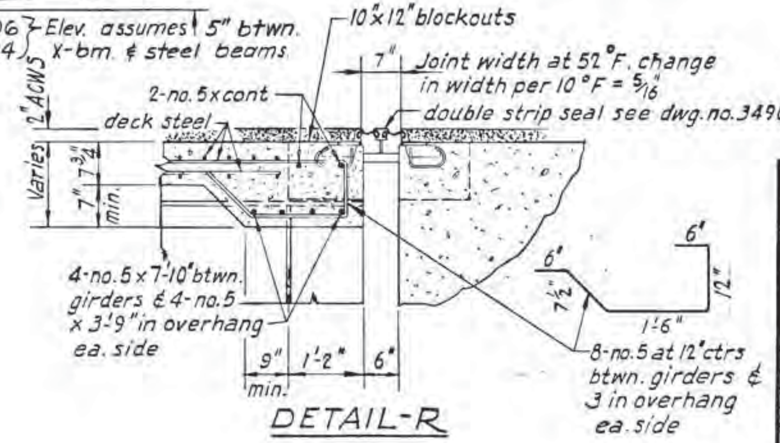
SECTION D-D

Scale: ¾" = 1'-0"



SECTION E-E

Scale: ¾" = 1'-0"



DETAIL-R

APPROVED: Walter J. Hart
BRIDGE ENGINEER
DESIGNED R. L. Malcom
DRAWN J. Hardin
CHECKED P. Rabb
REVIEWED J.M. Tindall 7/2/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

DATE	REVISION

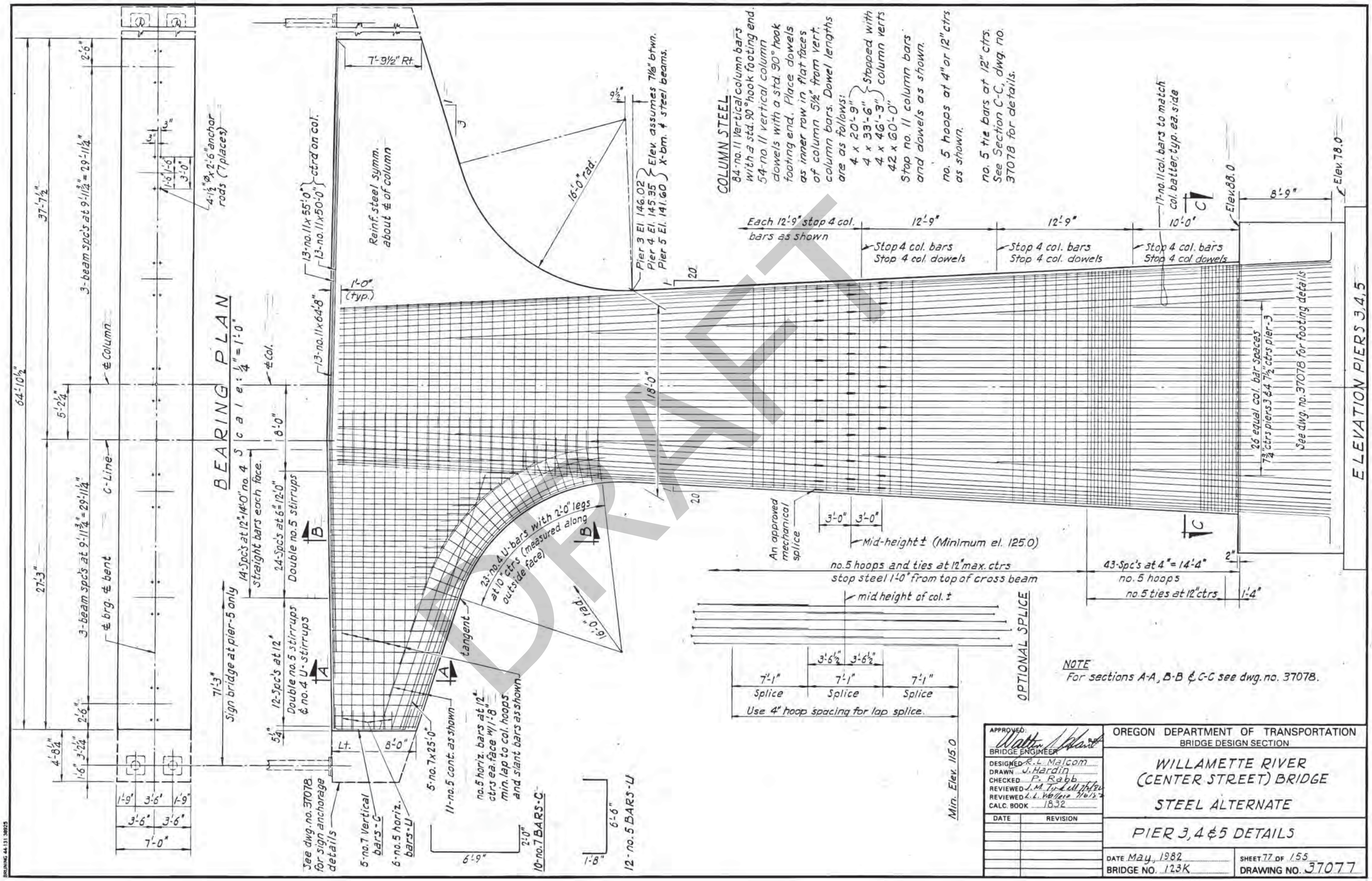
OREGON DEPARTMENT OF TRANSPORTATION
BRIDGE DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

STEEL ALTERNATE

PIER 2 OR 6 DETAILS

DATE May, 1982
BRIDGE NO. 123K
SHEET 76 OF 155
DRAWING NO. 37076



COLUMN STEEL

84-no. 11 Vertical column bars with a std. 90° hook footing end.

54-no. 11 vertical column dowels with a std. 90° hook footing end. Place dowels as inner row in flat faces of column 5½" from vert. column bars. Dowel lengths are as follows:

4 x 20'-9" } Stopped with
4 x 33'-6" } column verts
4 x 46'-3"
42 x 60'-0"

Stop no. 11 column bars and dowels as shown.

no. 5 hoops at 4" or 12" ctrs as shown.

no. 5 tie bars at 12" ctrs. See Section C-C, dwg. no. 37078 for details.

NOTE
For sections A-A, B-B & C-C see dwg. no. 37078.

APPROVED: Walter J. Hart
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DRAWN J. Hardin
CHECKED P. Rabb
REVIEWED J.M. 7/1/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

DATE	REVISION

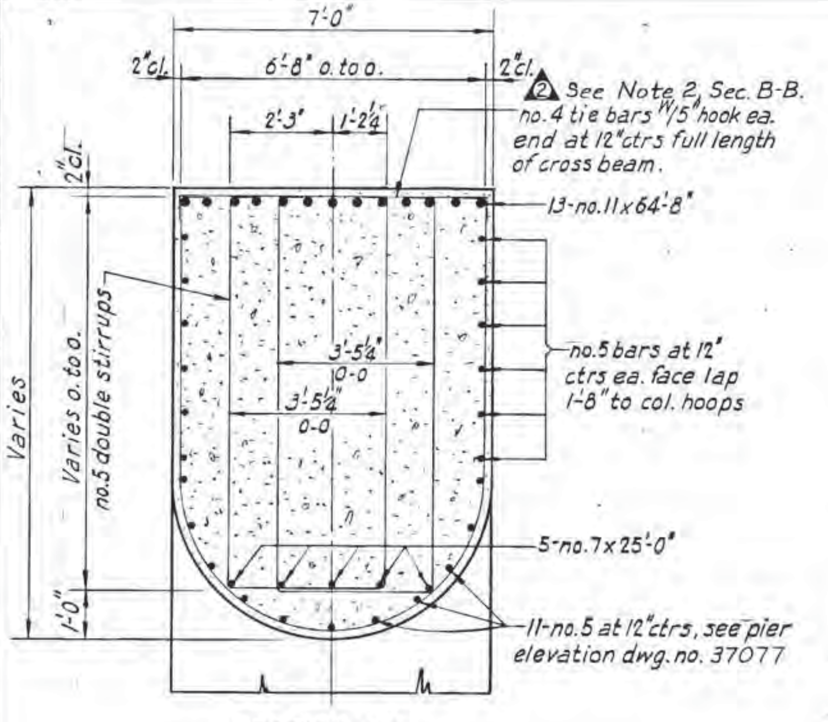
OREGON DEPARTMENT OF TRANSPORTATION
BRIDGE DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

STEEL ALTERNATE

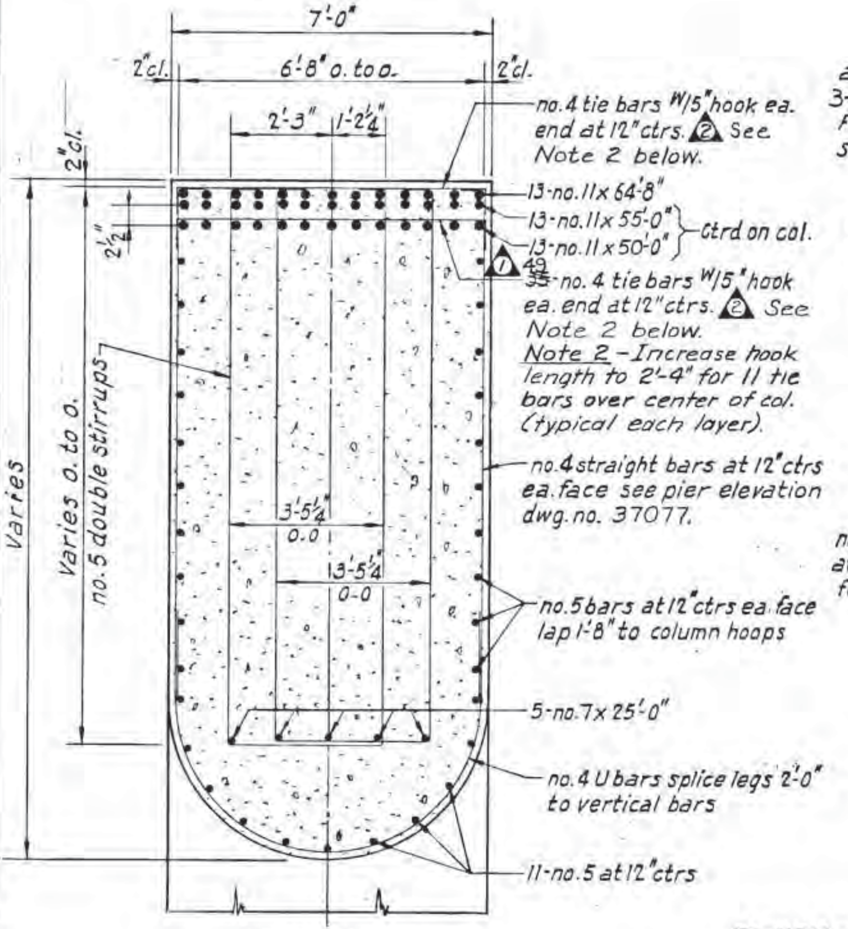
PIER 3,4 & 5 DETAILS

DATE May, 1982
SHEET 77 OF 155
BRIDGE NO. 123K
DRAWING NO. 37077



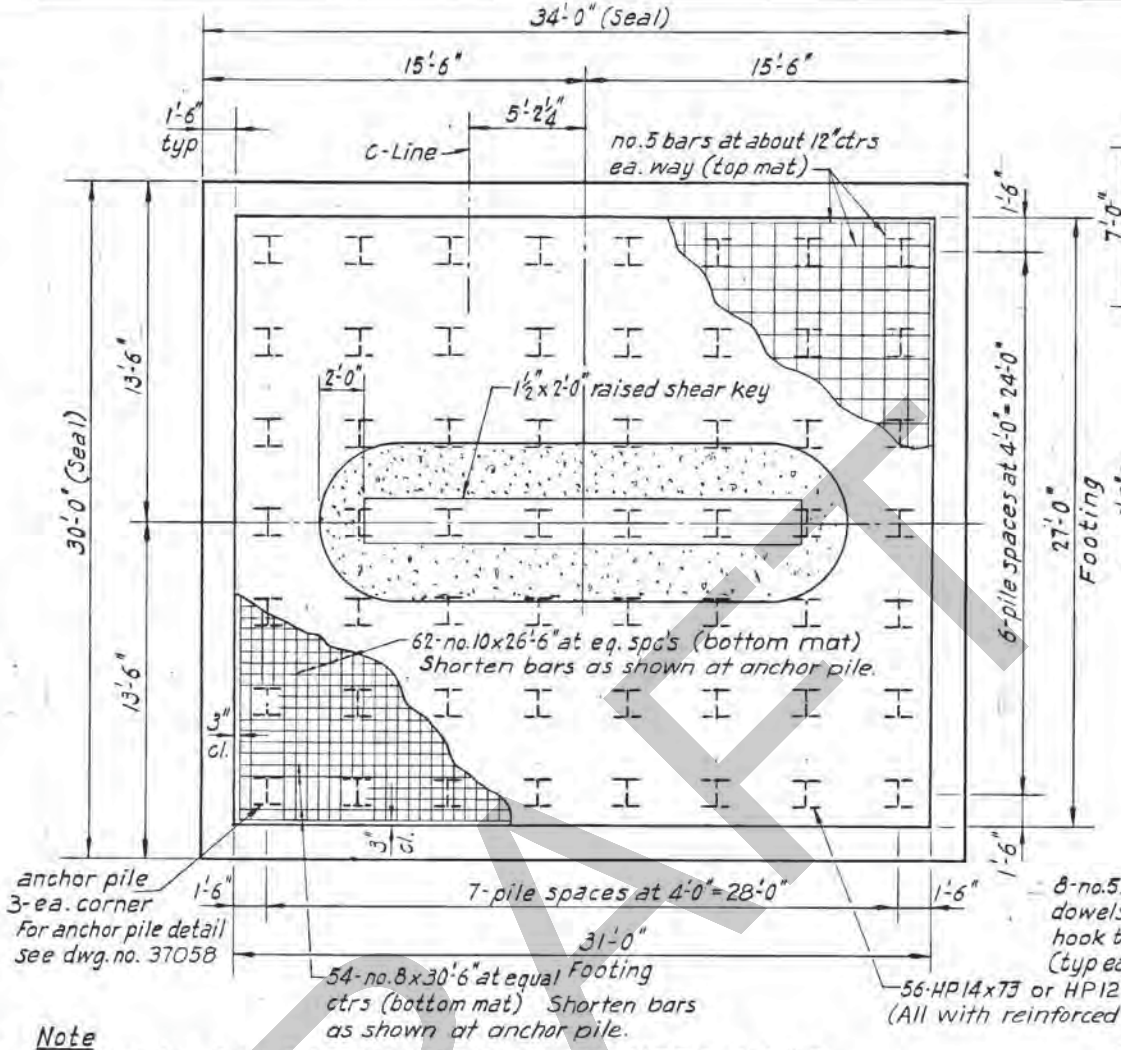
SECTION A-A

Scale: ½"=1'-0"



SECTION B-B

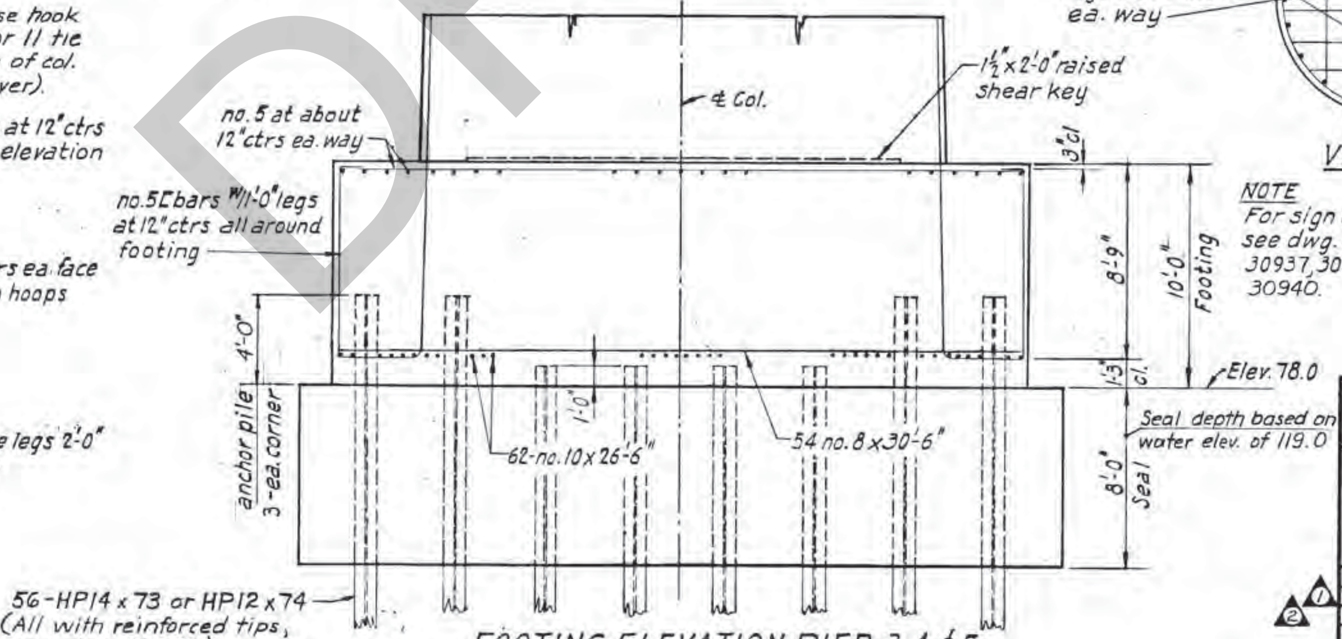
Scale: ½"=1'-0"



Note
For sections A-A, B-B & C-C locations, see dwg no. 37077

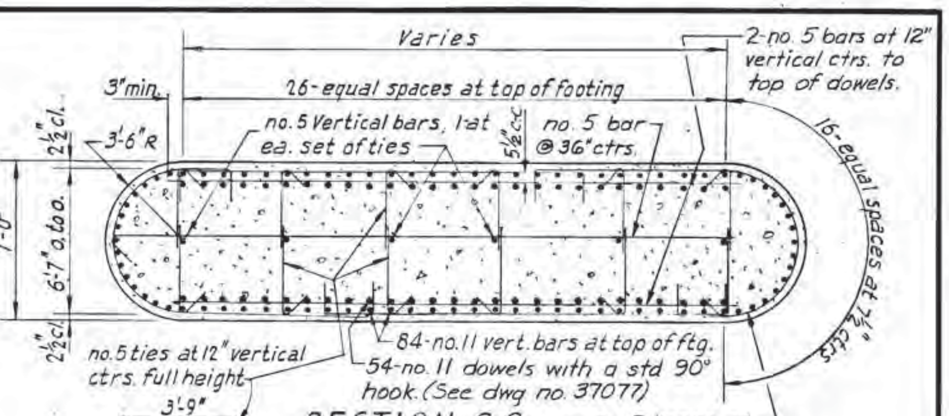
FOOTING PLAN PIERS 3,4 & 5

Scale: ¼"=1'-0"

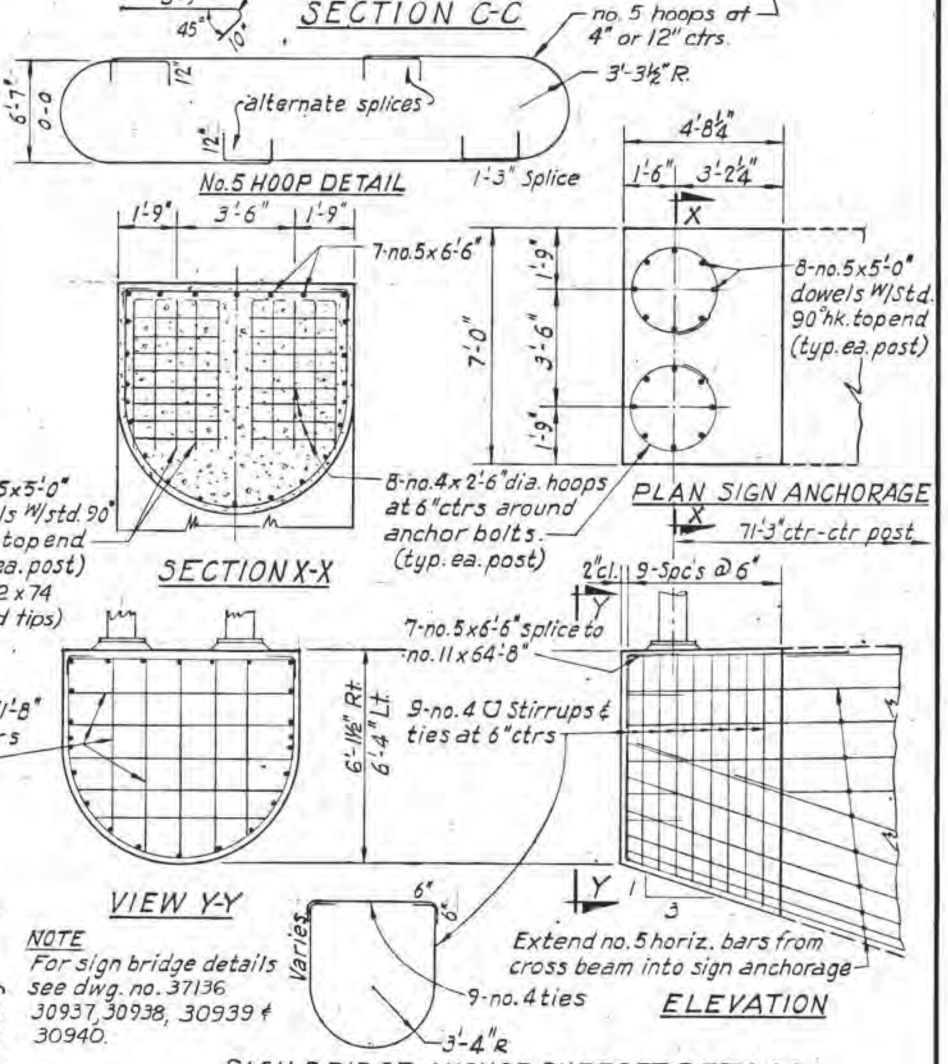


FOOTING ELEVATION PIER 3,4 & 5

Scale: ¼"=1'-0"



SECTION C-C



NOTE
For sign bridge details see dwg. no. 37136 30937, 30938, 30939 & 30940.

SIGN BRIDGE ANCHOR SUPPORT DETAILS

Scale: ⅜"=1'-0"

APPROVED: Walter J. Blant
BRIDGE ENGINEER
DESIGNED R.L. Malcom
DRAWN J. Hardin
CHECKED P. Rabb
REVIEWED J.M. Tindall 7/2/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

DATE	REVISION
① 7-21-83	Correct tie no.
② 7-21-83	Inc. tie length

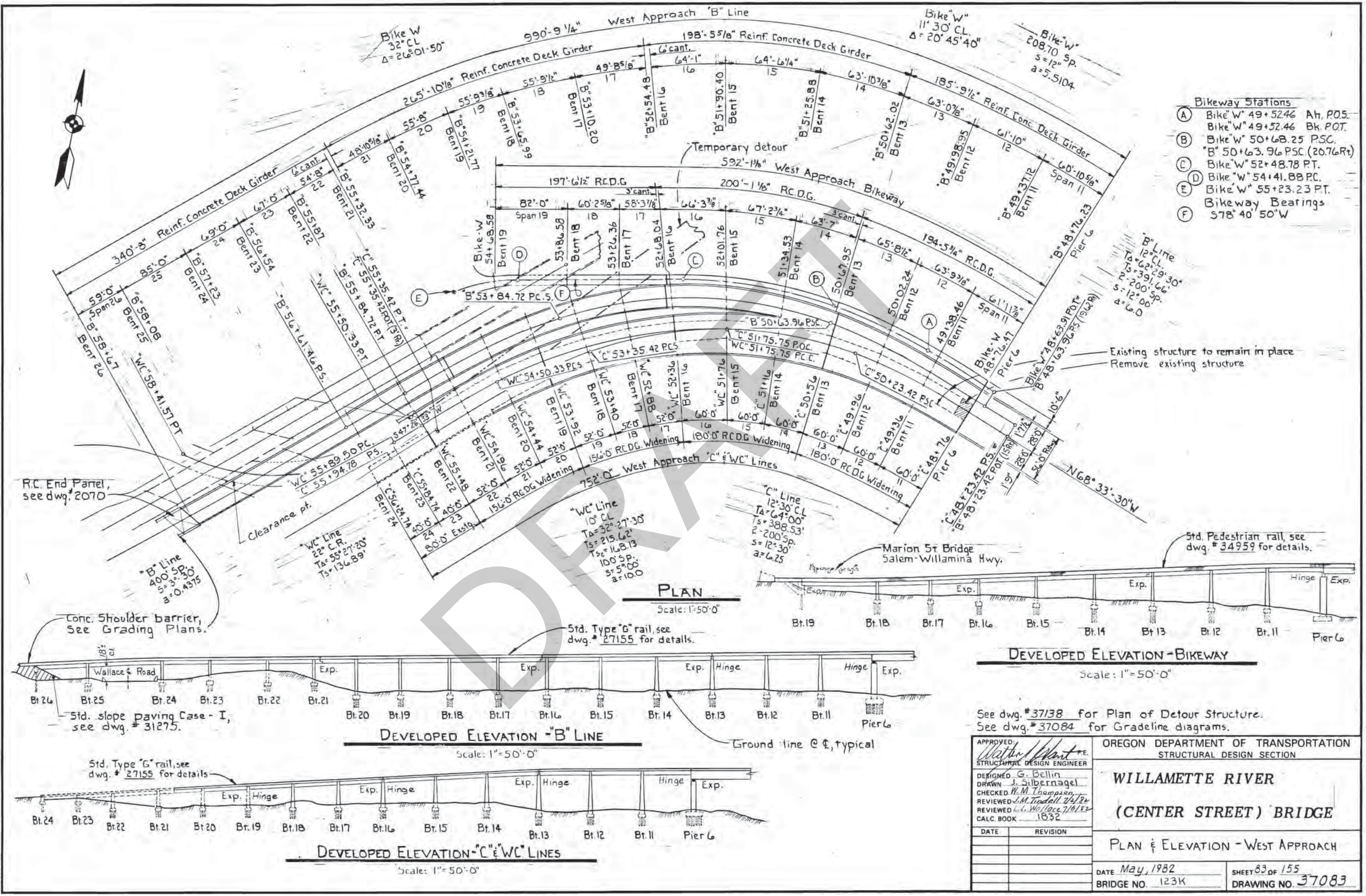
OREGON DEPARTMENT OF TRANSPORTATION
BRIDGE DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

STEEL ALTERNATE

PIER 3,4 & 5 DETAILS

DATE May, 1982
BRIDGE NO. 123K
SHEET 78 OF 155
DRAWING NO. 37078



PLAN

Scale: 1"=50'-0"

DEVELOPED ELEVATION - BIKEWAY

Scale: 1"=50'-0"

DEVELOPED ELEVATION - "B" LINE

Scale: 1"=50'-0"

DEVELOPED ELEVATION - "C" & "WC" LINES

Scale: 1"=50'-0"

Bikeway Stations

Ⓐ Bike "W" 49+52.46 Ah. P.O.S.
Bike "W" 49+52.46 Bk. P.O.T.
Ⓑ Bike "W" 50+68.25 P.S.C.
"B" 50+63.96 P.S.C. (20.76 Rt.)
Ⓒ Bike "W" 52+48.78 P.T.
Ⓓ Bike "W" 54+41.88 P.C.
Ⓔ Bike "W" 55+23.23 P.T.
Bikeway Bearings
Ⓕ S78° 40' 50"W

See dwg. #37138 for Plan of Detour Structure.
See dwg. #37084 for Gradeline diagrams.

APPROVED: Walter J. Hart P.E.
STRUCTURAL DESIGN ENGINEER
DESIGNED G. Bellin
DRAWN J. Silbernagel
CHECKED W. M. Thompson
REVIEWED J. M. Tindall 7/2/82
REVIEWED L. L. Wallace 7/6/82
CALC. BOOK 1832

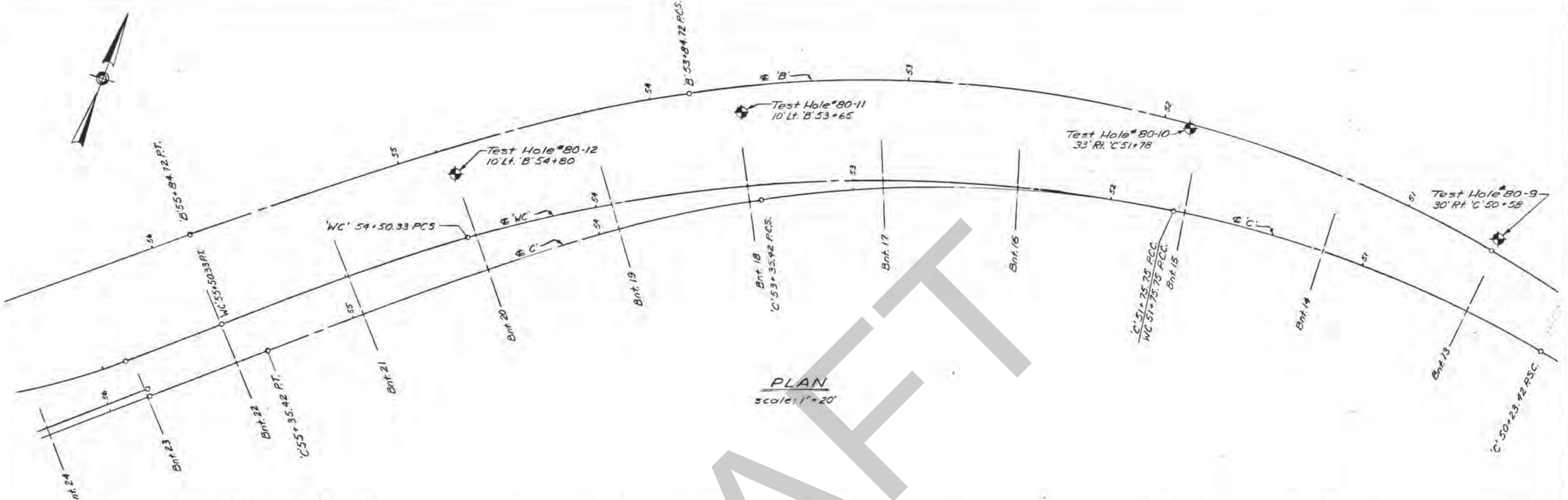
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

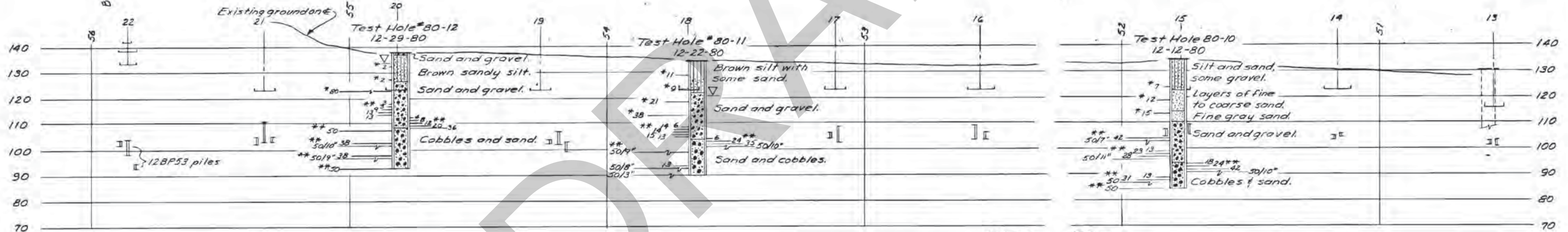
WILLAMETTE RIVER
(CENTER STREET) BRIDGE

PLAN & ELEVATION - WEST APPROACH

DATE May, 1982
BRIDGE NO. 123K
SHEET 83 OF 155
DRAWING NO. 37083



PLAN
scale: 1" = 20'



Range of existing timber pile tips shown — Lt. Cntr. Rt.

Existing footing elevations shown —

ELEVATION
scale: 1" = 20'

LEGEND OF MATERIALS

- ▨ Gravels, cobbles and boulders.
- ▨ Sand.
- ▨ Silts.
- ▨ Clays.
- ▨ Silty gravels, gravel-silt mixture.
- ▨ Silty sand, sand-silt mixture.
- ▨ Clayey gravel, gravel-clay mixture.
- ▨ Clayey sand, sand-clay mixture.
- ▨ Silty clay.
- ▨ Sand-silt-clay mixture, near equal proportions.
- ▨ Shale.
- ▨ Sandstone.
- ▨ Basalt rock, broken, solid, weathered, lava.
- ☐

∇ Elevation ground water encountered.
* Standard Penetration test.
** Oregon Miniature Pile test.
RQD = Rock Quality Designation
U = Undisturbed Sample
C = Core Sample

Foundation data shown on this drawing, in some instances, is a consolidation of and a revision in terminology from the original field drilling logs. The original field drilling logs are available for review through the office of the Structural Design Engineer in Salem.

APPROVED: Walter J. Blant
STRUCTURAL DESIGN ENGINEER
DESIGNED
DRAWN JD
CHECKED
REVIEWED
REVIEWED
CALC. BOOK

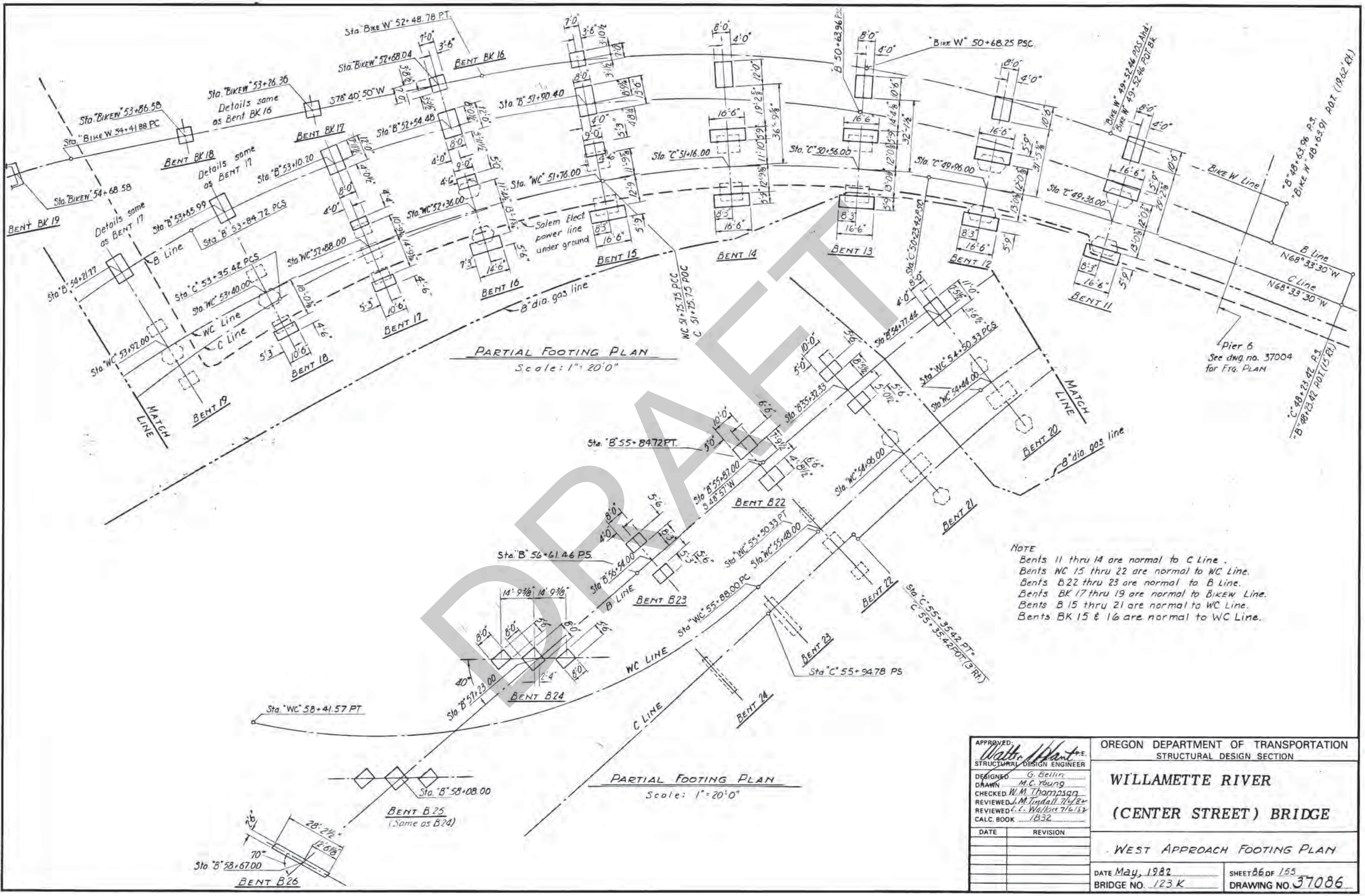
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

FOUNDATION DATA

DATE Jan. 1981
BRIDGE NO. 123K
SHEET 85 OF 155
DRAWING NO. 37085



NOTE
Bents 11 thru 14 are normal to C Line.
Bents WC 15 thru 22 are normal to WC Line.
Bents B22 thru 23 are normal to B Line.
Bents BK 17 thru 19 are normal to BIKEW Line.
Bents B 15 thru 21 are normal to WC Line.
Bents BK 15 & 16 are normal to WC Line.

APPROVED: Walter J. Hart P.E.
STRUCTURAL DESIGN ENGINEER
DESIGNED G. Bellin
DRAWN M.C. Young
CHECKED W.M. Thompson
REVIEWED J.M. Tindall 7/4/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

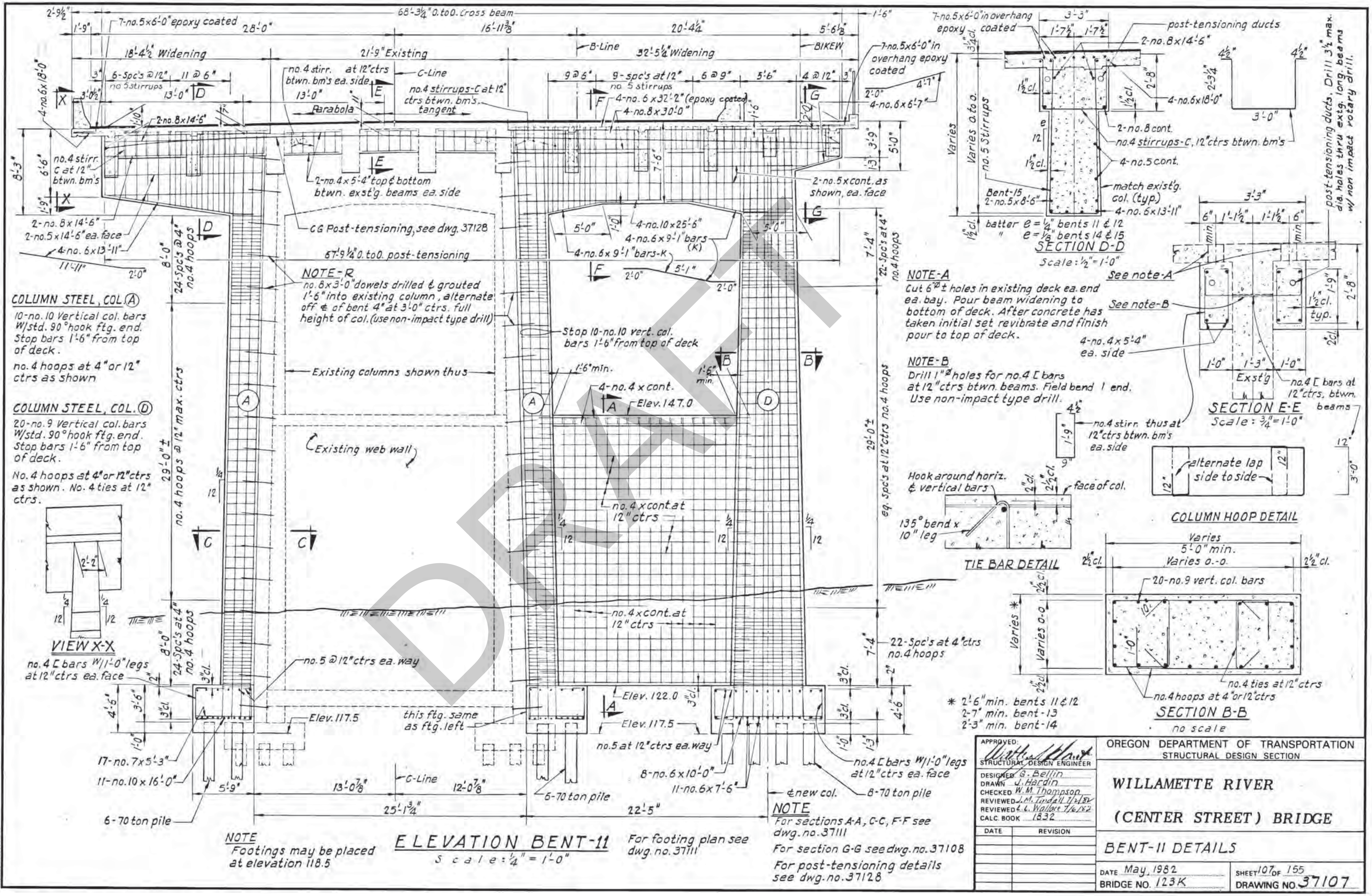
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

WEST APPROACH FOOTING PLAN

DATE May, 1982
BRIDGE NO. 123 K
SHEET 86 OF 155
DRAWING NO. 37086



COLUMN STEEL, COL. Ⓐ

10-no.10 Vertical col. bars W/std. 90° hook ftg. end. Stop bars 1'-6" from top of deck.

no.4 hoops at 4" or 12" ctrs as shown

COLUMN STEEL, COL. Ⓓ

20-no.9 Vertical col. bars W/std. 90° hook ftg. end. Stop bars 1'-6" from top of deck.

No.4 hoops at 4" or 12" ctrs as shown. No.4 ties at 12" ctrs.

NOTE-R

no.8x3'-0" dowels drilled & grouted 1'-6" into existing column, alternate off ℄ of bent 4" at 3'-0" ctrs. full height of col. (use non-impact type drill)

NOTE-A

Cut 6"∅± holes in existing deck ea. end ea. bay. Pour beam widening to bottom of deck. After concrete has taken initial set revibrate and finish pour to top of deck.

NOTE-B

Drill 1"∅ holes for no.4 C bars at 12" ctrs btwn. beams. Field bend 1 end. Use non-impact type drill.

* 2'-6" min. bents 11 & 12
2-7" min. bent-13
2'-3" min. bent-14

NOTE

Footings may be placed at elevation 118.5

ELEVATION BENT-11

Scale: ¼" = 1'-0"

For footing plan see dwg. no. 37111

NOTE

For sections A-A, C-C, F-F see dwg. no. 37111

For section G-G see dwg. no. 37108

For post-tensioning details see dwg. no. 37128

APPROVED: [illegible] P.E.
STRUCTURAL DESIGN ENGINEER
DESIGNED G. Bellin
DRAWN J. Hardin
CHECKED W.M. Thompson
REVIEWED J.M. Tindall 7/2/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

DATE	REVISION

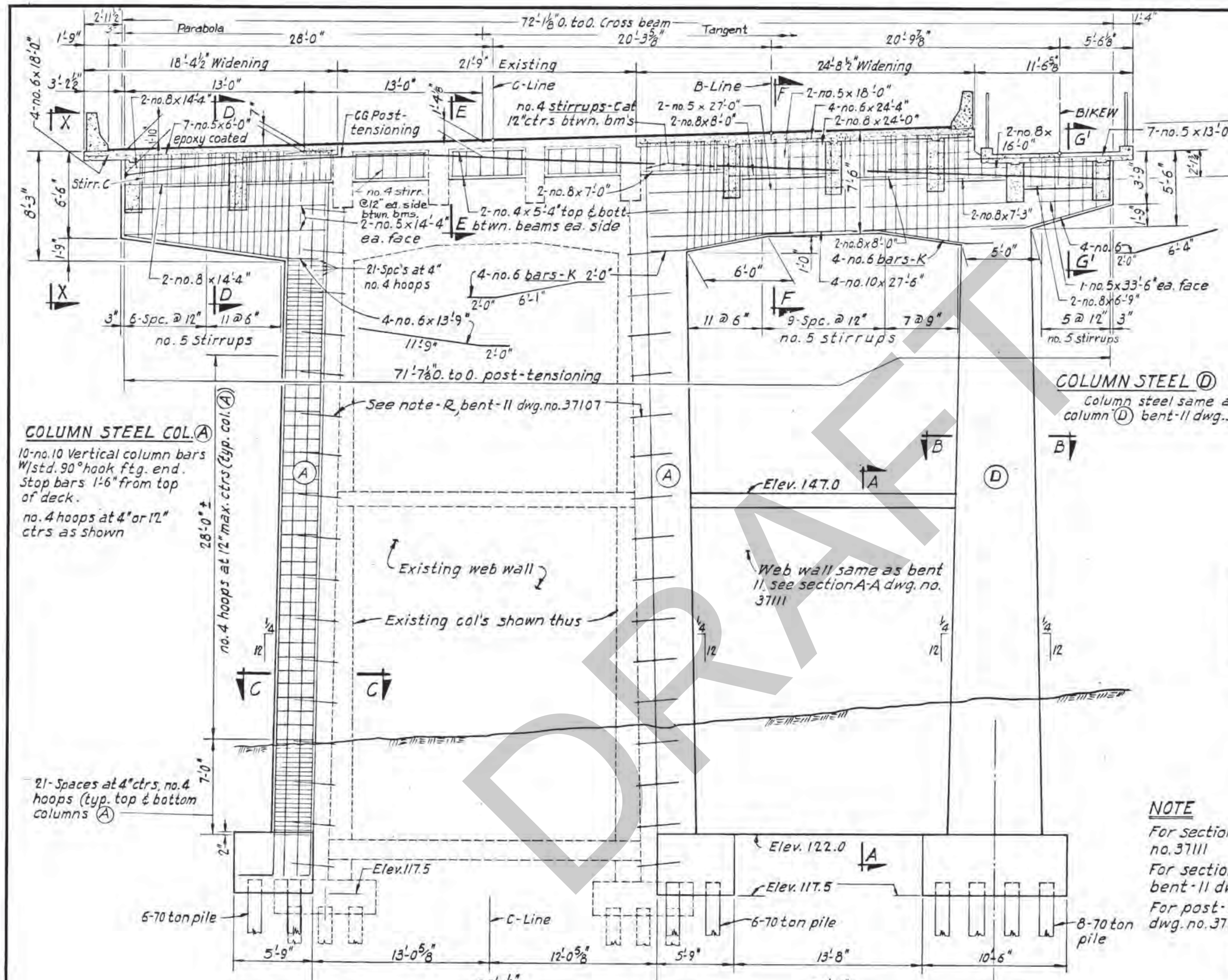
OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

BENT-11 DETAILS

DATE May, 1982
BRIDGE NO. 123K
SHEET 107 OF 155
DRAWING NO. 37107

BRUNING 44-131 38925



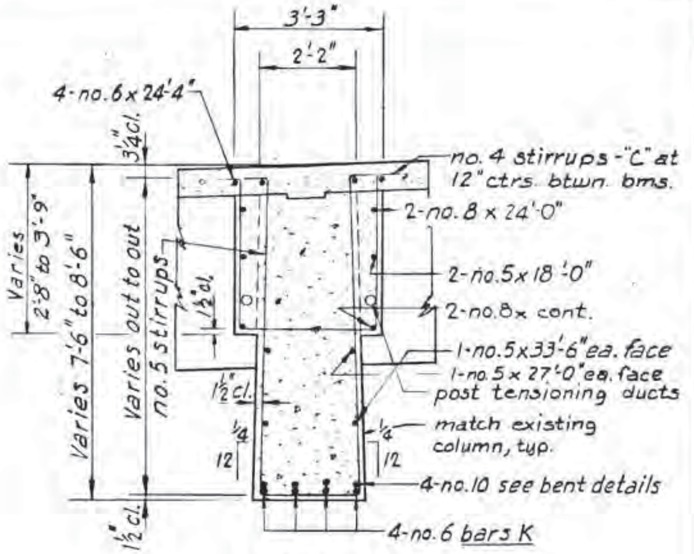
ELEVATION BENT-12

Scale: ¼" = 1'-0"

NOTE

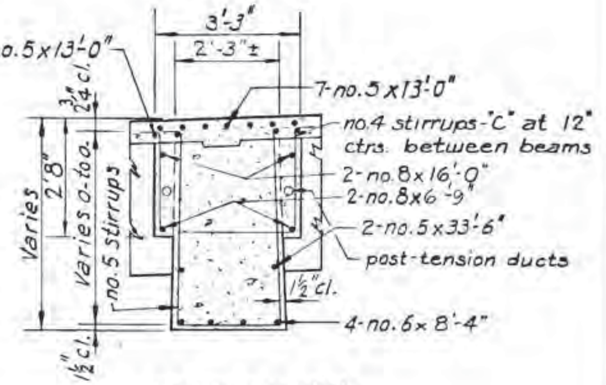
Footings, footing steel, piling for bent-12 are the same as bent-11 see dwg. no. 37107 & no. 37111

Footings may be placed at elevation 118.5



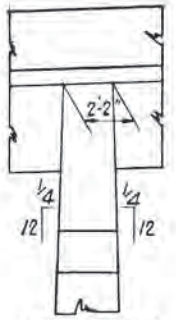
SECTION F-F

Scale: ½" = 1'-0"

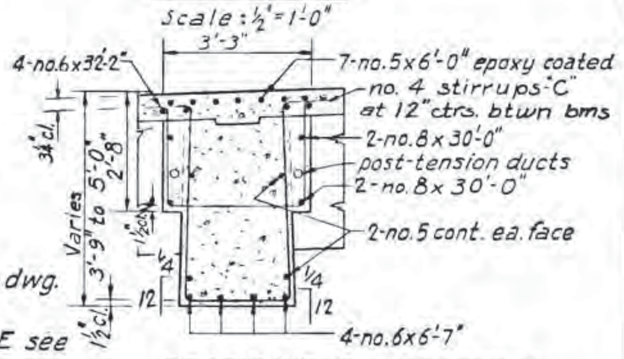


SECTION G'-G'

Scale: ½" = 1'-0"



VIEW X-X



SECTION G-G

Scale: ½" = 1'-0"

SEE BENT-11 dwg.no.37107

NOTE

For sections A-A & C-C see dwg. no. 37111

For sections B-B, D-D & E-E see bent-11 dwg.no.37107

For post-tensioning details see dwg. no. 37128

APPROVED: Walter A Hunt
STRUCTURAL DESIGN ENGINEER

DESIGNED G. Bellin
DRAWN J. Hardin
CHECKED W.M. Thompson
REVIEWED J.M. Tindall 7/2/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1632

DATE	REVISION

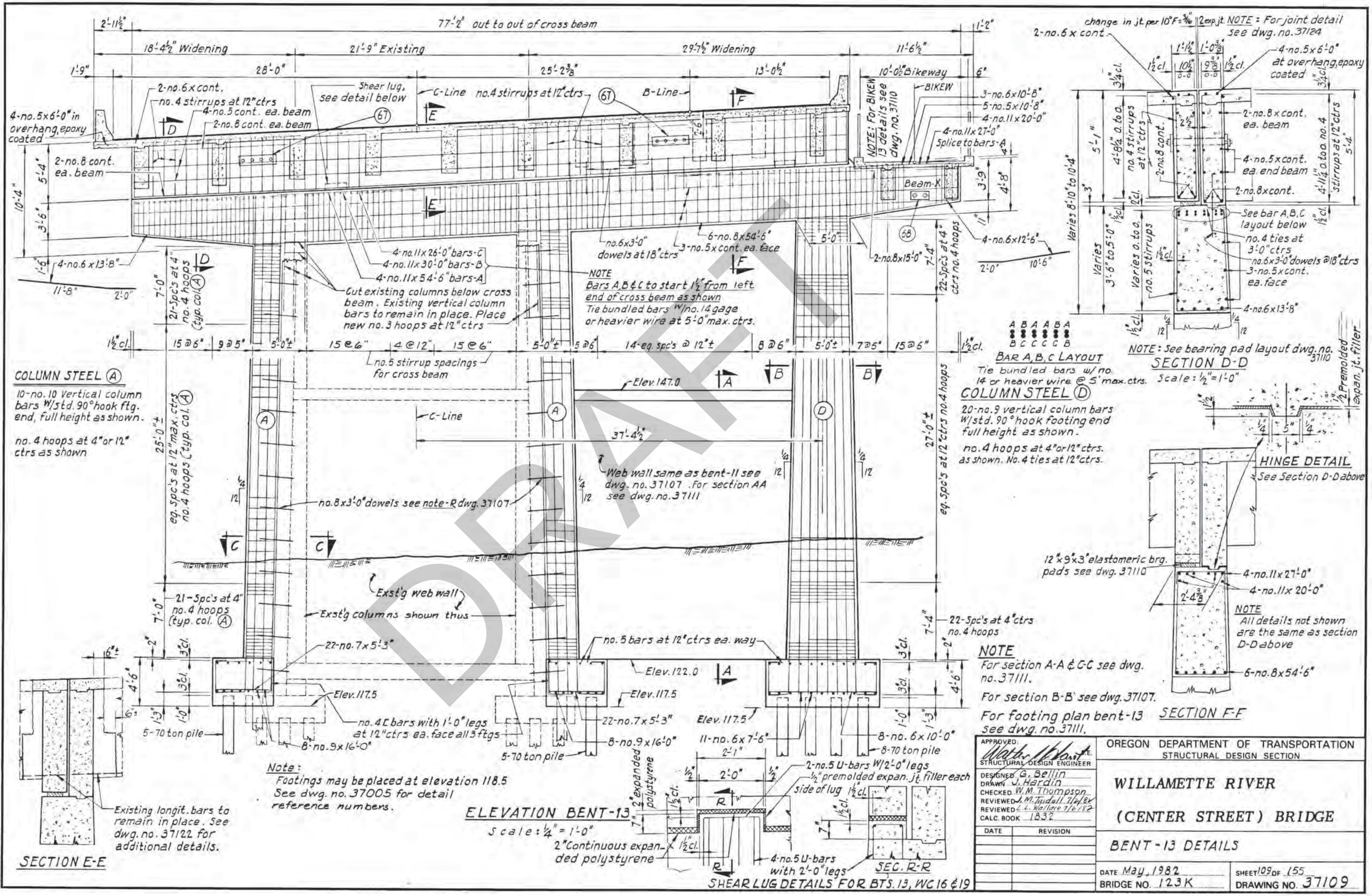
OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

BENT-12 DETAILS

DATE May, 1982
BRIDGE NO. 123K

SHEET 108 OF 155
DRAWING NO. 37108



NOTE
For section A-A & C-C see dwg. no. 37111.
For section B-B see dwg. 37107.
For footing plan bent-13 see dwg. no. 37111.

APPROVED: Walter H. Hart P.E.
STRUCTURAL DESIGN ENGINEER
DESIGNED G. Bellin
DRAWN J. Hardin
CHECKED W.M. Thompson
REVIEWED J.M. Tindall 7/2/82
REVIEWED L.L. Wallace 7/6/82
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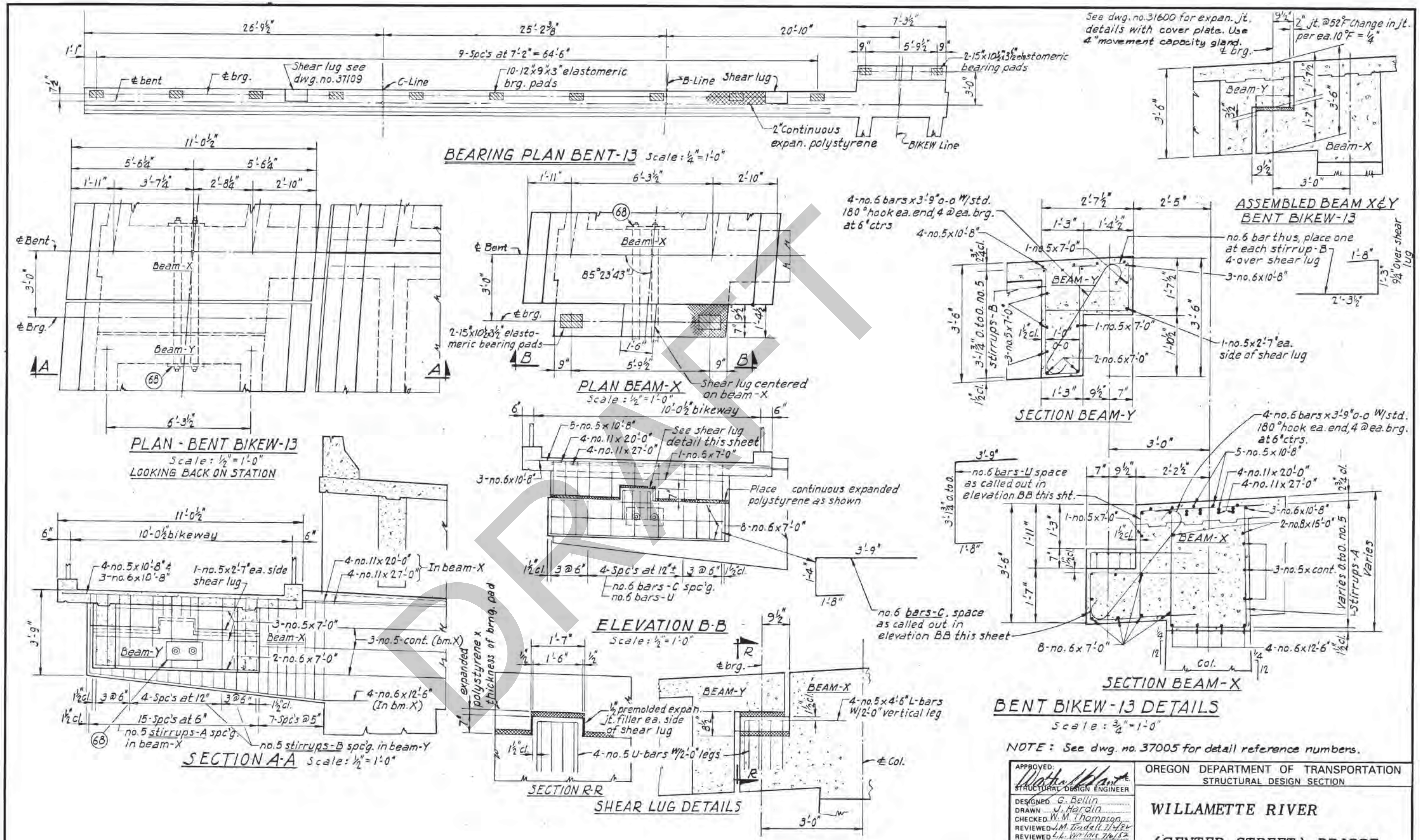
OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

BENT-13 DETAILS

DATE May, 1982
BRIDGE NO. 123K
SHEET 109 OF 155
DRAWING NO. 37109



BENT BIKEW-13 DETAILS

Scale: ¾"=1'-0"

NOTE: See dwg. no. 37005 for detail reference numbers.

APPROVED:
STRUCTURAL DESIGN ENGINEER

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DRAWN J. Hardin
CHECKED W. M. Thompson
REVIEWED J.M. Tindall 7/1/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

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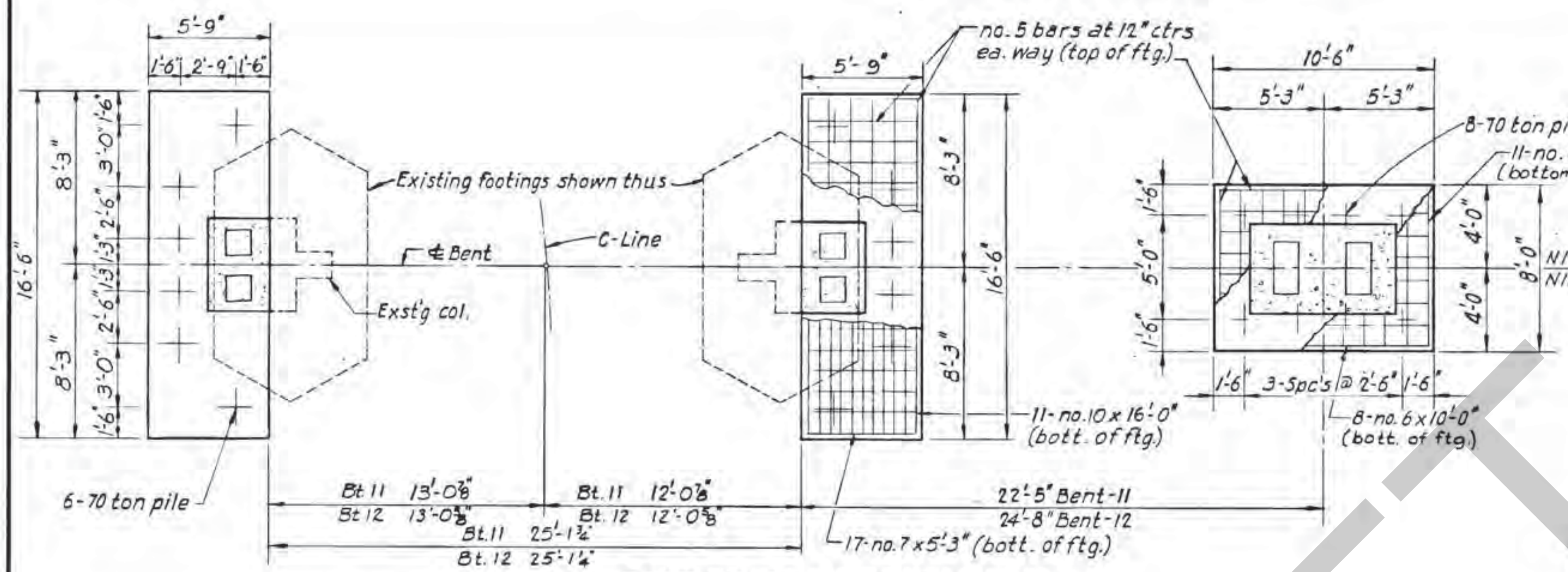
OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

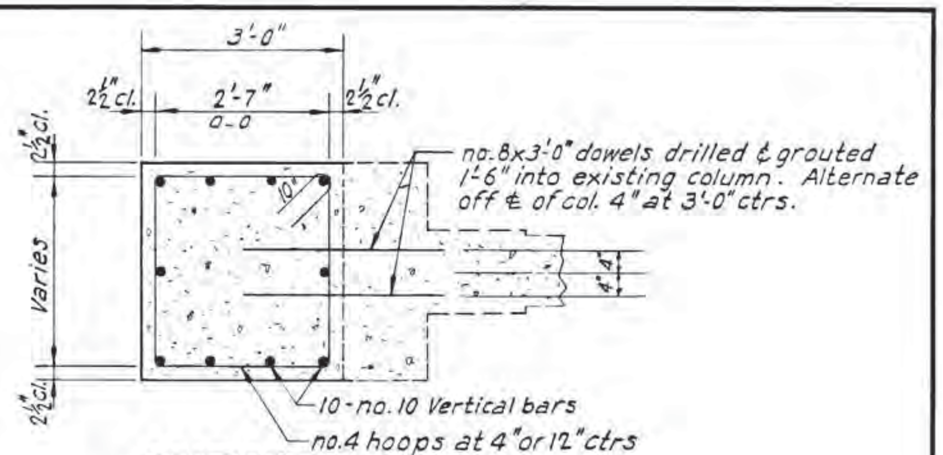
BENT BIKEW-13 DETAILS

DATE May, 1982
BRIDGE NO. 123K
SHEET 110 OF 155
DRAWING NO. 37110



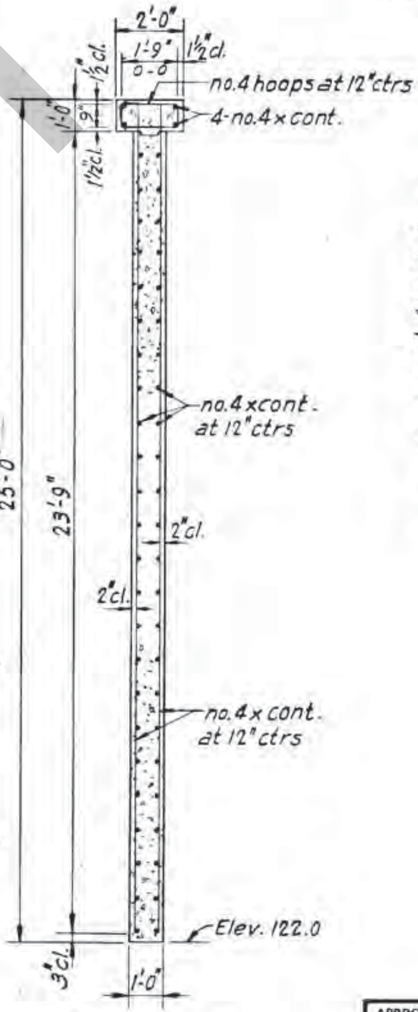
FOOTING PLAN BENTS-11 & 12

Scale: ¼" = 1'-0"



SECTION C-C

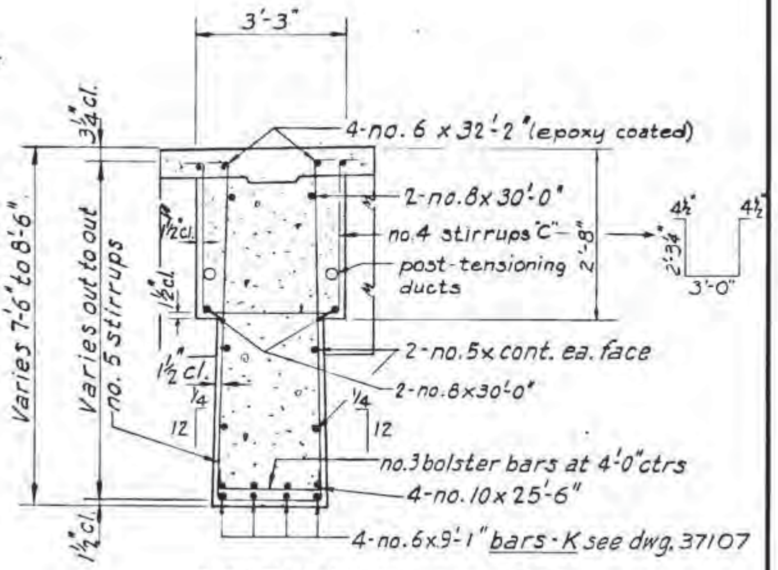
Typical for columns Ⓐ bents 11, 12 & 13



SECTION A-A

Scale: ⅜" = 1'-0"

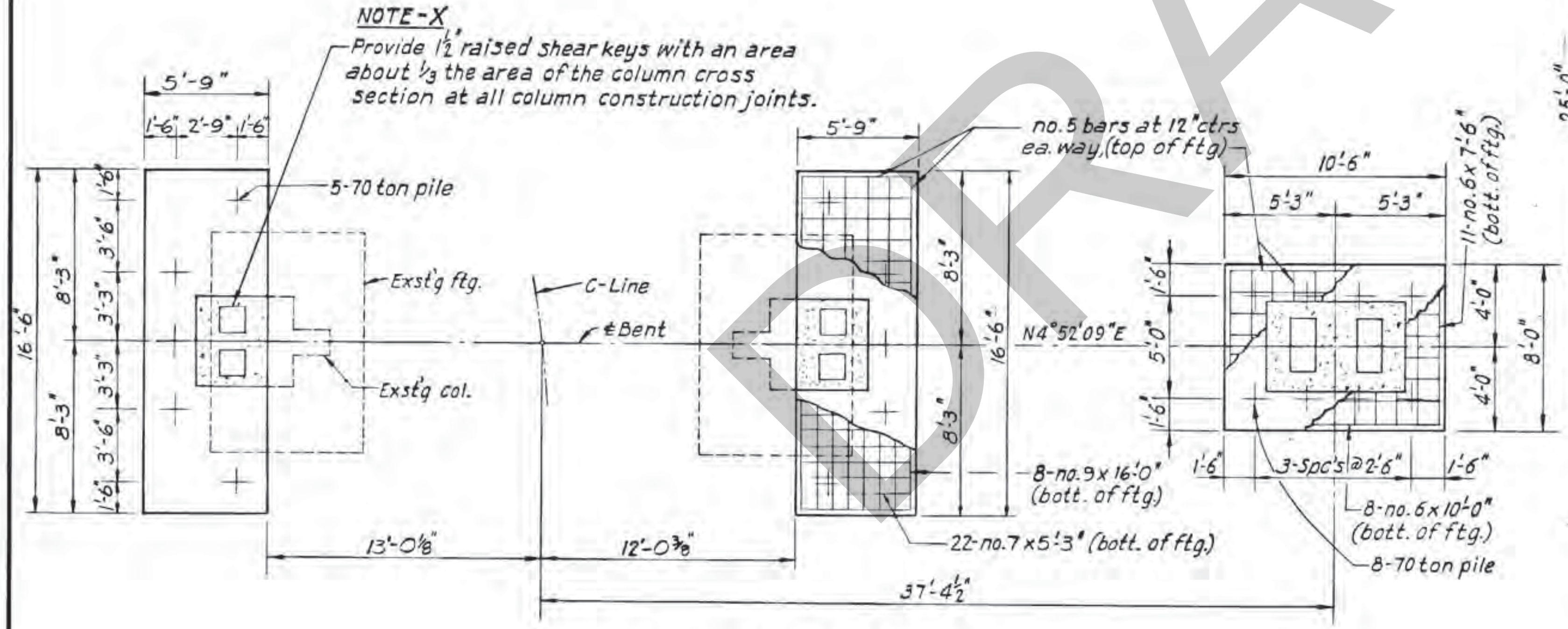
See dwg. no. 37107, bent-11 & dwg. no. 37108, bent-12 for typ. wall section A-A & dwg. no. 37109, bent-13.



SECTION F-F

Scale: ½" = 1'-0"

Taken at x-beam bent-11 see dwg. no. 37107



FOOTING PLAN BENT-13

Scale: ¼" = 1'-0"

APPROVED:
STRUCTURAL DESIGN ENGINEER
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DRAWN J. Hardin
CHECKED W. M. Thompson
REVIEWED J. M. Tindall 7/6/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

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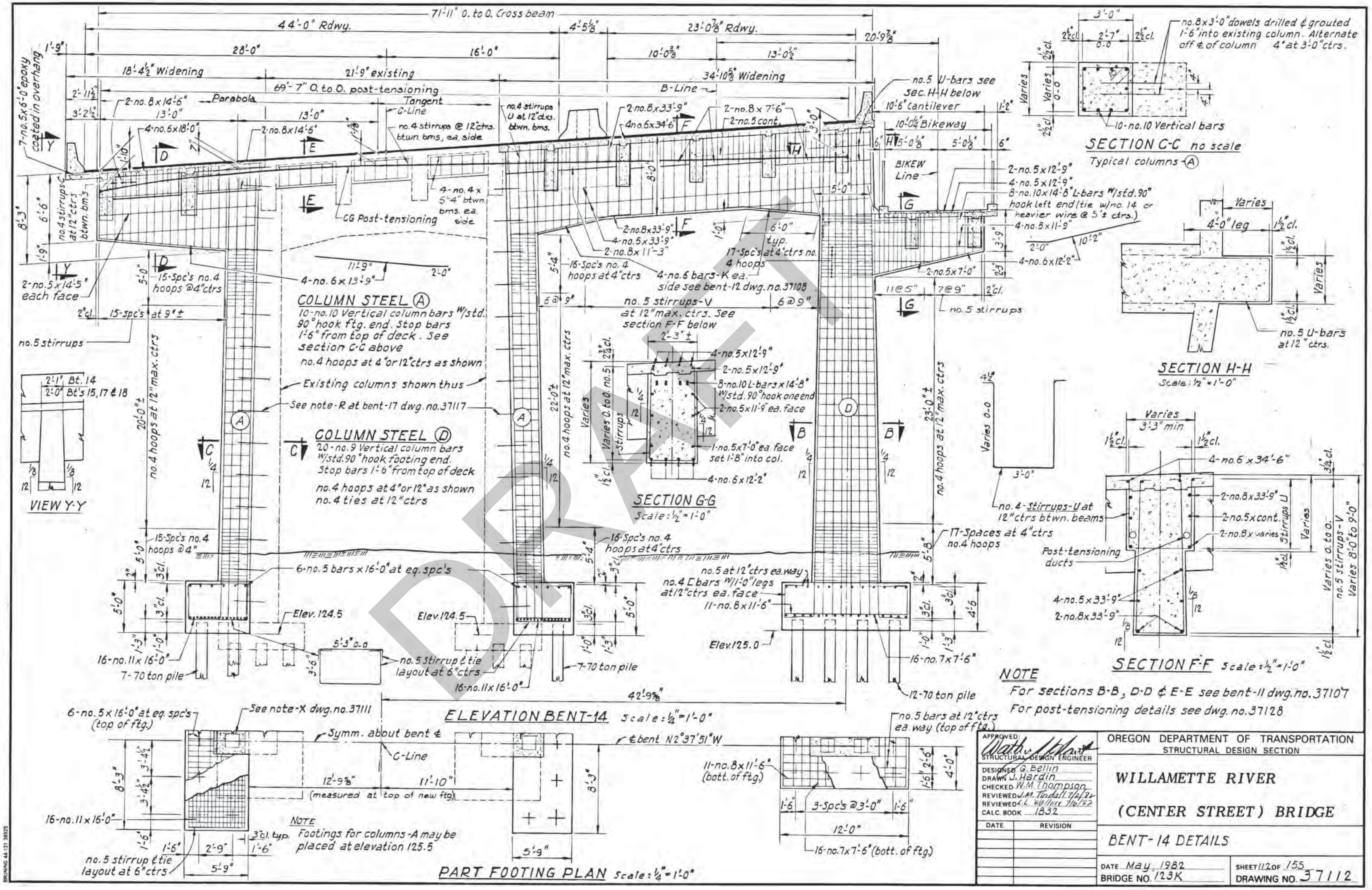
OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

FOOTING PLAN & BENT DETAILS
BENTS 11, 12 & 13

DATE May, 1982
SHEET 111 OF 155
BRIDGE NO. 123K
DRAWING NO. 37111



NOTE
Footings for columns-A may be placed at elevation 125.5

NOTE
For sections B-B, D-D & E-E see bent-11 dwg.no.37107
For post-tensioning details see dwg. no.37128

APPROVED: Walter J. Hart
STRUCTURAL DESIGN ENGINEER
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REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

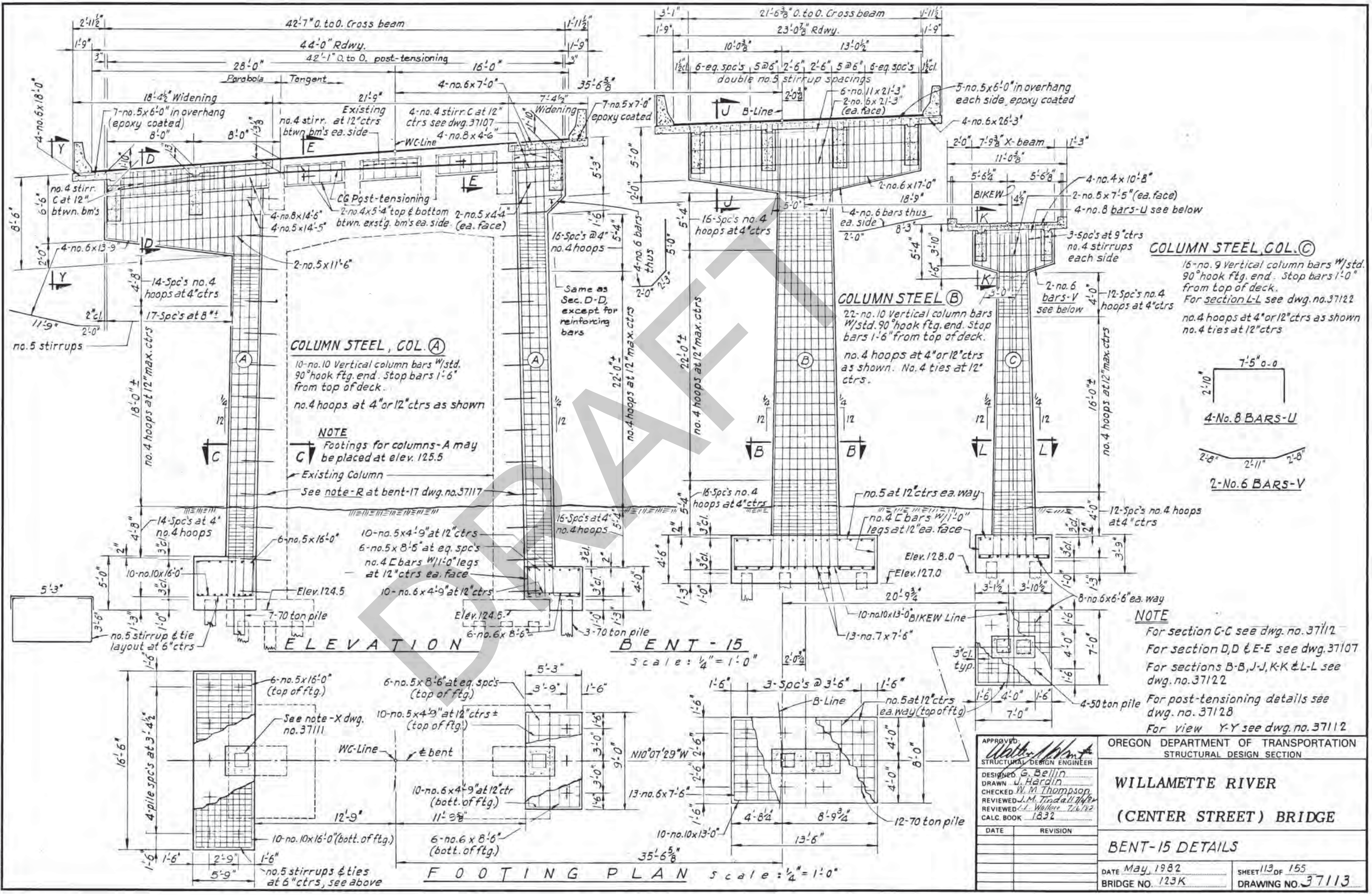
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

BENT-14 DETAILS

DATE May, 1982
BRIDGE NO. 123K
SHEET 112 OF 155
DRAWING NO. 37112



NOTE

For section C-C see dwg. no. 37112

For section D,D & E-E see dwg. 37107

For sections B-B, J-J, K-K & L-L see dwg. no. 37122

For post-tensioning details see dwg. no. 37128

For view Y-Y see dwg. no. 37112

APPROVED: Walter [illegible]
STRUCTURAL DESIGN ENGINEER
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DRAWN J. Hardin
CHECKED W. M. Thompson
REVIEWED J. M. Tindall 7/4/82
REVIEWED J. L. Wallace 7/6/82
CALC. BOOK 1832

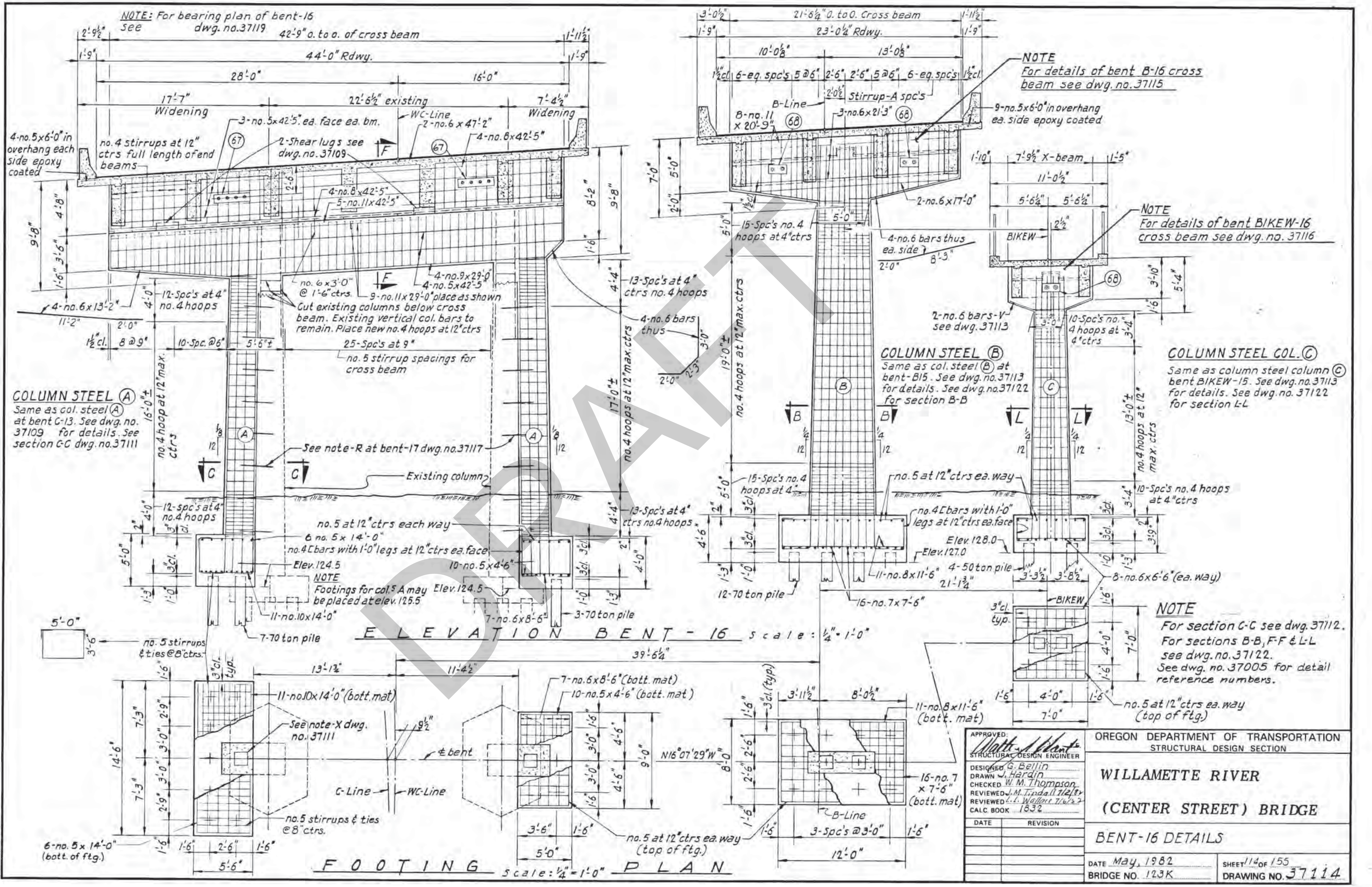
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

BENT-15 DETAILS

DATE May, 1982
BRIDGE NO. 123K
SHEET 113 OF 155
DRAWING NO. 37113



NOTE
For section C-C see dwg. 37112.
For sections B-B, F-F & L-L see dwg. no. 37122.
See dwg. no. 37005 for detail reference numbers.

APPROVED: Walter J. Hart
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DRAWN J. Hardin
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REVIEWED J.M. Tindall 7/12/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

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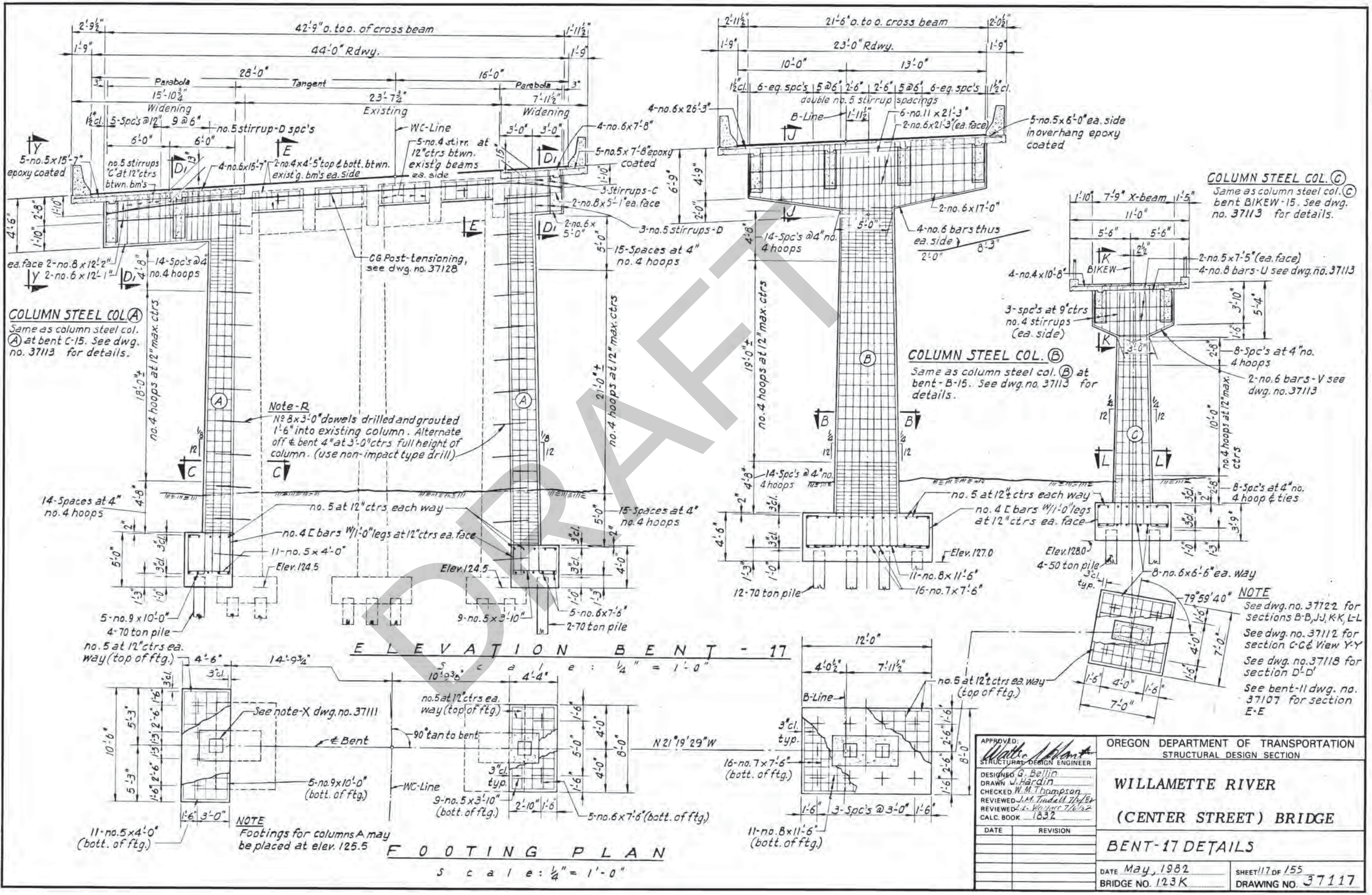
OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

BENT-16 DETAILS

DATE May, 1982
BRIDGE NO. 123K
SHEET 114 OF 155
DRAWING NO. 37114



COLUMN STEEL COL. Ⓐ
Same as column steel col. Ⓐ at bent C-15. See dwg. no. 37113 for details.

COLUMN STEEL COL. Ⓑ
Same as column steel col. Ⓑ at bent-B-15. See dwg. no. 37113 for details.

COLUMN STEEL COL. Ⓒ
Same as column steel col. Ⓒ bent BIKEW-15. See dwg. no. 37113 for details.

NOTE
See dwg. no. 37722 for sections B-B, JJ, K-K, L-L
See dwg. no. 37112 for section C-C & View Y-Y
See dwg. no. 37118 for section D'-D'
See bent-11 dwg. no. 37107 for section E-E

NOTE
Footings for columns A may be placed at elev. 125.5

APPROVED: Walter J. Hunt
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DRAWN J. Hardin
CHECKED W. M. Thompson
REVIEWED J. M. Tindall 7/2/82
REVIEWED L. L. Wallace 7/6/82
CALC. BOOK 1832

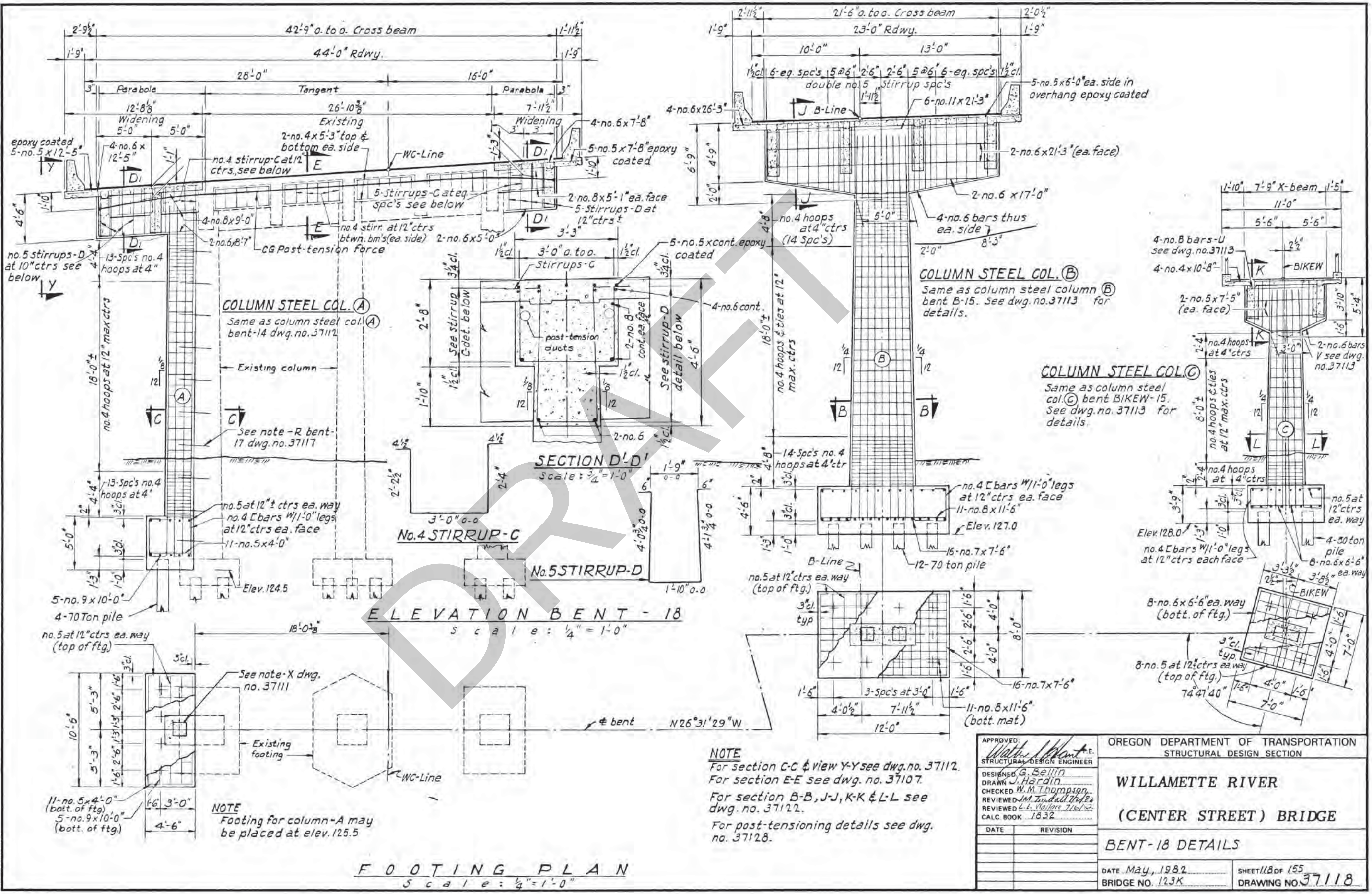
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

BENT-17 DETAILS

DATE May, 1982
BRIDGE NO. 123K
SHEET 117 OF 155
DRAWING NO. 37117



NOTE
Footing for column-A may be placed at elev. 125.5

NOTE
For section C-C & view Y-Y see dwg. no. 37112.
For section E-E see dwg. no. 37107.
For section B-B, J-J, K-K & L-L see dwg. no. 37122.
For post-tensioning details see dwg. no. 37128.

APPROVED: Walter J. Hunt P.E.
STRUCTURAL DESIGN ENGINEER
DESIGNED G. Bellin
DRAWN J. Hardin
CHECKED W. M. Thompson
REVIEWED J.M. Tindall 7/21/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

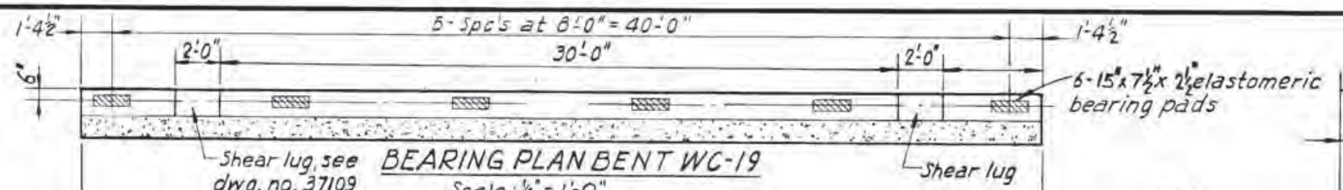
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

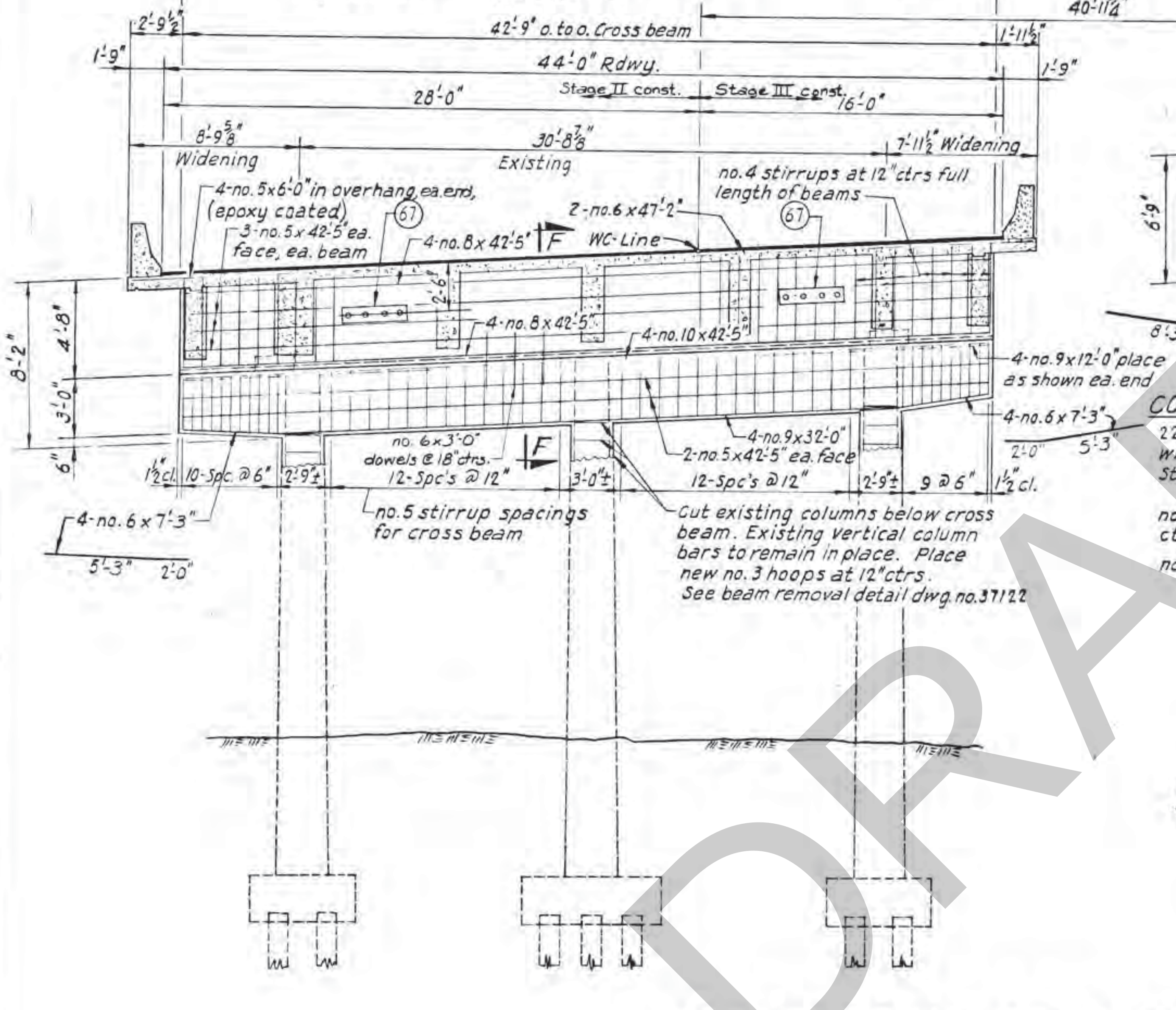
WILLAMETTE RIVER
(CENTER STREET) BRIDGE

BENT-18 DETAILS

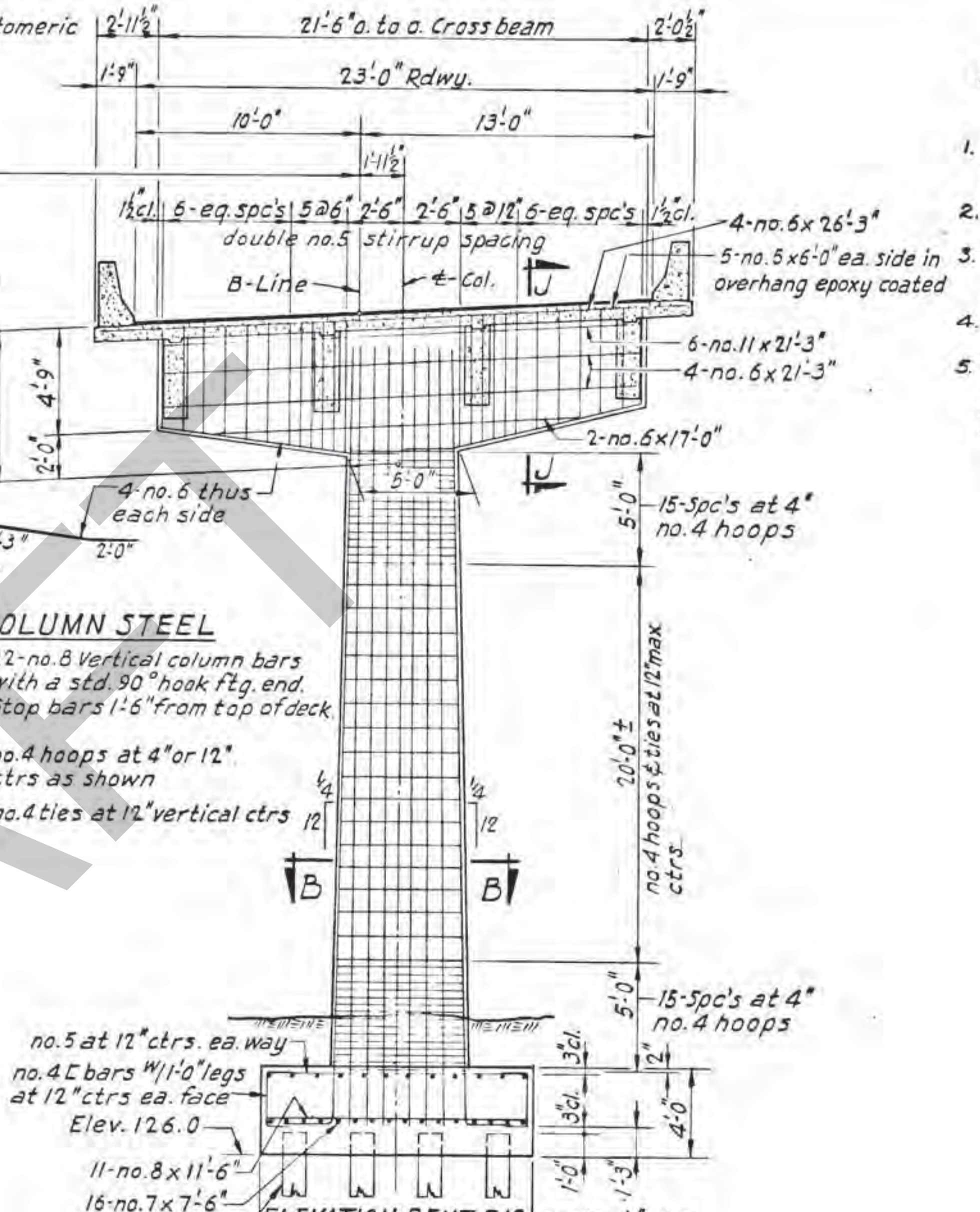
DATE May, 1982
BRIDGE NO. 123K
SHEET 118 OF 155
DRAWING NO. 37118



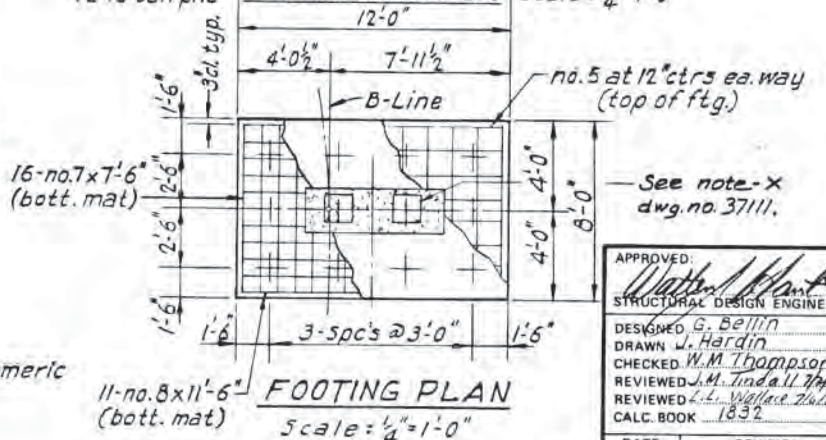
BEARING PLAN BENT WC-19
Scale: ¼" = 1'-0"



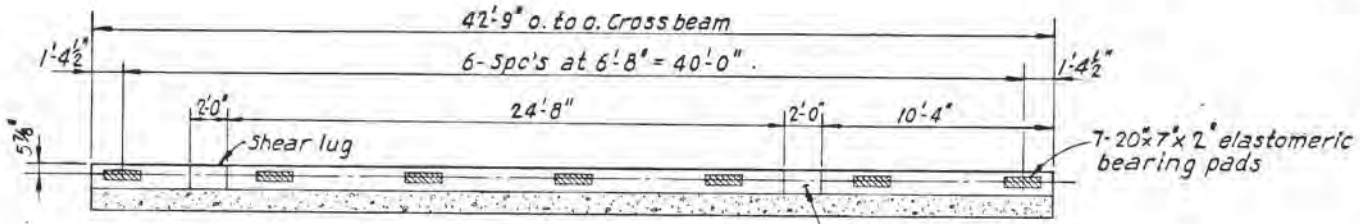
ELEVATION BENT-19
Scale: ¼" = 1'-0"



ELEVATION BENT-B19 Scale: ¼" = 1'-0"



FOOTING PLAN
Scale: ¼" = 1'-0"



BEARING PLAN BENT WC-16
Scale: ¼" = 1'-0"

△1 CONSTRUCTION SEQUENCE BENT 19

1. Support existing longitudinal beams for 45 ton capacity each side of existing Beam 19.
2. Cut columns and beam pedestals and remove existing Beam 19.
3. Remove Stage II portion of existing longitudinal and end beams and construct all of new Beam 19.
4. Construct Stage II portion of new longitudinal and end beams.
5. Complete Stage III portion of longitudinal and end beams after East bound traffic has been rerouted to new Center St. structure.

NOTE
For sections B-B, F-F, J-J & existing cross beam removal for bent WC-19 see dwg. no. 37122
See dwg. no. 37005 for detail reference numbers.

APPROVED: Walter H. Hart P.E.
STRUCTURAL DESIGN ENGINEER
DESIGNED G. Bellin
DRAWN J. Hardin
CHECKED W.M. Thompson
REVIEWED J.M. Tindall 7/7/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

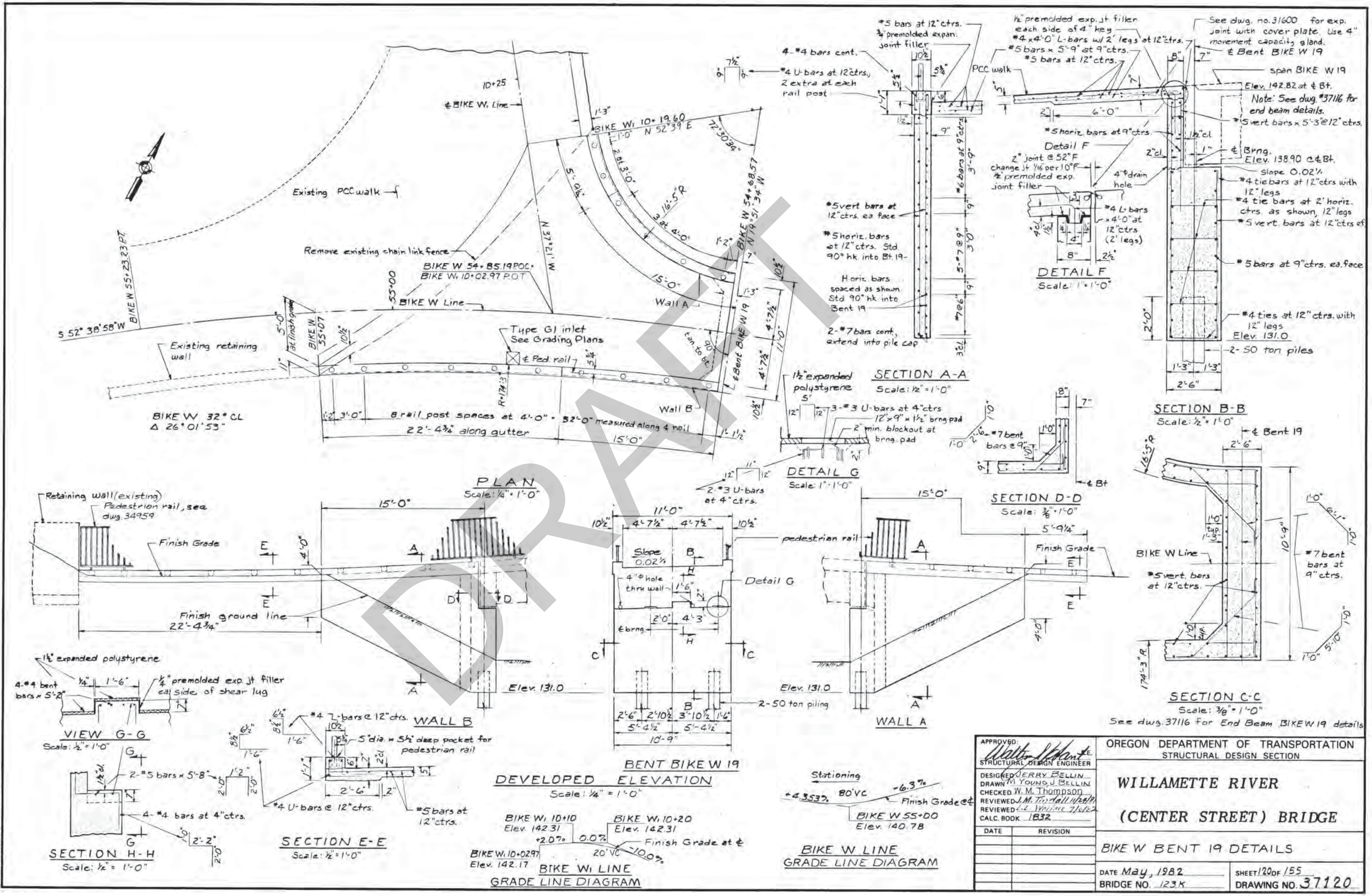
DATE	REVISION
△1 8-11-82	Add const. sequence

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

BENT-19 DETAILS

DATE May, 1982
BRIDGE NO. 123K
SHEET 119 OF 155
DRAWING NO 37119



APPROVED: Walter J. Hart
STRUCTURAL DESIGN ENGINEER
DESIGNED JERRY BELLIN
DRAWN M YOUNG, J BELLIN
CHECKED W. M. Thompson
REVIEWED J.M. Tindall 11/24/81
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

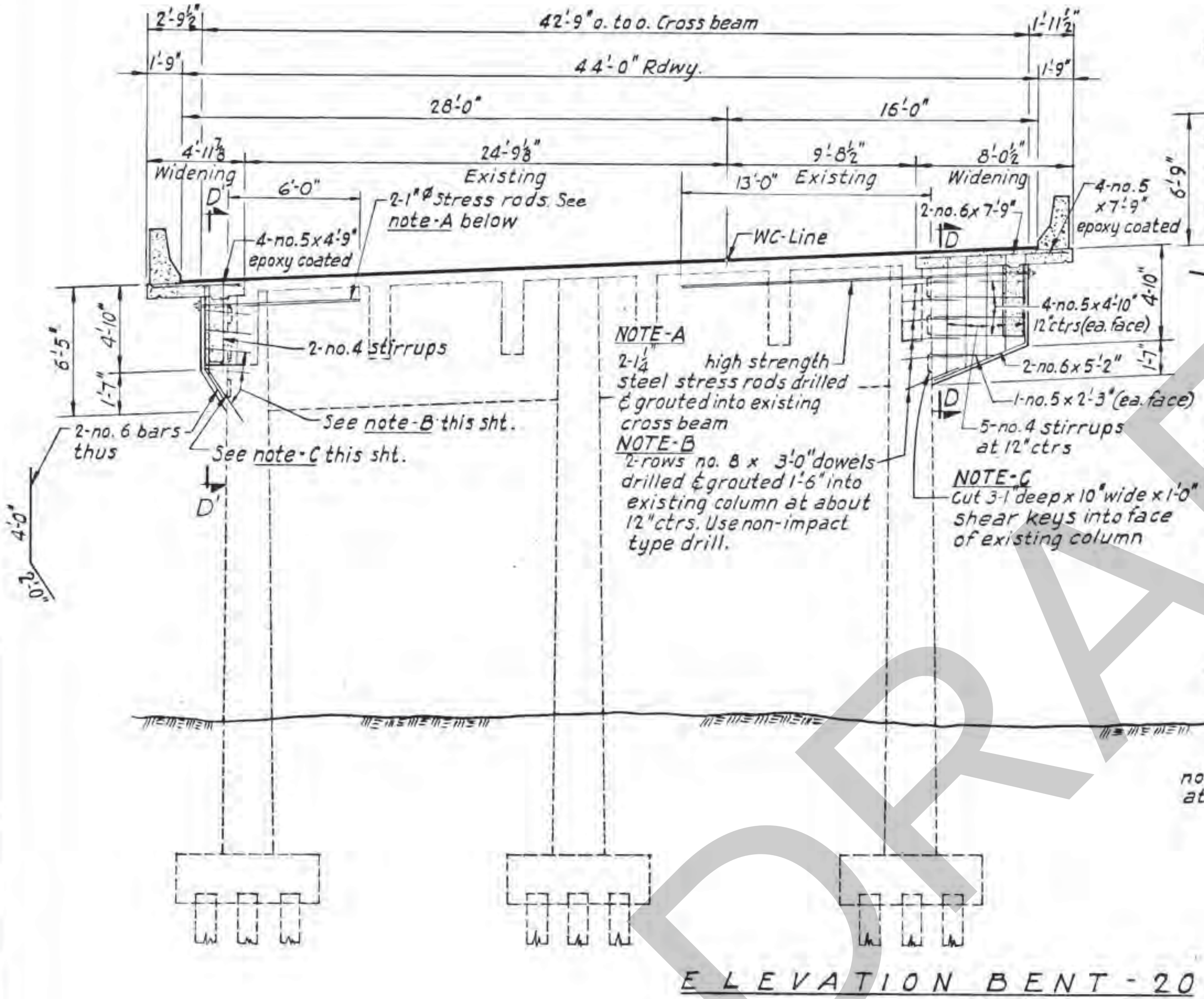
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER (CENTER STREET) BRIDGE

BIKE W BENT 19 DETAILS

DATE May, 1982
BRIDGE NO. 123K
SHEET 120 OF 155
DRAWING NO. 37120

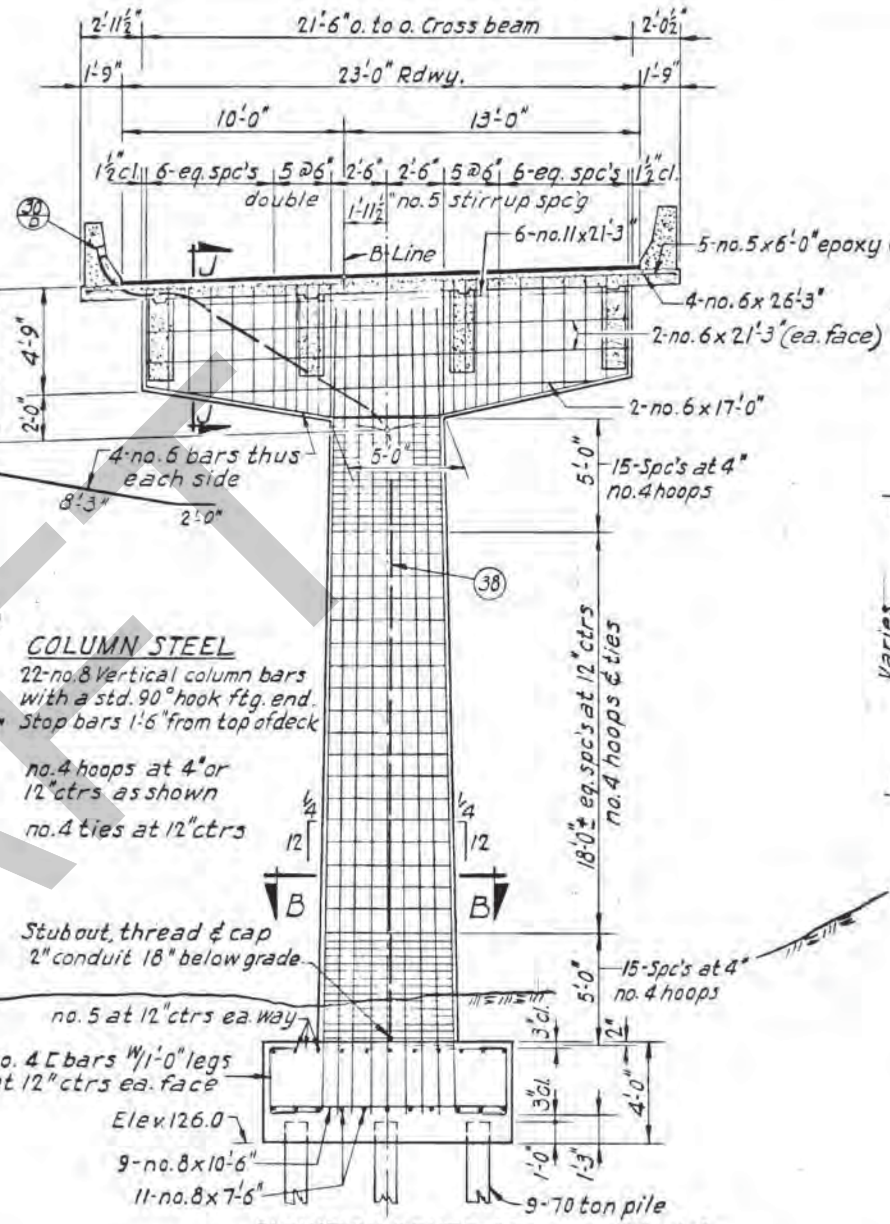


NOTE-A
2-1¼" high strength steel stress rods drilled & grouted into existing cross beam

NOTE-B
2-rows no. 8 x 3'-0" dowels drilled & grouted 1'-6" into existing column at about 12"ctrs. Use non-impact type drill.

NOTE-C
Cut 3-1" deep x 10" wide x 1'-0" shear keys into face of existing column

ELEVATION BENT-20
Scale: ¼"=1'-0"

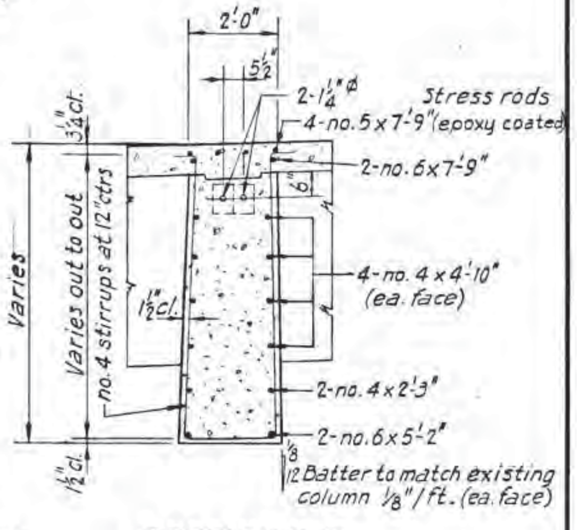


COLUMN STEEL
22-no.8 Vertical column bars with a std. 90° hook ftg. end. Stop bars 1'-6" from top of deck

no.4 hoops at 4" or 12"ctrs as shown

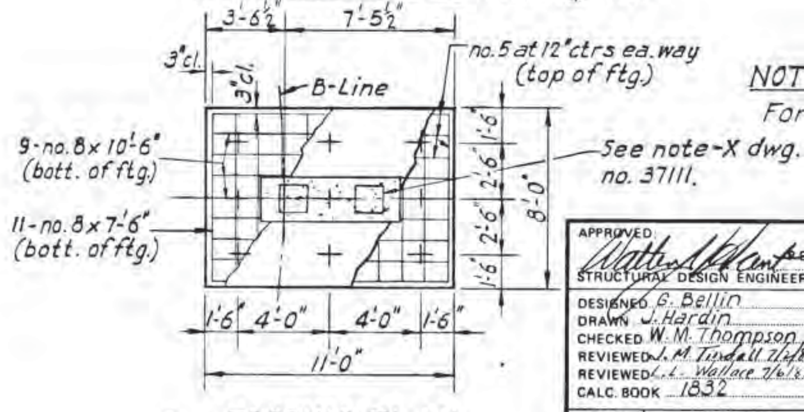
no.4 ties at 12"ctrs

ELEVATION BENT B-20 Scale: ¼"=1'-0"



SECTION D-D
Scale: ½"=1'-0"

Section D'-D' similar



FOOTING PLAN
Scale: ¼"=1'-0"

NOTE
For sections B-B & J-J see dwg. no. 37122
See dwg. no. 37005 for detail reference nos.

APPROVED
STRUCTURAL DESIGN ENGINEER
DESIGNED G. Bellin
DRAWN J. Hardin
CHECKED W. M. Thompson
REVIEWED J. M. Tindall 7/2/82
REVIEWED L. L. Wallace 7/6/82
CALC. BOOK 1832

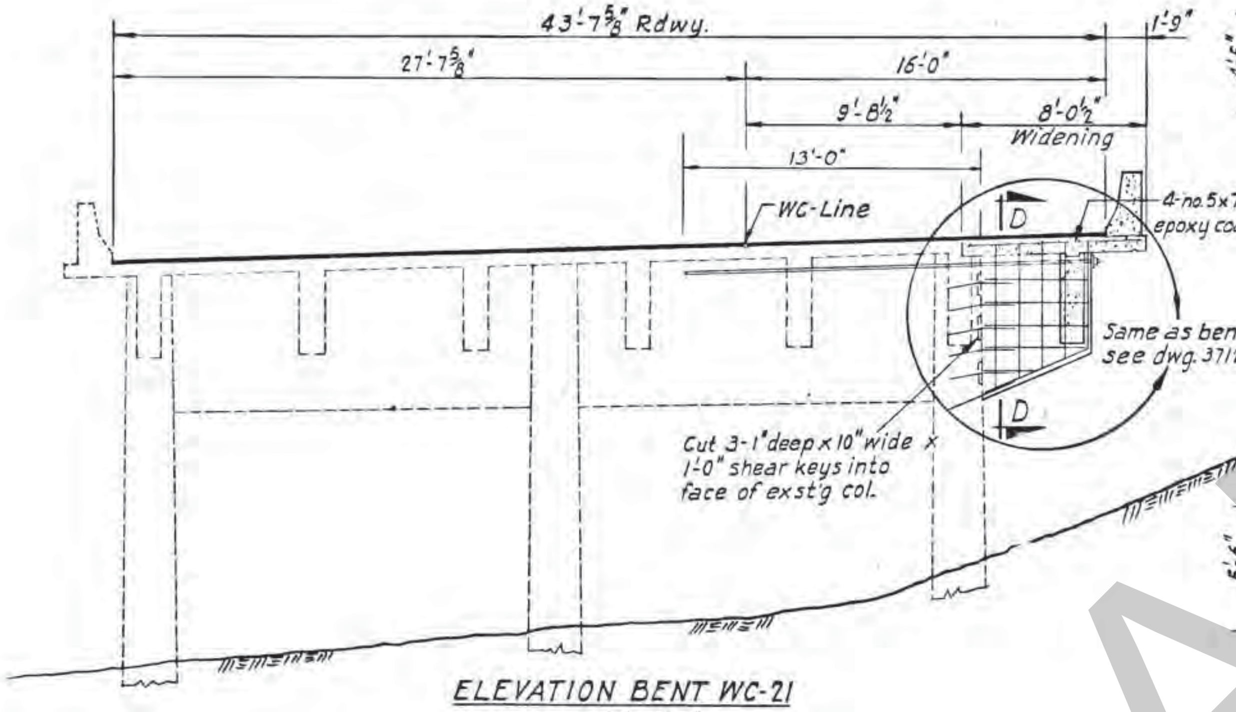
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

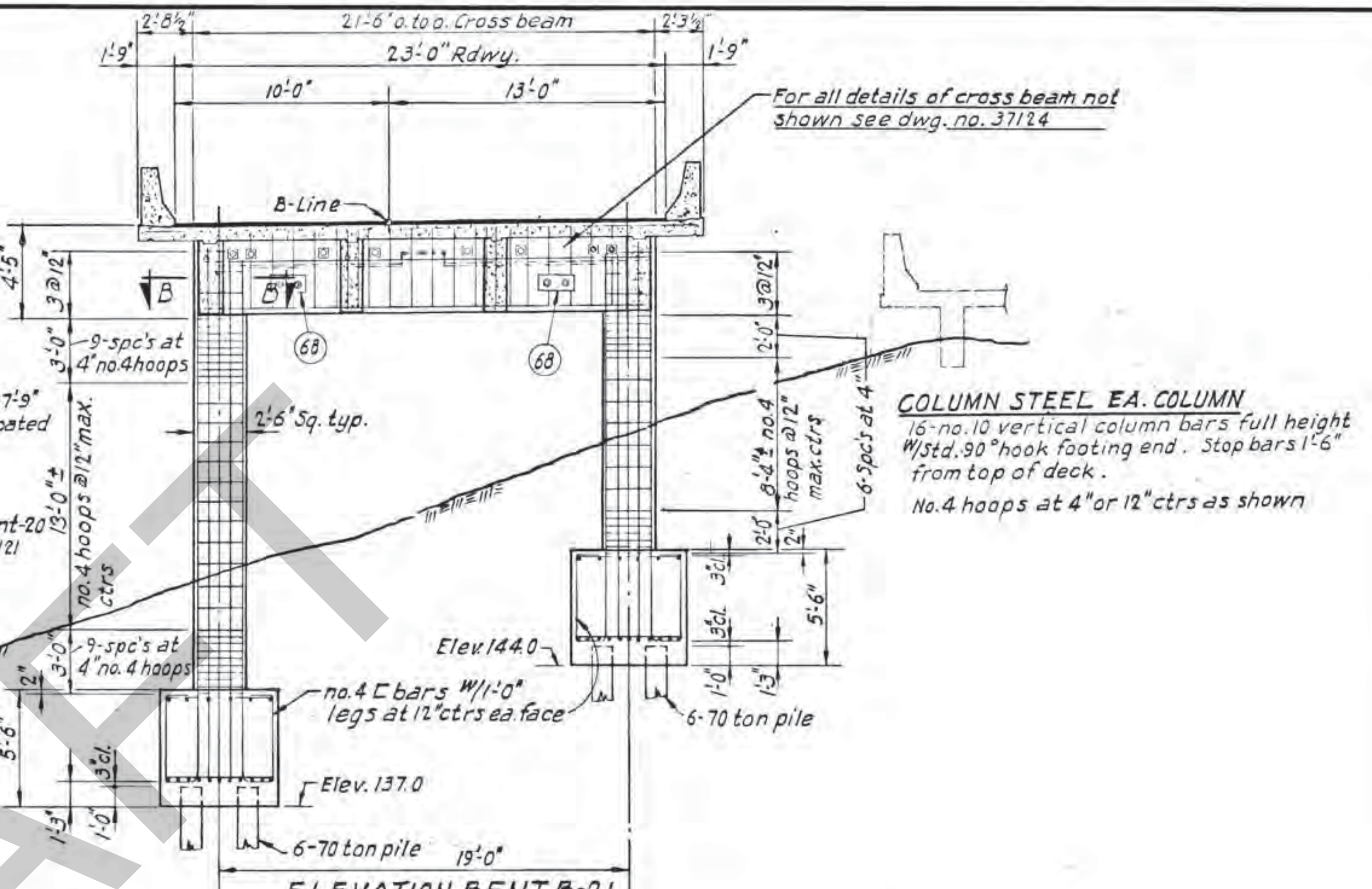
BENT-20 DETAILS

DATE May, 1982
BRIDGE NO. 123K
SHEET 121 OF 155
DRAWING NO. 37121



ELEVATION BENT WC-21

Scale: ¼" = 1'-0"



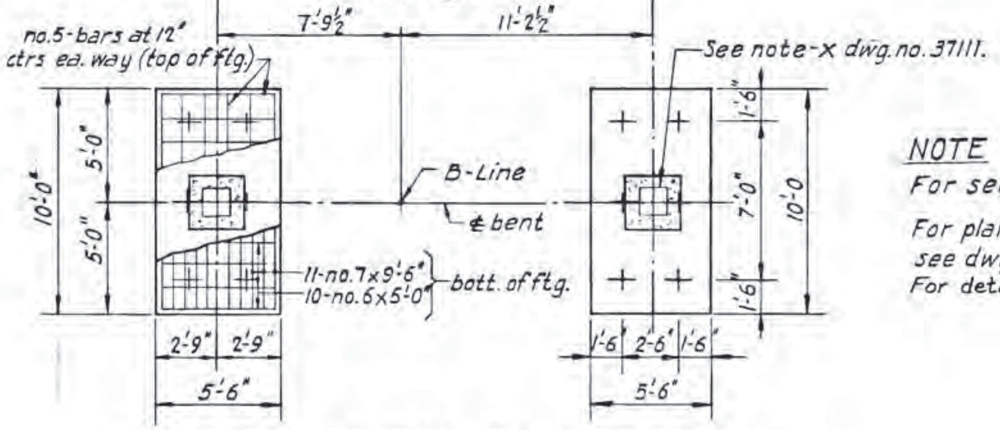
COLUMN STEEL EA. COLUMN

16-no.10 vertical column bars full height w/Std. 90° hook footing end. Stop bars 1'-6" from top of deck.

No.4 hoops at 4" or 12" ctrs as shown

ELEVATION BENT B-21

Scale: ¼" = 1'-0"



TYPICAL FOOTING PLAN BENT-B21

Scale: ¼" = 1'-0"

NOTE

For section B-B see bent-22 dwg. no. 37125.

For plan view and sections of bent B-21 see dwg. no. 37124.

For detail reference numbers see dwg. no. 37005.

APPROVED: Walter Hart P.E.
STRUCTURAL DESIGN ENGINEER

DESIGNED G. Bellin
DRAWN J. Hardin
CHECKED W.M. Thompson
REVIEWED J.M. Tindall 7/6/82
REVIEWED L.L. Wallace 7/6/82
CALC. BOOK 1832

DATE	REVISION

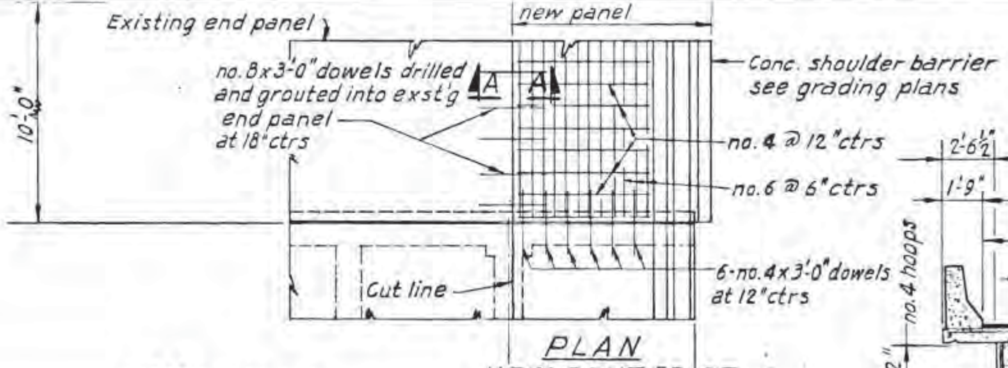
OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER

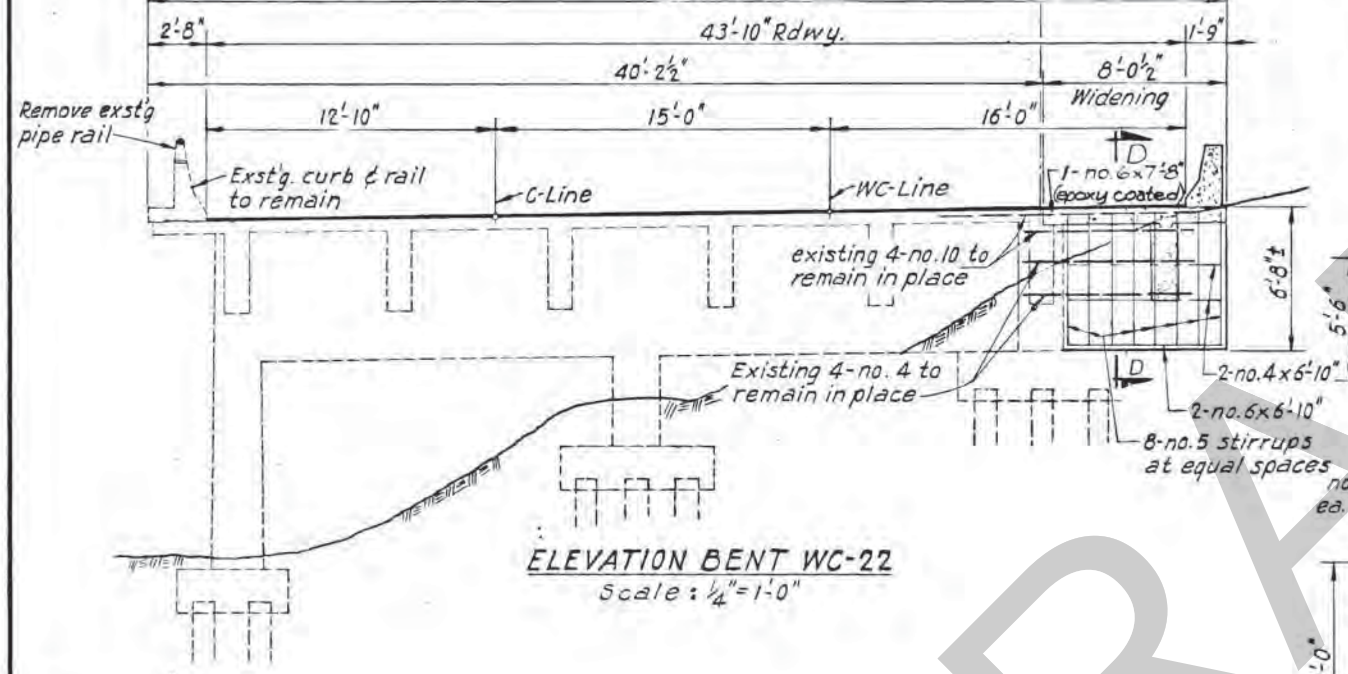
(CENTER STREET) BRIDGE

BENT-21 DETAILS

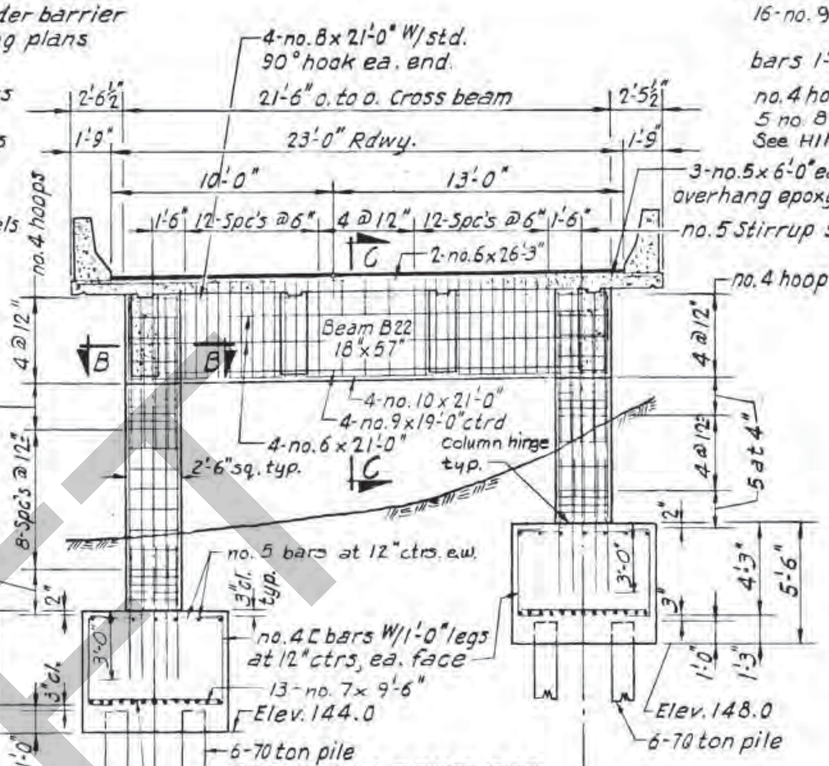
DATE May, 1982
BRIDGE NO. 123K
SHEET 123 OF 155
DRAWING NO. 37123



PLAN
NEW BENT-22 RT.



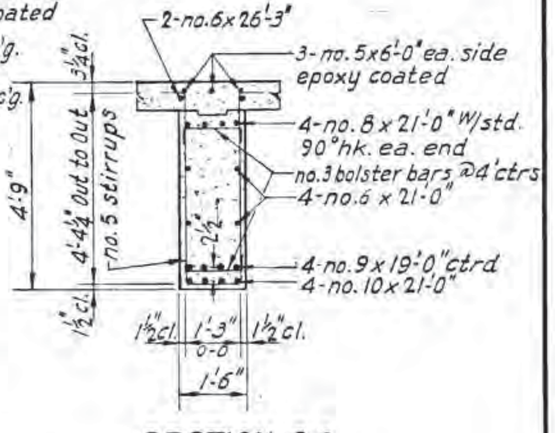
ELEVATION BENT WC-22
Scale: ¼" = 1'-0"



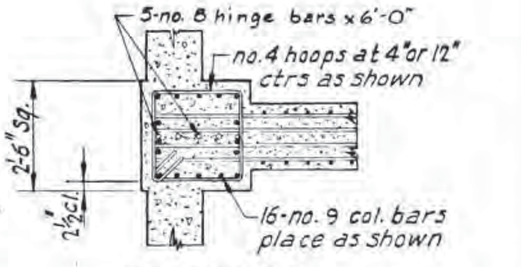
ELEVATION BENT-B22
Scale: ¼" = 1'-0"

COLUMN STEEL BT. B22 EA. COLUMN

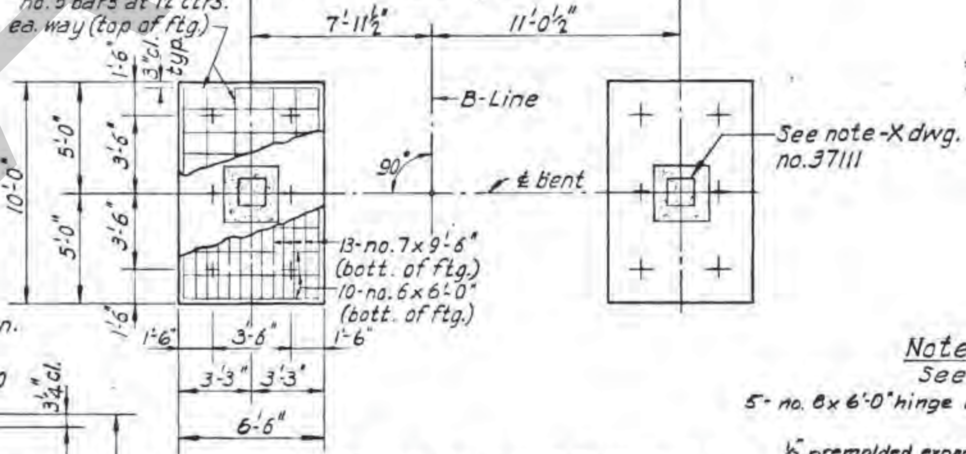
16-no. 9 Vertical column bars
Stop bars 1'-6" from top of conc. deck
no. 4 hoops at 4" or 12" ctrs as shown
5 no. 8 x 6'-0" hinge bars top & bottom of column
See HINGE DETAIL below



SECTION C-C
Scale: ½" = 1'-0"

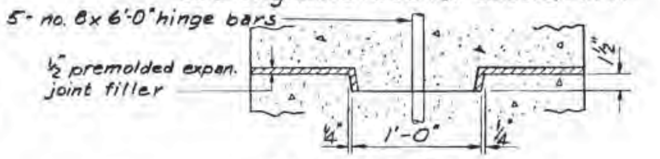


SECTION-BB
Scale: ½" = 1'-0"

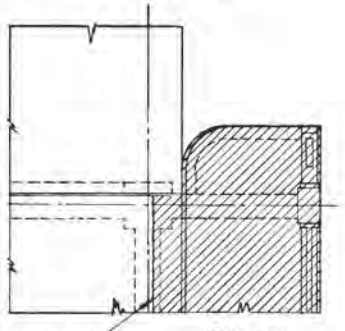


FOOTING PLAN BENT B-22
Scale: ¼" = 1'-0"

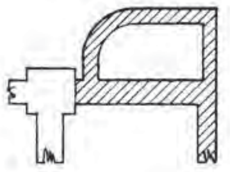
Note
See dwg. no. 37005 for section A-A



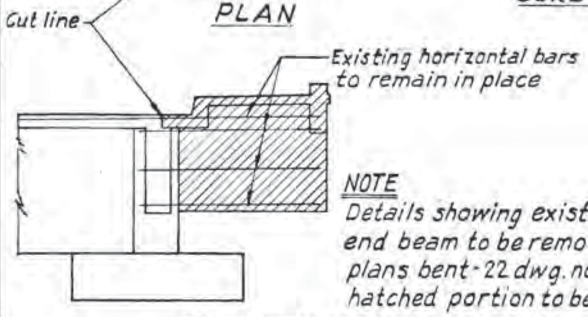
HINGE DETAIL
Scale: 1½" = 1'-0"



PLAN

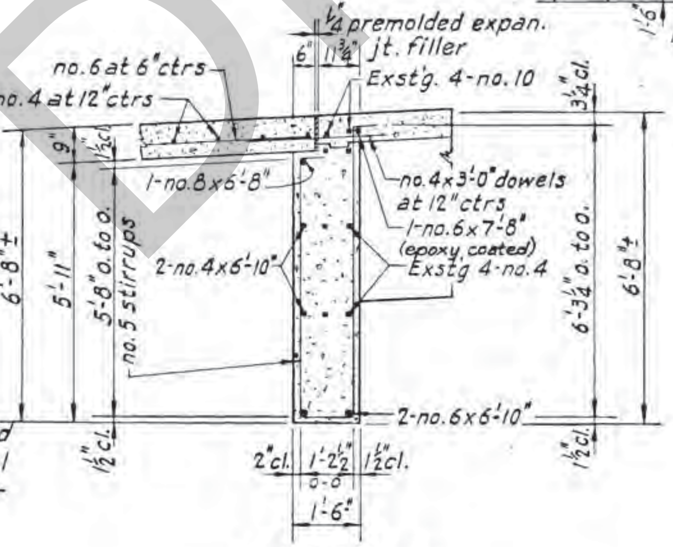


CURB BEAM



NOTE
Details showing existing sidewalk and end beam to be removed. See original plans bent-22 dwg. no. 10103. All cross-hatched portion to be removed.

ELEVATION
EXISTING BENT-22 RIGHT



SECTION D-D
Scale: ½" = 1'-0"

APPROVED: Walter ... P.E.
STRUCTURAL DESIGN ENGINEER
DESIGNED G. Bellin
DRAWN J. Hardin
CHECKED W. M. Thompson
REVIEWED J. M. Tindall 7/2/82
REVIEWED L. L. Wallace 7/6/82
CALC. BOOK 1832

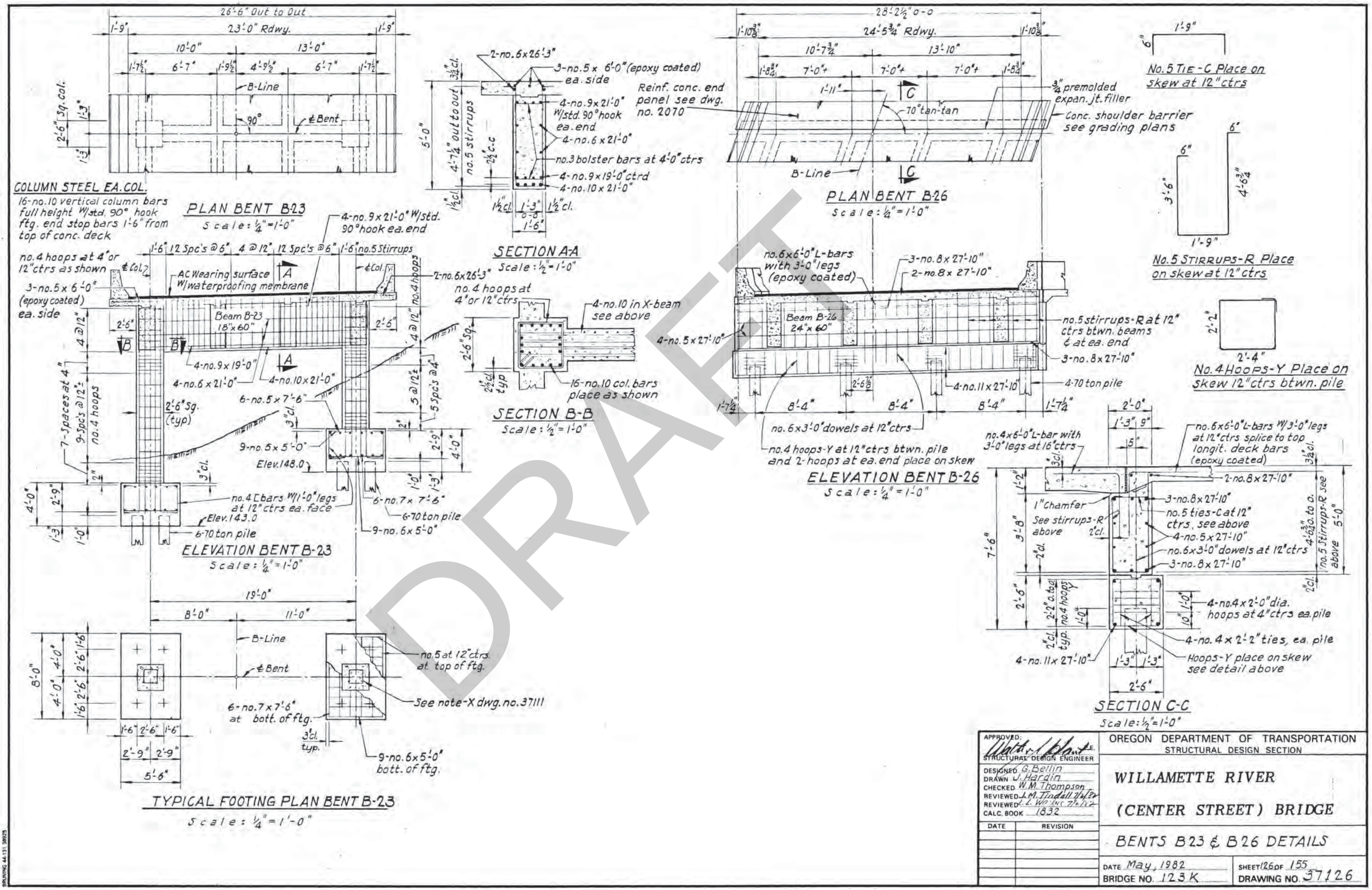
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

BENT-22 DETAILS

DATE May, 1982
BRIDGE NO. 123K
SHEET 125 OF 155
DRAWING NO. 37125



APPROVED: Walter J. Blant
STRUCTURAL DESIGN ENGINEER
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DRAWN J. Hardin
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REVIEWED L.L. Walker 7/6/82
CALC. BOOK 1832

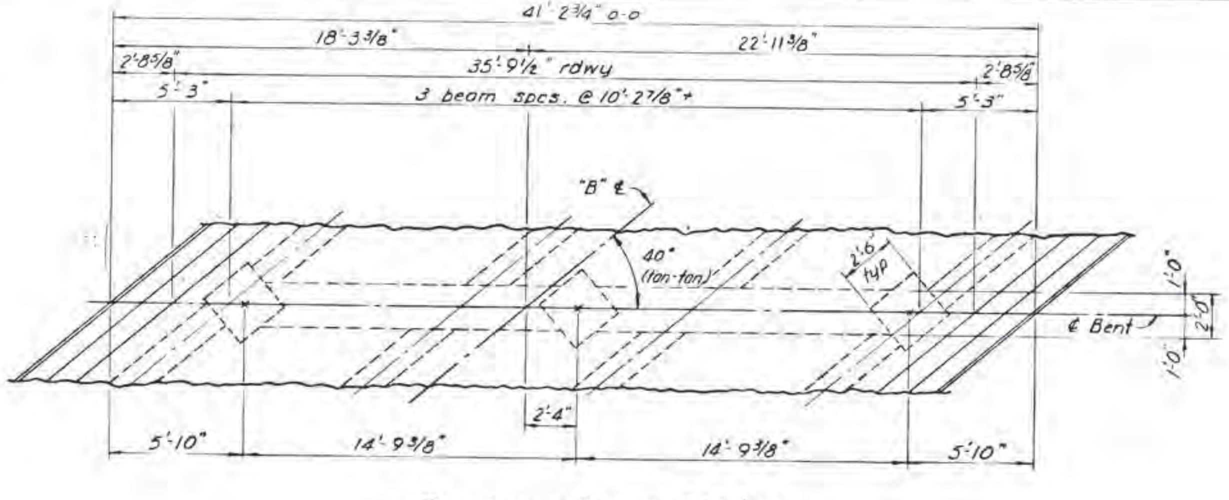
DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

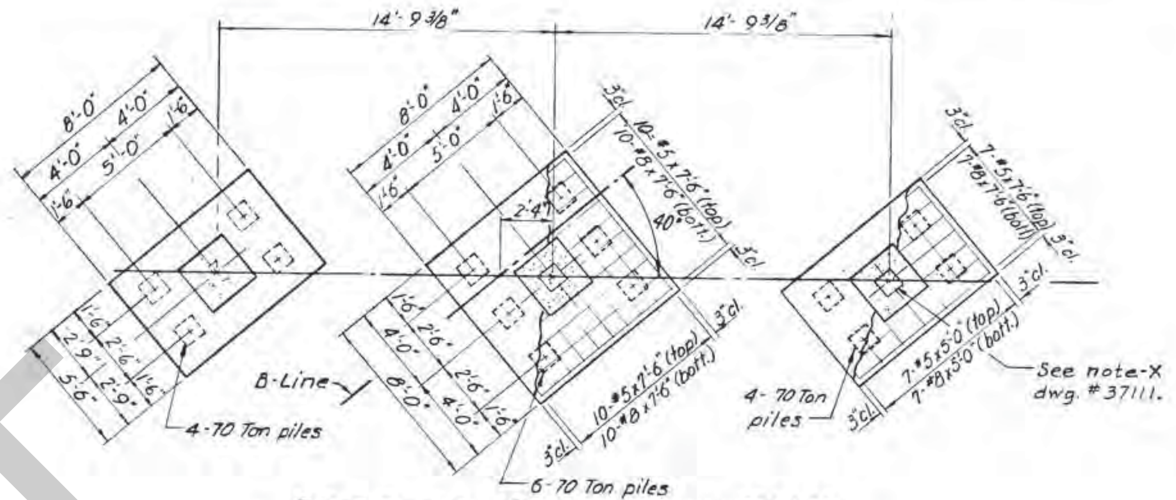
BENTS B23 & B26 DETAILS

DATE May, 1982
BRIDGE NO. 123 K
SHEET 126 OF 155
DRAWING NO. 37126



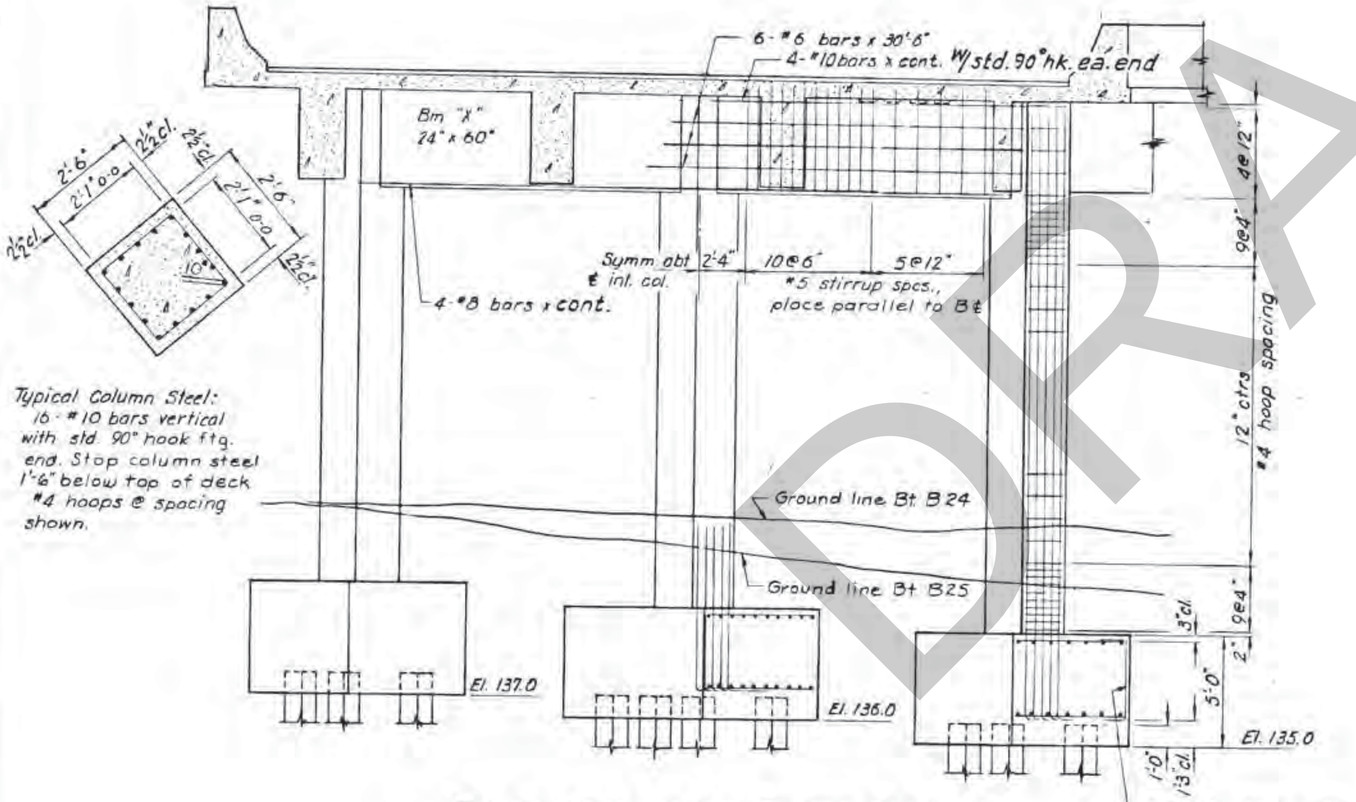
PLAN ~ BENTS B24 & 25

Scale: 1/4" = 1'-0"



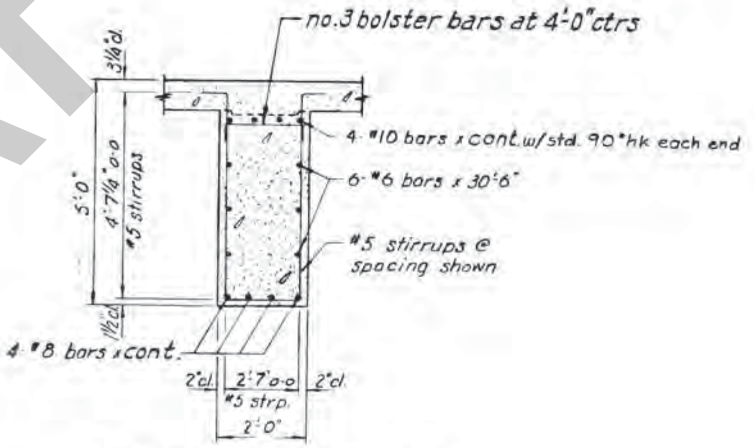
FOOTING PLAN ~ BENTS B24 & 25

Scale: 1/4" = 1'-0"



ELEVATION ~ BENTS B24 & 25

Scale: 1/4" = 1'-0"



TYPICAL SECTION ~ BM "X"

Scale: 1/2" = 1'-0"

APPROVED: Walter [illegible]
STRUCTURAL DESIGN ENGINEER

DESIGNED G.H. Bellin
DRAWN M.C. Young
CHECKED W.M. Thompson
REVIEWED J.M. Tisdall 7/14/82
REVIEWED L.L. Wallace 7/16/82
CALC. BOOK 1832

DATE	REVISION

OREGON DEPARTMENT OF TRANSPORTATION
STRUCTURAL DESIGN SECTION

WILLAMETTE RIVER
(CENTER STREET) BRIDGE

BENT B24 & 25 DETAILS

DATE May, 1982
BRIDGE NO. 123 K
SHEET 127 OF 155
DRAWING NO. 37127

Important Information

About Your Geotechnical Report

IMPORTANT INFORMATION

SHANNON & WILSON
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

Attachment to and part of Report: 105679
Date: May 2021
To: Ho-ping Wei
Black & Veatch, Inc..

Important Information About Your Geotechnical/Environmental Report

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors that were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary, because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports, and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland

Appendix C. Pump Station and Reservoir Seismic Vulnerability Assessment

WATER SYSTEM SEISMIC RESILIENCE STUDY

CITY OF SALEM PUBLIC WORKS DEPARTMENT
SALEM, OREGON

Final Technical Memorandum: Pump Station and Reservoir Seismic Vulnerability Assessment

January 6th, 2023
SEFT Project Number: B20028.00

SEFT Consulting Group
4800 SW Griffith Drive, Suite 100
Beaverton, OR 97005

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1.0 Introduction and Background

1.1 City of Salem Water System Description

The City of Salem relies on the North Santiam River Watershed (including the North Santiam River and Detroit Lake) to supply water for the City's approximately 170,000 residents and commercial customers. Water flows down the North Santiam River to the raw water intake at the Geren Island Water Treatment Facility near Stayton. Large diameter transmission mains deliver water from Geren Island to the 100-million-gallon Franzen Reservoir located in Turner and/or the City's transmission and distribution system that is supported by numerous pump stations and storage reservoirs within and adjacent to the City of Salem service area. The City also operates four aquifer storage and recovery (ASR) wells in Woodmansee Park.

1.2 Seismic Resilience Study

Based on Oregon Health Authority requirements, the City of Salem has retained a team, led by Black & Veatch, to perform a water system seismic resilience study. This study has established post-earthquake level of service goals for the City's water system following a Magnitude 9.0 (M9.0) Cascadia Subduction Zone (CSZ) earthquake, identified a water system backbone, evaluated the expected performance of selected City water system components following an M9.0 CSZ earthquake, and identified preliminary recommendations for improvements that should be implemented to enable the City to more rapidly restore water service after a major earthquake, and to meet community social and economic needs.

This Technical Memorandum (TM) presents SEFT's observations, findings, and recommendations related to a preliminary structural and nonstructural seismic assessment of selected City of Salem water system facilities (10 pump stations, Turner Control Facility, 8 reservoirs, and 5 reservoir control buildings). The components of the water system that have been evaluated by SEFT as part of this effort are summarized in Table 1.1 (pump stations and control facilities), Table 1.2 (reservoirs), and Table 1.3 (reservoir control buildings). The locations of these components are illustrated in Figure 1.1. To complete this scope of work, SEFT utilized the available original design drawings, seismic retrofit drawings, and previous reports indicated in Table 1.4 (pump stations and control facilities) and Table 1.5 (reservoirs and reservoir control buildings), that were provided to the Black & Veatch team by the City.

Table 1.1 Summary of Evaluated Pump Stations and Control Facilities

Pump Station or Control Building	**Construction Type**	**Year of Original Construction**	**Year(s) of Modification or Retrofit**
ASR #1 and #2	Reinforced Masonry Shear Wall	1995	1998
ASR #4	Reinforced Masonry Shear Wall	1998	--
ASR #5	Reinforced Masonry Shear Wall with Octagonal Steel Framed Pavilion	1998	--
Boone Road (original)	Reinforced Masonry Shear Wall	1976	2018
Creekside	Reinforced Masonry Shear Wall	1998	--
Deer Park	Reinforced Masonry Shear Wall	Unknown	Unknown(1) & 2013
Edwards	Masonry Shear Wall and Steel Frame		--
Limelight	Reinforced Masonry Shear Wall	1998	--
Mountain View	Reinforced Masonry Shear Wall	1994	--
Salem/Keizer Intertie #1	Reinforced Masonry Shear Wall	2012	--
Turner Control Facility	Reinforced Masonry Shear Wall (above-grade) and Reinforced Concrete Shear Wall (below-grade)	2007(2)	--

(1) An electrical room addition was constructed abutting to the south side of the original Deer Park Pump Station at an unknown date. This addition approximately doubled the size of the pump station.
(2) The original Turner Control Facility was substantially replaced by the 2007 construction. However, a small subgrade portion of the original Turner Control Facility was integrated into the new structure.

Table 1.2 Summary of Evaluated Reservoirs

Reservoir	**Construction Type**	**Year of Original Construction**	**Year(s) of Modification or Retrofit**
Candalaria	0.5 MG(1) Rectangular Reinforced Concrete	1940	2006
Champion Hill	2.2 MG Strand-Wound Circular Prestressed Concrete	2005	--
Eola Reservoir #1B	0.86 MG Circular Reinforced Concrete	1999	--
Fairmount	10 MG Rectangular Reinforced Concrete	1936	--
Grice Hill	2.2 MG Strand-Wound Circular Prestressed Concrete	2001	--
Lone Oak	5.6 MG Strand-Wound Circular Prestressed Concrete	2003	--
Mill Creek #1	2.2 MG Strand-Wound Circular Prestressed Concrete	2013	--
Mountain View	10 MG Strand-Wound Circular Prestressed Concrete	1971	--

(1) million gallon (MG)

Table 1.3 Summary of Evaluated Reservoir Control Buildings

Reservoir Control Building	Construction Type	Year of Original Construction	Year(s) of Modification or Retrofit
Champion Hill	Reinforced Masonry Shear Wall (above-grade) and Reinforced Concrete Shear Wall (below-grade)	2005	--
Fairmount	Reinforced Concrete Shear Wall	1936	--
Grice Hill	Reinforced Masonry Shear Wall	2001	--
Lone Oak	Reinforced Masonry Shear Wall (above-grade) and Reinforced Concrete Shear Wall (below-grade)	2003	--
Mill Creek #1	Reinforced Masonry Shear Wall (above-grade) and Reinforced Concrete Shear Wall (below-grade)	2013	--

Table 1.4 Available Pump Station and Control Facility Documents

Pump Station or Control Building	Design Drawing, As-Built Drawing or Evaluation Report	Date
ASR #1 and #2	"Aquifer Storage & Recovery Project" by Stettler Company	April 1995
	"Aquifer Storage & Recovery Well No. 2" by Stettler Company	November 1997
ASR #4	"Aquifer Storage & Recovery Well No. 4" by Stettler Company	February 1998
ASR #5	"Aquifer Storage & Recovery Well No. 5" by Stettler Company	November 1997
Boone Road	"Boone Road Pump Station" by C & G Engineering	August 1976
	"Boone Road Water Pump Station Upgrades" by Murraysmith	September 2018
Creekside	"Creekside S-3 Pump Station" by Multi/Tech Consultants	September 1997
Deer Park	"Deer Park Pump Station Improvements" by Landis Consulting	January 2013
Edwards	"Intermediate Level Booster Pumps and Piping Edwards Pump Station" by Clark & Groff Engineers Inc.	January 1966
Limelight	"Limelight Pump Station" by Multi/Tech Consultants	March 1997
Mountain View	"Mt. View Pump Station for the City of Salem" by KMC, Inc.	January 1994
Salem/Keizer Intertie #1	"Keizer Intertie (Cherry Ave. N) Water Booster Pump Station" by Westech Engineering, Inc.	June 2012
Turner Control Facility	"75 MGD Transmission Conduit Phase 2 Delaney Road to Turner Control" by Black & Veatch Corporation	October 2007

Table 1.5 Available Reservoir and Reservoir Control Building Documents

Reservoir	Design Drawing, As-Built Drawing or Evaluation Report	Date
Candalaria	"Proposed Candalaria Reservoir" by R.D. Cooper	May 1940
	"Salem Concrete Reservoirs (Candalaria, Chacarun, Glen Creek and Skyline) Seismic Retrofit Project" by Black & Veatch Corporation	January 2006
	"City of Salem's 0.5 Million Gallon Candalaria Reservoir Evaluation" by Murray, Smith & Associates, Inc.	August 2011
Champion Hill	"2.2 Million Gallon Champion Hill Reservoir" by Westech Engineering	August 2005
Eola Reservoir #1B	"Eola 1B Water Reservoir" by Multi/Tech Consultants	May 1999
Fairmount	"Fairmount Reservoir" by Stevens & Koon	April 1936
	"Fairmount Reservoir Seismic Evaluation" by Black & Veatch Corporation	April 2007
	"Fairmount Reservoir Structural Evaluation" by Carollo Engineers	April 2018
Grice Hill	"Grice Hill Reservoir & Waterline Extension" by Westech Engineering	May 2001
Lone Oak	"5.6 Million Gallon Lone Oak Reservoir" by CH2M Hill	July 2003
Mill Creek #1	"Mill Creek Reservoir As-Built Drawings" by Westech Engineering	December 2014
Mountain View	"Mountain View Reservoir" by Stevens, Thomsen & Runyan Inc.	May 1971

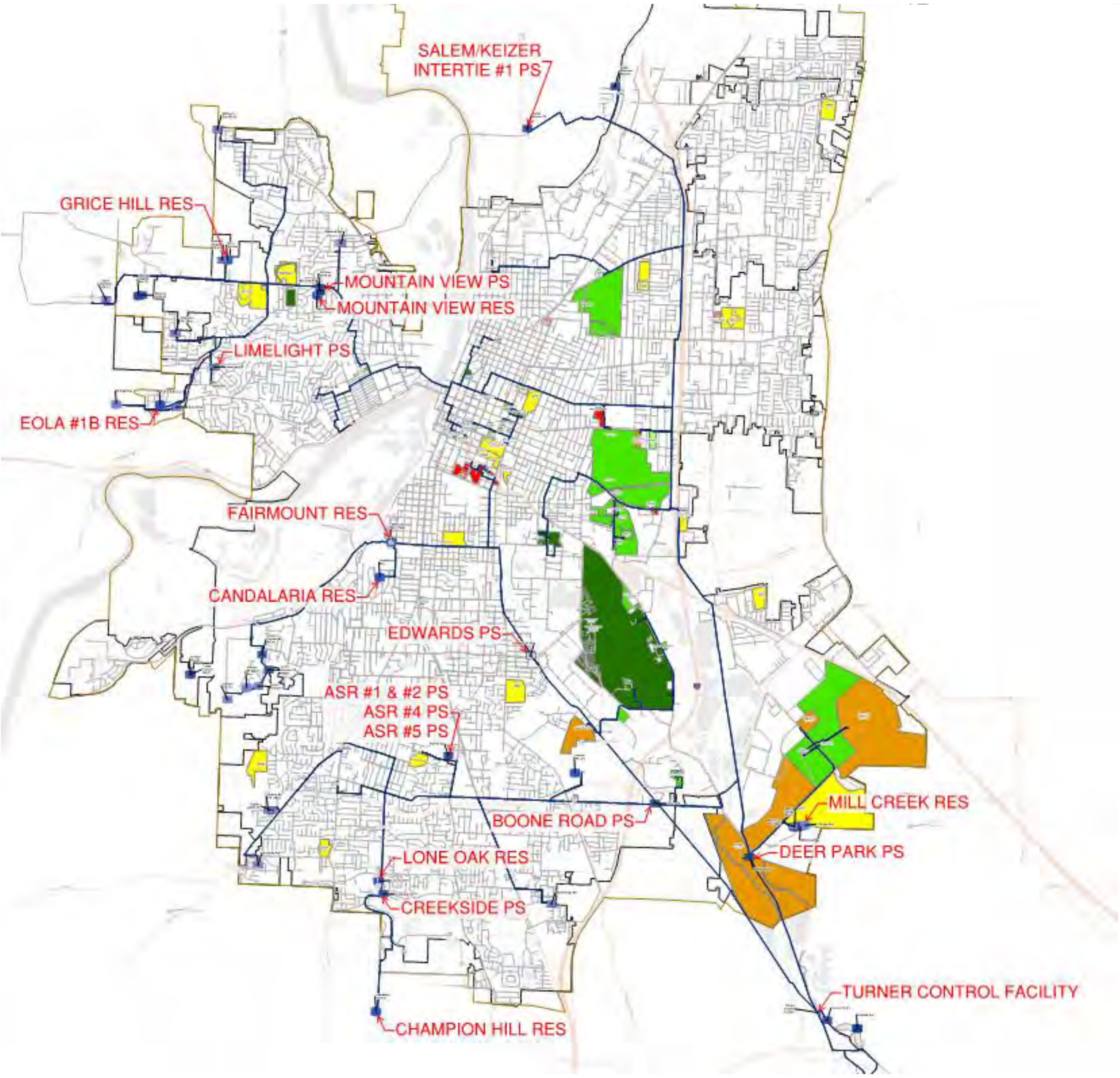


Figure 1.1 City of Salem Water System General Location Map

2.0 Evaluation Methodology and Seismic Performance Objectives

2.1 Seismic Hazard

This evaluation considered a single seismic hazard level associated with a Magnitude 9.0 (M9.0) scenario earthquake originating on the Cascadia Subduction Zone (CSZ). As part of this project, Shannon and Wilson, Inc. conducted a geotechnical seismic hazard assessment (Shannon & Wilson, 2021). In their report, Shannon & Wilson provided estimates of the spectral acceleration and permanent ground deformation (PGD) for liquefaction-induced settlement, liquefaction-induced lateral spreading, and earthquake-induced landslide associated with the M9.0 CSZ scenario earthquake. The geotechnical data that was used as the basis for SEFT's structural evaluation is summarized in Table 2.1 (pump stations and control facilities) and Table 2.2 (reservoirs and reservoir control buildings).

2.2 Seismic Performance Objectives

In the initial phase of this project, the Black & Veatch/SEFT team worked with the City of Salem to establish proposed level of service (LOS) goals for the City of Salem water system following a major earthquake as described in Black & Veatch (2021). The structural and nonstructural performance objectives used for evaluation of water system components for the M9.0 CSZ scenario earthquake were based on the post-earthquake performance of facilities that will be required to achieve these LOS goals (i.e., Immediate Occupancy structural performance and Operational nonstructural performance) and are described in Sections 2.2.1 and 2.2.2. Additionally, this evaluation identified several structures that are not currently expected to achieve Life Safety structural performance (see Section 2.2.1 for definition) for the M9.0 CSZ scenario earthquake and represent a potential safety hazard to City staff and contractors.

2.2.1 Structural Performance Objective

Immediate Occupancy: "Immediate Occupancy" refers to the post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical- and lateral-force-resisting systems of the building retain almost all their pre-earthquake strength and stiffness. The risk of life-threatening injury from structural damage is very low, and although some minor structural repairs might be appropriate, these repairs would generally not be required before re-occupancy. Continued use of the building is not limited by its structural condition but might be limited by damage or disruption to nonstructural elements of the building, furnishings, or equipment and availability of external utility services.

Life Safety: "Life Safety" refers to the post-earthquake damage state in which significant damage to the structure has occurred but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this damage has not resulted in large falling debris hazards, either inside or outside the building. Injuries might occur during the earthquake, however, the overall risk of life-threatening injury as a result of structural damage is expected to be low. It should be possible to repair the structure; however, for economic reasons, this repair might not be practical. Although the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing before re-occupancy.

2.2.2 Nonstructural Performance Objectives

Operational: "Operational" refers to the performance level where most nonstructural systems required for normal use of the building are functional, although minor cleanup and repair of some items might be required. Achieving the Operational nonstructural performance level requires considerations of many elements beyond those that are normally within the sole province of the structural engineer's responsibilities. For Operational nonstructural performance, in addition to ensuring that nonstructural components are properly mounted and braced within the structure, it is often necessary to provide emergency standby equipment to provide utility services from external sources that might be disrupted. It might also be necessary to perform qualification testing to ensure that all necessary equipment will function during or after strong shaking.

2.3 Water System Evaluation Methodology

2.3.1 Pump Stations, Control Facilities, and Control Buildings

The seismic structural evaluations of pump stations, control facilities and reservoir control buildings were completed using the Tier 1 screening procedure of the standard by the American Society of Civil Engineers (ASCE), ASCE 41-17 *Seismic Evaluation and Retrofit of Existing Buildings*. This Tier 1 procedure uses a checklist-based approach to identify potential seismic structural deficiencies that have been commonly observed in past earthquakes. The Tier 1 procedure also uses quick-check calculations to identify potential deficiencies in the primary components of the seismic lateral-force-resisting system.

The seismic nonstructural evaluation of pump stations, control facilities, and reservoir control buildings was completed using the nonstructural seismic evaluation checklists presented in ASCE 41-17 supplemented by the Technical Council on Lifeline Earthquake Engineering (TCLEE) Monograph No. 22 *Seismic Screening Checklists for Water and Wastewater Facilities*. Similar to the ASCE 41 Tier 1 structural evaluation procedure, this checklist-based evaluation approach is used to identify potential seismic nonstructural deficiencies that have been commonly observed in past earthquakes.

2.3.2 Reservoirs

The seismic evaluation approach for the conventionally reinforced concrete reservoirs (Candalaria and Eola #1B Reservoirs) has been adapted from an American Society of Civil Engineering (ASCE) seismic evaluation and retrofit standard, ASCE 41-17 *Seismic Evaluation and Retrofit of Existing Buildings.* This standard provides a tool for identifying potential structural and nonstructural seismic deficiencies. The ASCE 41 Tier 1 screening process uses a quick-check calculation approach with unreduced (no response modification factor, *R*) and non-amplified (no importance factor, *I*) seismic forces. The demand-capacity ratio for seismic force resisting system elements is compared to ASCE 41 specified component modification factors (*m*-factors) to evaluate the acceptability of components of the structure for the Immediate Occupancy structural performance objective. Earthquake-induced hydrodynamic forces were calculated using the procedure outlined in American Concrete Institute (ACI) standard ACI 350.3-06 *Seismic Design of Liquid-Containing Concrete Structures and Commentary* (for Candalaria, Fairmount and Eola #1B Reservoirs), as modified by ASCE 7-16 *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. However, *R* and *I*-factors were set equal to 1.0 for consistency with the ASCE 41 evaluation approach. Consistent with ACI 350.3, soil loads were neglected where they act to decrease the demand on buried portions of reservoir concrete walls.

The approach used for the seismic evaluation of the Fairmount Reservoir was to complete a desktop review of the reservoir structural evaluation performed by Carollo Engineers in 2018 and our observations in the field.

For the five strand-wound, circular, prestressed concrete reservoirs (Champion Hill, Grice Hill, Lone Oak, Mill Creek #1, and Mountain View Reservoirs), a different evaluation approach was used because ASCE 41-17 does not include quick-check evaluations and acceptance criteria that are applicable to this type of reservoir. American Water Works Association (AWWA) standard design checks were performed to evaluate primary components of the seismic load path (roof to wall connection, circumferential strand, and seismic cables connecting the wall to foundation). Earthquake-induced hydrodynamic forces were calculated using the procedure outlined in AWWA D110-13 *Wire- and Strand-Wound, Circular, Prestressed Concrete Water Tanks*, as modified by ASCE 7-16 *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. Consistent with AWWA D110, soil loads were neglected where they act to decrease the demand on buried portions of reservoir concrete walls.

The Mill Creek #1 Reservoir, also a strand-wound, circular, prestressed concrete reservoir, was built in 2013. Since this reservoir is relatively new and was designed per the latest seismic standards, the seismic assessment of this reservoir was conducted based on a desktop review of the reservoir drawings and our observations in the field.

Freeboard calculations were completed based on both the applicable AWWA or ACI design standard, and ASCE 7-16. The conclusions and recommendations of this study have been based on the more conservative of the freeboard estimates calculated using these standards.

The seismic nonstructural evaluation of reservoir components was completed using the nonstructural seismic evaluation checklists presented in ASCE 41-17, supplemented by TCLEE Monograph No. 22. Similar to the ASCE 41 Tier 1 structural evaluation procedure, this checklist-based evaluation approach is used to identify potential seismic nonstructural deficiencies that have been commonly observed in past earthquakes.

Table 2.1 Summary of Mapped Seismic Hazards at Pump Stations and Control Facilities (Source: Shannon & Wilson, 2021)

Pump Station or Control Building	**Short Period Spectral Accel. (g)**	**One-Second Spectral Accel. (g)**	**Liquefaction-Induced Settlement (inches)**	**Liquefaction-Induced Lateral Spreading (inches)**	**Earthquake-Induced Landslide PGD (feet)**
ASR #1 and #2	0.28	0.12	NA	NA	NA
ASR #4	0.28	0.12	NA	NA	NA
ASR #5	0.28	0.12	NA	NA	NA
Boone Road	0.53	0.29	NA	NA	NA
Creekside	0.27	0.12	NA	NA	NA
Deer Park	0.35	0.12	NA	NA	NA
Edwards	0.68	0.32	NA(1)	NA(1)	NA(1)
Limelight	0.33	0.13	NA	NA	NA
Mountain View	0.39	0.12	NA	NA	NA
Salem/Keizer Intertie #1	0.74	0.29	1	NA	NA
Turner Control Facility	0.64	0.28	1	NA	NA

(1) Geologic maps may not adequately capture geohazard, see Shannon & Wilson (2021) for additional information.

Table 2.2 Summary of Mapped Seismic Hazards at Reservoirs and Reservoir Control Buildings

(Source: Shannon & Wilson, 2021)

Reservoir and Control Building	Short Period Spectral Accel. (g)	One-Second Spectral Accel. (g)	Liquefaction-Induced Settlement (inches)	Liquefaction-Induced Lateral Spreading (inches)	Earthquake-Induced Landslide PGD (feet)
Candalaria	0.43	0.12	NA	NA	NA
Champion Hill	0.27	0.12	NA(1)	NA(1)	NA(1)
Eola Reservoir #1B	0.30	0.13	NA	NA	NA
Fairmount	0.46	0.12	NA	NA	NA
Grice Hill	0.30	0.13	NA	NA	NA
Lone Oak	0.27	0.12	NA	NA	NA
Mill Creek #1	0.27	0.12	NA	NA	NA
Mountain View	0.39	0.12	NA	NA	NA

(1) Geologic maps may not adequately capture geohazard, see Shannon & Wilson (2021) for additional information.

3.0 Expected Seismic Structural and Nonstructural Performance

3.1 Pump Stations and Control Facilities

The expected structural and nonstructural seismic performance of selected City water pump stations and control facilities (i.e., Turner Control Facility) has been evaluated for a M9.0 CSZ scenario earthquake. The following sections provide a short narrative description of each pump station or control building evaluated, followed by tables that summarize the potential seismic structural and nonstructural deficiencies identified by the seismic evaluation using the ASCE 41-17 Tier 1 and TCLEE Monograph No. 22 checklist-based procedures. For each pump station or control building, selected images from the design drawings and/or site visit photos are provided to help illustrate the identified potential structural and nonstructural deficiencies.

Site visits to these pump stations and control facilities were conducted by SEFT on May 11th, 14th, 18th, and 25th, 2021. Site observation was limited to those areas readily accessible to view, and did not include any areas concealed by existing finishes, such as ceilings, soffits, etc. Site observation did not include entry into any permit required confined spaces. A detailed structural condition assessment of these structures was not included in the scope of this project.

3.1.1 ASR #1 and #2 Pump Station

The ASR #1 Pump Station structure was built in 1995 at 4635 Sunnyside Road SE. The ASR #2 structure was constructed in 1998 as an addition to the original ASR #1 Pump Station. The ASR #1 and #2 Pump Station structure (see Figure 3.1) is an above-grade, single-story, reinforced masonry shear wall structure with a plywood sheathed wood framed roof. The building is rectangular in plan, with approximate dimensions of 12 feet in the north-south direction by 54 feet in the east-west direction.

This pump station supports Wells #1 and #2 of the City's aquafer storage and recover (ASR) system. Water is drawn from the ASR system during the higher water demand summer season and the aquafer is recharged during the wintertime. One pump supports each of the wells and primarily serve the S2 pressure zone. A chlorination station is currently located within the pump station but will soon be relocated to a new common treatment building (chlorination, de-chlorination, pH adjustment, and corrosion control) that will support all the ASR wells on the site and is currently being constructed adjacent to the ASR #4 Pump Station. Therefore, the chlorination station nonstructural components within the ASR #1 and #2 Pump Station were not included in the scope of this seismic assessment.

This pump station does not have an emergency generator and does not have a pre-wired connection to hook-up a portable generator. The SCADA antenna for the ASR #1 and #2 Pump Station also supports the ASR #5 Pump Station and transmits to the antenna at the ASR #4 Pump Station that functions as a repeater to send information off this ASR site.

Table 3.1 presents a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.1, the ASR #1 and #2 Pump Station is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Similarly, based on the structural deficiencies identified, the ASR #1 and #2 Pump Station is not currently expected to achieve Life Safety structural performance and represents a potential safety hazard to City staff and contractors.

Table 3.1 ASR #1 and #2 Pump Station Evaluation Summary

Potential Deficiencies	Description
Structural	• The original design drawings do not indicate how the masonry walls of the ASR #2 addition were connected to the walls of the original ASR #1 structure. • There is a step in the roof elevation between the ASR#1 and ASR #2 portions of the structure. Based on the original drawings, the load path to transfer seismic forces at this step from the roof diaphragm to the west masonry wall of the original ASR #1 structure is unclear (as it is concealed behind gypsum board). • The roof plywood sheathing to truss nailing schedule was not provided in the available original design drawings. • A positive connection does not appear to be provided between the sloped truss blocking and masonry wall top plate. Therefore, the load path is incomplete to transfer in-plane shear forces from the roof diaphragm to the masonry walls (see Figure 3.2). Additionally, blocking in approximately every third bay has a long vent slot that limits its capacity to transfer seismic forces from the roof diaphragm to the masonry wall. • Out-of-plane bracing for the perimeter and interior masonry walls is not adequate. The roof trusses are attached to the top plate of the perimeter masonry walls with hurricane ties, which are intended to provide capacity to primarily resist uplift, not to resist wall out-of-plane demands (see Figure 3.3). • Adequate cross ties between diaphragm chords are not provided in both directions. • No trim reinforcing is indicated at the sides of door and other openings in the original design drawings.

Table 3.1 ASR #1 and #2 Pump Station Evaluation Summary (cont.)

Potential Deficiencies	Description
Nonstructural	• Water system piping within the pump station is not braced (see Figure 3.4) • Valves in line with the water system piping are not braced (see Figure 3.4). • The air relief valve is not braced and is only supported by rigid small diameter piping (see Figure 3.4). • Vertical pump motors are not braced above the center of gravity of the motor (see Figure 3.5). • There does not appear to be adequate flexibility in the piping that is attached to the pumps to accommodate potential relative movement between the piping and the pump, and to prevent the piping from adding additional load to the pump anchorage. • Electrical cabinets do not appear to be anchored or braced to the wall near the top of the cabinets to prevent them from tipping over during an earthquake. • Light fixtures in the pump station do not include lens covers (see Figure 3.6). • Electrical conduits, hung from the roof and connected to the top of wall-mounted electrical panels, may not have adequate flexibility to account for differential movement between the floor and the pump station roof (see Figure 3.7). • The horizontal arm of the SCADA antenna may not be adequately connected to the supporting pole to prevent the antenna from being misaligned or damaged after an earthquake (see Figure 3.8). • A 4-inch split-face CMU veneer was added to the original ASR #1 structure as part of construction of the ASR #2 addition. Original design drawings do not indicate the use of veneer ties and it was not clear if veneer ties were installed based on field observations of the gap between the original 8-inch CMU walls and 4-inch CMU veneer (see Figure 3.9). • The six architectural concrete pillars around the perimeter of the pump station may not have adequate capacity to resist seismic forces (see Figure 3.10). The number of vertical reinforcing bars is unclear in the original design drawings and no tie reinforcing is indicated. • No anchorage was observed between the electrical transformer and concrete support pad (see Figure 3.11).

Figure 3.1 ASR #1 and #2 Pump Station

Figure 3.2 Inadequate Connection between Blocking and Masonry Wall Top Plate

Figure 3.3 Roof Truss to Masonry Wall Top Plate Connection

Figure 3.4 Unbraced Piping, Valves, and Air Relief Valve

Figure 3.5 Unbraced Pump Motor

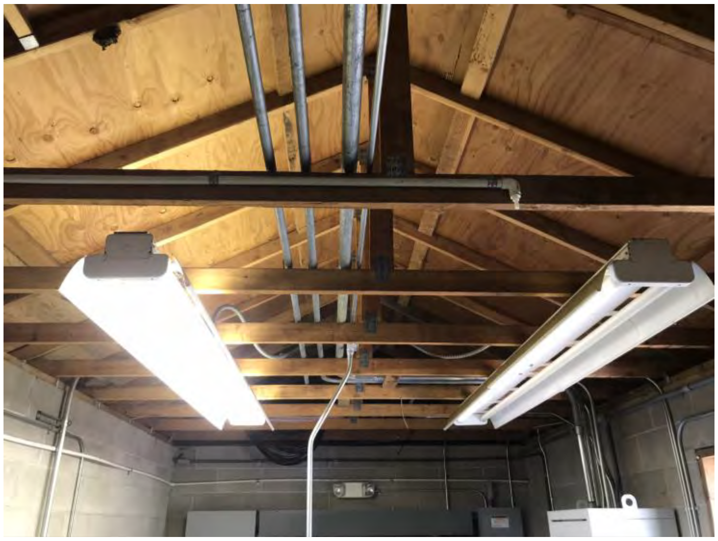


Figure 3.6 Light Fixtures without Lens Covers

Figure 3.7 Conduit Top Connection to Electrical Panels without Flexible Connection

Figure 3.8 SCADA Antenna

Figure 3.9 Unknow Masonry Veneer Ties to Backup Masonry Wall

Figure 3.10 Architectural Concrete Pillar

Figure 3.11 Unanchored Electrical Transformer

3.1.2 ASR #4 Pump Station

The ASR #4 Pump Station structure (see Figure 3.12) was built in 1998 at 4535 Sunnyside Road SE. This structure is an above-grade, single-story, reinforced masonry shear wall structure with a plywood sheathed wood framed roof. The building is rectangular in plan, with approximate dimensions of 12 feet in the north-south direction by 30 feet in the east-west direction. Note that the interior masonry wall between the storage room and chlorination room (as shown on the original design drawings) has been previously removed.

This pump station supports Well #4 of the City's ASR system. Water is drawn from the ASR system during the higher water demand summer season and the aquafer is recharged during the wintertime. One pump supports the well and primarily serve the S2 pressure zone. A chlorination station is currently located within the pump station but will soon be relocated to a new common treatment building (chlorination, de-chlorination, pH adjustment, and corrosion control) that will support all the ASR wells on the site and is currently being constructed adjacent to the ASR #4 Pump Station. Therefore, the chlorination station nonstructural components within the ASR #4 Pump Station were not included in the scope of this seismic assessment.

This pump station does not have an emergency generator or a pre-wired connection to hook-up a portable generator. The SCADA antenna for the ASR #4 Pump Station also functions as a repeater for the ASR # 1 and #2 Pump Station and ASR #5 Pump Station to send information off this ASR site.

Table 3.2 presents a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.2, the ASR #4 Pump Station is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Similarly, based on the structural deficiencies identified, the ASR #4 Pump Station is not currently expected to achieve Life Safety structural performance and represents a potential safety hazard to City staff and contractors.

Table 3.2 ASR #4 Pump Station Evaluation Summary

Potential Deficiencies	Description
Structural	• The roof plywood sheathing nailing schedule was not provided in the available original design drawings. • A positive connection does not appear to be provided between the sloped truss blocking and masonry wall top plate. Therefore, the load path is incomplete to transfer in-plane shear forces from the roof diaphragm to the masonry walls (see Figure 3.13). • The roof access hatch (for pump motor replacement) immediately adjacent to the east masonry wall creates a large opening in the diaphragm near a shear wall. This opening reduces the capacity of the diaphragm to transfer seismic forces to the shear wall below. • Out-of-plane bracing for the perimeter and interior masonry walls is not adequate. The roof trusses are attached to the top plate of the perimeter masonry walls with hurricane ties, which are intended to provide capacity to primarily resist uplift, not to resist wall out-of-plane demands (see Figure 3.14). Additionally, blocking in approximately every third bay has a long vent slot that limits its capacity to transfer seismic forces from the roof diaphragm to the masonry wall. • Adequate cross ties between diaphragm chords are not provided in both directions. • No trim reinforcing is indicated at the sides of door and other openings in the original design drawings.
Nonstructural	• Water system piping within the pump station is not braced (see Figure 3.15). • Valves in line with the water system piping are not braced (see Figure 3.15). • The air relief valve is not braced and is only supported by rigid small diameter piping (see Figure 3.15). • The vertical pump motor is not braced above the center of gravity of the motor (see Figure 3.16). • There does not appear to be adequate flexibility in the piping that is attached to the pump to accommodate potential relative movement between the piping and the pump, and to prevent the piping from adding additional load to the pump anchorage. • The pump station control cabinet does not appear to be anchored to the floor or wall (see Figure 3.17). • Light fixtures in the pump station do not include lens covers (see Figure 3.18). • No anchorage was observed between the electrical transformer and concrete support pad (see Figure 3.19).

Figure 3.12 ASR #4 Pump Station

Figure 3.13 Inadequate Connection between Blocking and Masonry Wall Top Plate

Figure 3.14 Roof Truss to Masonry Wall Top Plate Connection

Figure 3.15 Unbraced Piping, Valves, and Air Relief Valve

Figure 3.16 Unbraced Pump Motor

Figure 3.17 Unanchored Control Cabinet

Figure 3.18 Light Fixtures without Lens Covers

Figure 3.19 Unanchored Electrical Transformer

3.1.3 ASR #5 Pump Station

The ASR #5 Pump Station structure (see Figure 3.20) was built in 1998 at 4615 Sunnyside Road SE. The pump station equipment is housed in an above-grade, single-story, reinforced masonry shear wall structure with a plywood ceiling diaphragm. This pump station structure is trapezoidal in plan, with approximate overall dimensions of 40 feet in north-south direction and 12 feet in east-west direction. This masonry shear wall structure is integrated with a premanufactured, hexagonal steel framed pavilion that is used by visitors to Woodmansee Park. The City of Salem Parks Department uses the room at the south end of the pump station structure for storage. This room was not accessible during SEFT's site visit.

This pump station supports Well #5 of the City's aquafer storage and recover (ASR) system. Water is drawn from the ASR system during the higher water demand summer season and the aquafer is recharged during the wintertime. One pump supports the well and primarily serve the S2 pressure zone. A chlorination station is currently located within the pump station but will soon be relocated to a new common treatment building (chlorination, de-chlorination, pH adjustment, and corrosion control) that will support all the ASR wells on the site and is currently being constructed adjacent to the ASR #4 Pump Station. Therefore, the chlorination station nonstructural components within the ASR #5 Pump Station were not included in the scope of this seismic assessment.

This pump station does not have an emergency generator or a pre-wired connection to hook-up a portable generator. SCADA data from the ASR #5 Pump Station is transmitted by buried cable to the ASR #1 and #2 Pump Station which then transmits to the antenna at the ASR #4 Pump Station that functions as a repeater to send information off this ASR site.

Table 3.3 presents a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.3, the ASR #5 Pump Station is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Similarly, based on the structural deficiencies identified, the ASR #5 Pump Station is not currently expected to achieve Life Safety structural performance and represents a potential safety hazard to City staff and contractors. Note that the ASCE 41-17 Tier 1 procedure does not include checklists for the unique steel frames pavilion portion of the ASR #5 Pump Station structure. The interaction of the steel framed pavilion with the masonry shear wall structure below should be further investigated as part of a future detailed evaluation and seismic retrofit project.

Table 3.3 ASR #5 Pump Station Evaluation Summary

Potential Deficiencies	Description
Structural	• There does not appear to be either an adequate load path to transfer the seismic forces generated by the steel framed pavilion to the masonry shear wall structure or an adequate seismic separation to prevent unintended interaction between the steel framed pavilion and masonry shear wall structure. • The horizontal span for the ceiling diaphragm in the north-south direction is greater than the ASCE 41-17 Tier 1 limit for unblocked wood structural panel diaphragms if the interior masonry walls are not engaged as part of the seismic force resisting system. • The ceiling plywood sheathing nailing schedule was not provided in the available original design drawings. • Based on the original design drawings, it is unclear if blocking is provided between the ceiling sheathing and masonry wall top plate (see Figure 3.21). Therefore, the load path is potentially incomplete to transfer in-plane shear forces from the ceiling diaphragm to the masonry walls. • Out-of-plane bracing for the perimeter and interior masonry walls is not adequate. The ceiling joists are attached to the top plate of the perimeter masonry walls with hurricane ties, which are intended to provide capacity to primarily resist uplift, not resist wall out-of-plane demands (see Figure 3.21). • Adequate cross ties between diaphragm chords are not provided in both directions. • No vertical trim reinforcing is indicated at the sides of door and other openings in the original design drawings. • The free-standing masonry wall to the north of the pump station is not braced (see Figure 3.22). • Corrosion damage was observed at the base of the northern-most steel tube section columns of the pavilion (see Figure 3.23). If this corrosion damage is not adequately addressed, the seismic performance of the steel framed pavilion may be compromised.

Table 3.3 ASR #5 Pump Station Evaluation Summary (cont.)

Potential Deficiencies	Description
Nonstructural	• Water system piping within the pump station is not braced (see Figure 3.24) • Valves in line with the water system piping are not braced (see Figure 3.24). • The air relief valve is not braced and is only supported by rigid small diameter piping (see Figure 3.25). • The vertical pump motor is not braced above the center of gravity of the motor (see Figure 3.26). • There does not appear to be adequate flexibility in the piping that is attached to the pump to accommodate potential relative movement between the piping and the pump, and to prevent the piping from adding additional load to the pump anchorage. • Electrical conduits, hung from the roof and connected to the top of floor- and wall-mounted electrical panels and cabinets, may not have adequate flexibility to account for differential movement between the floor and the pump station roof (see Figure 3.27). • The pump station controls cabinet does not appear to be anchored to the floor or wall (see Figure 3.28). • Light fixtures in the pump station do not include lens covers (see Figure 3.29). • No anchorage was observed between the electrical transformer and concrete support pad (see Figure 3.30).

Figure 3.20 ASR #5 Pump Station

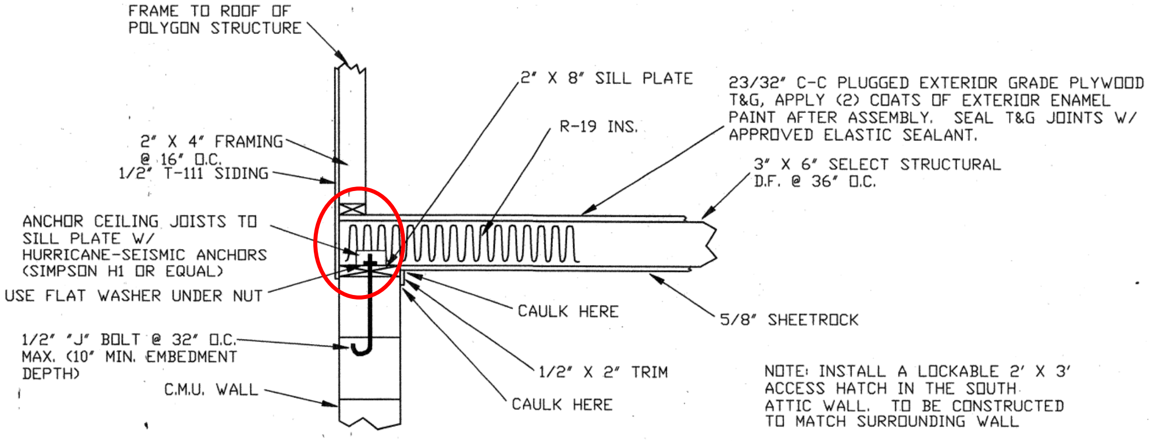


Figure 3.21 Inadequate Blocking and Masonry Wall Out-of-Plane Anchorage (Source: Detail 5 on Sheet A3 of 1997 design drawings by Stettler Company)

Figure 3.22 Free-Standing Masonry Wall without Bracing

Figure 3.23 Corrosion Damage at Northern-Most Pavilion Steel Column

Figure 3.24 Unbraced Piping, Valves, and Air Relief Valve

Figure 3.25 Unbraced Air Relief Valve

Figure 3.26 Unbraced Pump Motor

Figure 3.27 Conduit Top Connection to Motor Control Center without Flexible Connection

Figure 3.28 Unanchored Control Cabinet

Figure 3.29 Light Fixtures without Lens Covers

Figure 3.30 Unanchored Electrical Transformer

3.1.4 Boone Road Pump Station

The original Boone Road Pump Station structure (see Figure 3.31 and Figure 3.32) was built in 1977 at 3351 Boone Rd SE. This structure is an above-grade, single-story, reinforced masonry shear wall structure with a straight-sheathed wood framed roof. The building is rectangular in plan, with approximate dimensions of 34 feet in the north-south direction by 36 feet in the east-west direction. The north and south gable end walls are offset from the masonry shear walls below, creating a step in the roof diaphragm. As part of a recent expansion project at the Boone Road Pump Station site, the original Boone Road Pump Station structure underwent a partial seismic retrofit. The roof to wall connections were strengthened with a combination of steel brackets installed between the straight-sheathed roof decking and masonry wall, and screws were added between the roof decking and masonry wall top plate.

A new electrical building that serves the pump station was recently constructed to the west of the original Boone Road Pump Station (see Figure 3.31). This recently constructed electrical building and the recently installed emergency generator, fuel tank, surge tank, and electrical utility owned transformer were excluded from the scope of this seismic assessment.

The Boone Road Pump Station currently houses three pumps that deliver water from the G0 Level to the S2 Level. Both the pump station and electrical building have capacity to support a future expansion to deliver water to the S1 Level. Note that the S2 Level pumps at Edwards Pump Station serve to supplement the Boone Road Pump Station.

Table 3.4 presents a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.4, the Boone Road Pump Station is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Similarly, based on the structural deficiencies identified, the Boone Road Pump Station is not currently expected to achieve Life Safety structural performance and represents a potential safety hazard to City staff and contractors.

Table 3.4 Boone Road Pump Station Evaluation Summary

Potential Deficiencies	Description
Structural	• The roof configuration results in a partial height gable end that is offset from the masonry shear walls on the north and south sides of the building. These gable ends consist of plywood sheathing over the end glulam trusses, but the exact framing details and sheathing nail schedule are not clear in the available original design drawings. The load path may not be adequate to deliver seismic forces from the upper roof through these gable end walls and into the lower roof. • The roof diaphragm span and aspect ratio exceed the ASCE 41-17 Tier 1 limits for straight-sheathed diaphragms. • Adequate cross ties between diaphragm chords are not provided in both directions. The tension rod cross ties between diaphragm chords in the east-west direction do not provide adequate capacity to resist compressive cross tie forces.
Nonstructural	• Water system piping within the pump station is not braced (see Figure 3.33) • Valves in line with the water system piping are not braced (see Figure 3.33). • Vertical pump motors are not braced above the center of gravity of the motor (see Figure 3.34). • There does not appear to be adequate flexibility in the piping that is attached to the pumps to accommodate potential relative movement between the piping and the pump, and to prevent the piping from adding additional load to the pump anchorage. • The cable tray does not appear to have adequate longitudinal bracing (see Figure 3.35). In some locations the anchors for the transverse bracing appear to be improperly installed in a masonry head joint (see Figure 3.36). • Light fixtures in the pump station do not include lens covers (see Figure 3.37). • Metal floor grating lacks clip connecting the grating to the steel support framing (see Figure 3.38). • The horizontal arm of the SCADA antenna may not be adequately connected to the supporting pole to prevent the antenna from being misaligned or damaged after an earthquake (see Figure 3.39).

Figure 3.31 Boone Road Pump Station (right) and Electrical Building (left)

Figure 3.32 Boone Road Pump Station

Figure 3.33 Unbraced Piping and Valves

Figure 3.34 Unbraced Pump Motor

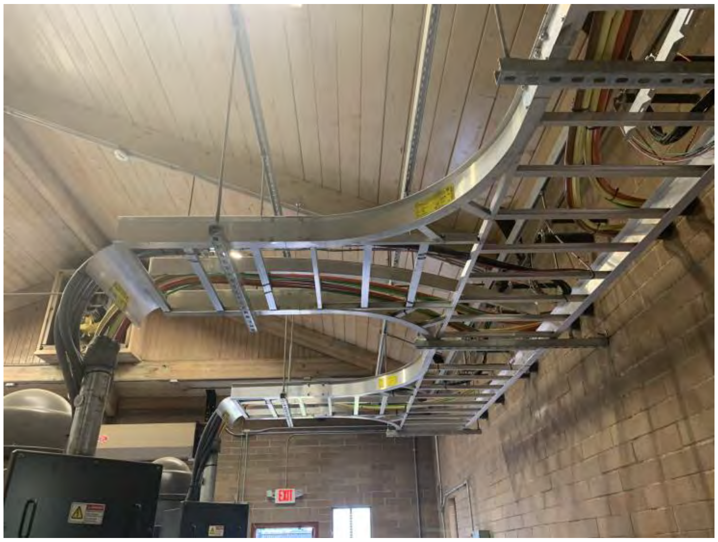
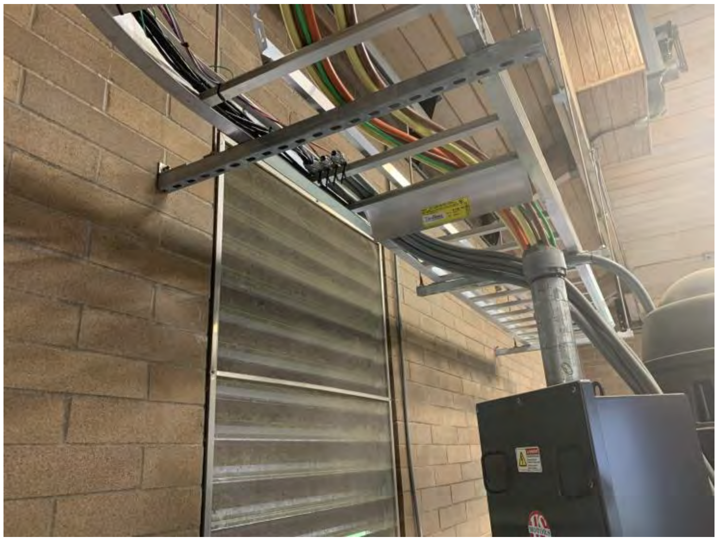


Figure 3.35 Cable Tray Lacks Longitudinal Bracing



(a) Overall View

(b) Close-up View

Figure 3.36 Cable Tray Transverse Brace with Anchor Installed in Masonry Head Joint

Figure 3.37 Light Fixtures without Lens Covers

Figure 3.38 Grating without Clip Connection to Supporting Steel Framing

Figure 3.39 SCADA Antenna

3.1.5 Creekside Pump Station

The Creekside Pump Station structure (see Figure 3.40) was built in 1998 at 6025 Lone Oak Road SE. This structure is an above-grade, single-story, reinforced masonry shear wall structure with a plywood sheathed wood framed roof. The building is rectangular in plan, with approximate dimensions of 20 feet in the north-south direction by 47 feet in the east-west direction.

The Creekside Pump Station houses three pumps that deliver water from the S2 Level to the Champion Hill Reservoir (S3 Level). There is a terraced retaining wall to the north of the pump station that was excluded from the scope of this seismic assessment.

Table 3.5 presents a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.5, the Creekside Pump Station is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Similarly, based on the structural deficiencies identified, the Creekside Pump Station is not currently expected to achieve Life Safety structural performance and represents a potential safety hazard to City staff and contractors.

Table 3.5 Creekside Pump Station Evaluation Summary

Potential Deficiencies	Description
Structural	• The roof plywood sheathing nailing schedule was not provided in the available original design drawings. • The roof diaphragm span for east-west oriented seismic forces exceeds the ASCE 41-17 Tier 1 limit for unblocked wood structural panel diaphragms. • The available original design drawings do not indicate that blocking was installed between the roof sheathing and masonry wall top plate in the bays between wood trusses. During the site visit, this area was blocked from view by soffit panels from the exterior and insulation in the attic space. Even if there is blocking installed, based on observation of similar construction, it is likely that blocking connections to the sheathing or masonry wall top plate are deficient. Therefore, the load path is likely inadequate to transfer in-plane shear forces from the roof diaphragm to the masonry walls. • The available original design drawings do not indicate how the roof diaphragm is connected to the gable end masonry walls. Therefore, the load path is potentially incomplete to transfer in-plane shear forces from the roof diaphragm to the masonry walls.

Table 3.5 Creekside Pump Station Evaluation Summary (cont.)

Potential Deficiencies	Description
Structural (cont.)	• Out-of-plane bracing of perimeter and interior masonry walls is not adequate. The roof trusses do not appear to be attached to the top plate of the perimeter masonry walls with any connection hardware intended to resist wall out-of-plane demands (see Figure 3.41). • Adequate cross ties between diaphragm chords are not provided in both directions.
Nonstructural	• Water system piping within the pump station is not braced (see Figure 3.42). • Valves in line with the water system piping are not braced (see Figure 3.42). • Air relief valves are not braced and are only supported by rigid small diameter piping (see Figure 3.43). • Vertical pump motors are not braced above the center of gravity of the motor (see Figure 3.44). • There does not appear to be adequate flexibility in the piping that is attached to the pumps to accommodate potential relative movement between the piping and the pump, and to prevent the piping from adding additional load to the pump anchorage. • Anchorage of electrical cabinets to the housekeeping pads was not visible from the outside of the cabinets and may not be adequate. Details for how the housekeeping pad is reinforced and connected to the slab on grade are not provided in the available original design drawings and may not be adequate. Additionally, electrical cabinets do not appear to be anchored or braced to the wall near the top of the cabinets to prevent them from tipping over during an earthquake. • The emergency generator air intake support frame, muffler, and exhaust pipe do not appear to be adequately anchored/braced (see Figure 3.45, Figure 3.46, and Figure 3.47). • No anchorage was observed between the electrical transformer and concrete support pad (see Figure 3.48).

Figure 3.40 Creekside Pump Station

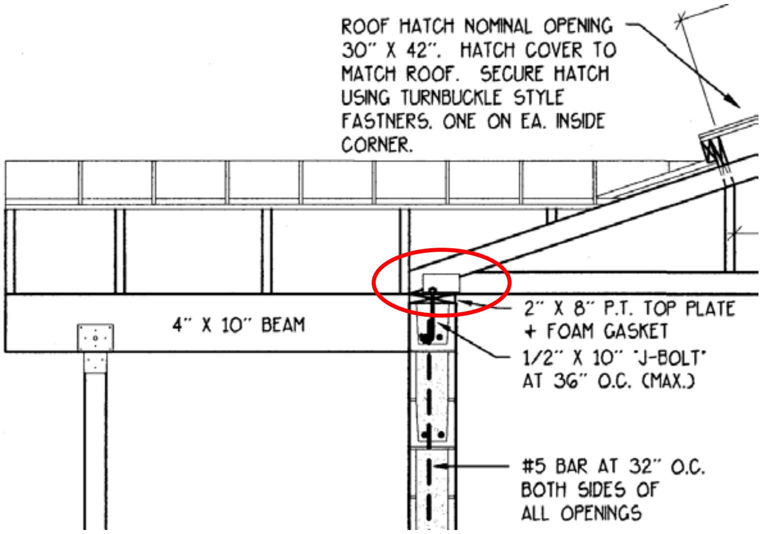


Figure 3.41 Inadequate Wall Out-of-Plane Anchorage Connection
(Source: Section A-A on Sheet A 2.3 of 1997 design drawings by Multi/Tech Consultants)

Figure 3.42 Unbraced Piping and Valves

Figure 3.43 Unbraced Air Relief Valves

Figure 3.44 Unbraced Pump Motor

Figure 3.45 Unanchored Emergency Generator Air Intake Support Frame

Figure 3.46 Unbraced Emergency Generator Muffler

Figure 3.47 Unbraced Emergency Generator Exhaust Pipe

Figure 3.48 Unanchored Electrical Transformer

3.1.6 Deer Park Pump Station

The Deer Park Pump Station structure (see Figure 3.49) was built in 1982 at 5475 Turner Rd SE, with an electrical room addition built between 2008 and 2010 immediately to the south of the original pump station structure. Roll-up door installation and associated modifications were constructed in 2013. The design drawings for the original pump station building and the electrical room addition were not available for review as part of this assessment. Based on site visit observations and the 2013 modification drawings, this structure is an above-grade, single-story, reinforced masonry shear wall structure with a plywood sheathed wood framed roof. The building is rectangular in plan, with approximate dimensions of 44 feet in the north-south direction by 20 feet in the east-west direction.

The Deer Park Pump Station houses three pumps that deliver water from the City's main transmission line (G0 Level) to the Mill Creek Reservoir (S1 Level). Note that the S1 Level pumps at Edwards Pump Station server as a backup to Deer Park Pump Station.

Table 3.6 presents a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.6, the Deer Park Pump Station is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Similarly, based on the structural deficiencies identified, the Deer Park Pump Station is not currently expected to achieve Life Safety structural performance and represents a potential safety hazard to City staff and contractors.

Table 3.6 Deer Park Pump Station Evaluation Summary

Potential Deficiencies	Description
Structural	• No design drawings were available for the original construction of the pump station or the electrical room addition. The size, spacing, and detailing of the steel reinforcing for the masonry walls is unknown. Additionally, it is unknown how the masonry walls from the electrical room addition are connected to the walls of the original pump station. • The roof plywood sheathing to truss nail size and spacing are unknown. • The original south wall of the pump station (now an interior wall between the pump and electrical rooms) is not adequately engaged to resist seismic forces from the roof diaphragm (see Figure 3.50). Without engaging this interior wall, the roof diaphragm span for east-west oriented seismic forces exceeds the ASCE 41-17 Tier 1 limit for unblocked wood structural panel diaphragms. • A positive connection does not appear to be provided between the sloped truss blocking and masonry wall top plate in the original pump station (see Figure 3.51). In the electrical room addition, the view of the area where blocking would be installed was obstructed by insulation. Even if there is blocking installed, based on observation of similar construction, it is likely that blocking connections to the sheathing or masonry wall top plate are deficient. Therefore, the load path is likely inadequate to transfer in-plane shear forces from the roof diaphragm to the masonry walls. • The details of how the roof diaphragm is connected to the south gable end masonry wall are unknown. Therefore, the load path is potentially not adequate to transfer in-plane shear forces from the roof diaphragm to the masonry walls. • Out-of-plane bracing of perimeter and interior masonry walls is not adequate. The roof trusses are not attached to the top plate of the perimeter masonry walls with any metal connector hardware that is designed to resist wall out-of-plane demands (see Figure 3.52). • Adequate cross ties between diaphragm chords are not provided in both directions.

Table 3.6 Deer Park Pump Station Evaluation Summary (cont.)

Potential Deficiencies	Description
Nonstructural	• Water system piping within the pump station is not braced (see Figure 3.53) • Valves in line with the water system piping are not braced (see Figure 3.53). • It is unknown if adequate dowels are provided between the pump support concrete pedestal and the floor slab to resist the expected shear and overturning demands. • There does not appear to be adequate flexibility in the piping that is attached to the pumps to accommodate potential relative movement between the piping and the pump, and to prevent the piping from adding additional load to the pump anchorage. • The pipe support stanchion base plates are missing anchors into the concrete slab (see Figure 3.54). • Anchorage of electrical cabinets to the concrete slab on grade was not visible from the outside of the cabinets and may not be adequate. Additionally, electrical cabinets do not appear to be anchored or braced to the wall near the top of the cabinets to prevent them from tipping over during an earthquake (see Figure 3.55). • The emergency generator starter batteries may not be adequately restrained. A restrainer bracket (similar to the one that would be expected to be installed for the emergency generator starter batteries) was observed inside the pump station (see Figure 3.56). • The horizontal arm of the SCADA antenna may not be adequately connected to the supporting pole to prevent the antenna from being misaligned or damaged after an earthquake (see Figure 3.57). • The wood pole and anchorage of the pole-mounted transformer to the pole may not be adequately designed to resist the seismic forces generated by the transformer

Figure 3.49 Deer Park Pump Station

Figure 3.50 Sheathing Not Connected to Top Plate of Interior Masonry Wall

Figure 3.51 Inadequate Connection between Blocking and Masonry Wall Top Plate

Figure 3.52 Inadequate Connection between Truss Chord and Masonry Wall Top Plate

Figure 3.53 Unbraced Piping and Valves

Figure 3.54 Pipe Support Stanchion Missing Anchors into Floor Slab

Figure 3.55 Electrical Cabinet with Unknown Anchorage Details

Figure 3.56 Emergency Generator Starter Battery Restraint Bracket Observed in Pump Station

Figure 3.57 SCADA Antenna

3.1.7 Edwards Pump Station

The Edwards Pump Station structure (see Figure 3.58) was built in 1961 at Edward Dr SE, with intermediate level pump and piping modification completed in 1966. The design drawings for the original pump station building (i.e., 1961 construction) were not available for review as part of this assessment. Based on site observations and the 1966 modification drawings, this structure is an above-grade, single-story structure with a straight-sheathed wood framed roof. The lateral-force-resisting-system consists of Structural Clay Research (SCR) brick shear walls at the perimeter of the building and built-up steel frames in the east-west direction in the main (S2 Level) pump room area. Roof straight-sheathing is supported by a combination of steel frames, wood framing, and masonry walls. The building is L-shaped in plan, with approximate overall dimensions of 51 feet in the north-south direction by 39 feet in the east-west direction.

The Edwards Pump Station houses three pumps that deliver water from the City's main transmission line (G0 Level) to the S1 Level and three additional pumps that deliver water to the S2 Level. Note that the S1 Level pumps at Edwards Pump Station serve as a backup to Deer Park Pump Station and the S2 Level pumps at Edwards Pump Station serve to supplement the Boone Road Pump Station.

Table 3.7 presents a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.7, the Edwards Pump Station is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Similarly, based on the structural deficiencies identified, the Edwards Pump Station is not currently expected to achieve Life Safety structural performance and represents a potential safety hazard to City staff and contractors.

Table 3.7 Edwards Pump Station Evaluation Summary

Potential Deficiencies	Description
Structural	• Per the Technical Memorandum "Seismic Geohazard Evaluation Report, City of Salem Seismic Resilience Study" by Shannon & Wilson, Inc., there is evidence of soil settlement resulting from past uncontrolled water releases at the pump stations and uncertainty associated with the liquefaction potential of the soil in the area around the pump station. • Cracking of the masonry wall was observed at the southwest corner of the building near the base of the wall (see Figure 3.59).

Table 3.7 Edwards Pump Station Evaluation Summary (cont.)

Potential Deficiencies	Description
Structural (cont.)	• No design drawings were available for the original construction of the pump station. It has been assumed that the SCR brick walls are unreinforced. Additionally, member sizes and connection details are unknown for the roof straight-sheathing, wood framing, and steel frames in the main pump room. The load path may be incomplete or inadequate to transfer seismic forces from the roof diaphragm to the masonry walls and/or steel frames. • The shear stress in the masonry walls exceeds the ASCE 41-17 Tier 1 limit for unreinforced masonry. • The height-to-thickness ratio for the masonry walls exceeds the ASCE 41-17 Tier 1 limit for unreinforced masonry. • The flexural stress in the steel moment frame beams exceeds the ASCE 41-17 Tier 1 limit. • The story drift ratio for the steel moment frames exceeds the ASCE 41-17 Tier 1 limit. • The roof diaphragm spans and aspect ratios exceed the ASCE 41-17 Tier 1 limit for straight-sheathed diaphragms. • Adequate cross ties between diaphragm chords are not provided in both directions. • Independent secondary columns are not provided for all roof framing that is supported by unreinforced masonry walls/pilasters (see Figure 3.60).
Nonstructural	• Water system piping that penetrates through the pump station floor may not have adequate flexibility to accommodate the potential differential movement between the pump station and the surrounding soil at the pipe penetration. • Water system piping within the pump station is not braced (see Figure 3.61) • Valves in line with the water system piping are not braced (see Figure 3.61). • Vertical pump motors are not braced above the center of gravity of the motor (see Figure 3.62). • It is unknown if adequate dowels are provided between the pump support concrete pedestal and the floor slab to resist the expected shear and overturning demands. • There does not appear to be adequate flexibility in the piping that is attached to the pumps to accommodate potential relative movement between the piping and the pump, and to prevent the piping from adding additional load to the pump anchorage.

Table 3.7 Edwards Pump Station Evaluation Summary (cont.)

Potential Deficiencies	Description
Nonstructural (cont.)	• Anchorage of electrical cabinets to the housekeeping pads was not visible from the outside of the cabinets and may not be adequate. Details for how the housekeeping pad is reinforced and connected to the slab on grade are not provided in the available original design drawings and may not be adequate. Additionally, electrical cabinets do not appear to be anchored or braced to the wall near the top of the cabinets to prevent them from tipping over during an earthquake (see Figure 3.63). • Electrical conduits, hung from the roof and connected to the top of floor- and wall-mounted electrical panels and cabinets, may not have adequate flexibility to account for differential movement between the floor and the pump station roof (see Figure 3.64). • The emergency generator starter batteries may not be adequately restrained. • Pendant lights in the pump station do not include lens covers (see Figure 3.65). • The antenna may not be adequately braced to prevent the antenna from being misaligned or damaged after an earthquake (see Figure 3.66). • The HVAC condenser unit is not anchored to the concrete pad (see Figure 3.67). • Metal floor grating lacks clip connecting the grating to the steel support framing (see Figure 3.68). • The overhead bridge crane is not laterally braced and may damage other equipment during an earthquake (see Figure 3.69). • The rolling lifts are unrestrained and may potentially damage the piping and valves during an earthquake (see Figure 3.70). • The ladders are unrestrained and may topple into and potentially damage the piping and valves during an earthquake (see Figure 3.70). • The wood pole and anchorage of the pole-mounted transformer to the pole may not be adequately designed to resist the seismic forces generated by the transformer (see Figure 3.71).

Figure 3.58 Edwards Pump Station

Figure 3.59 Cracking of Masonry Wall

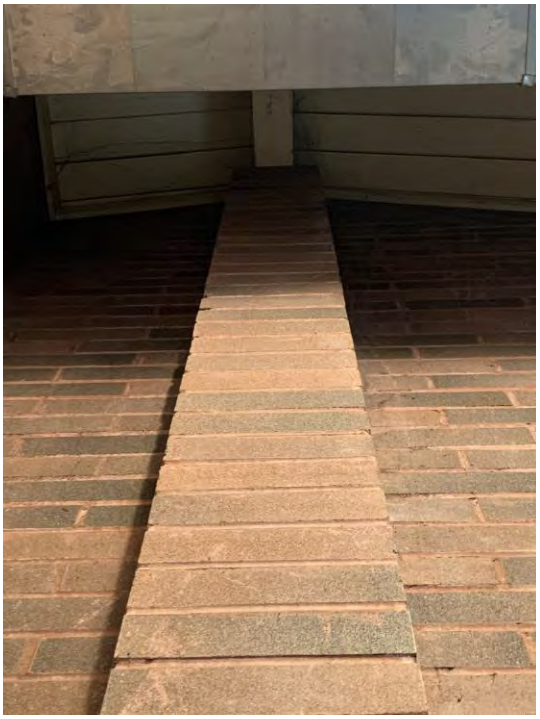


Figure 3.60 Masonry Pilaster Supporting Roof Framing

Figure 3.61 Unbraced Piping and Valves

(a) S1 Level Pumps

(b) S2 Level Pumps

Figure 3.62 Unbraced Pump Motor

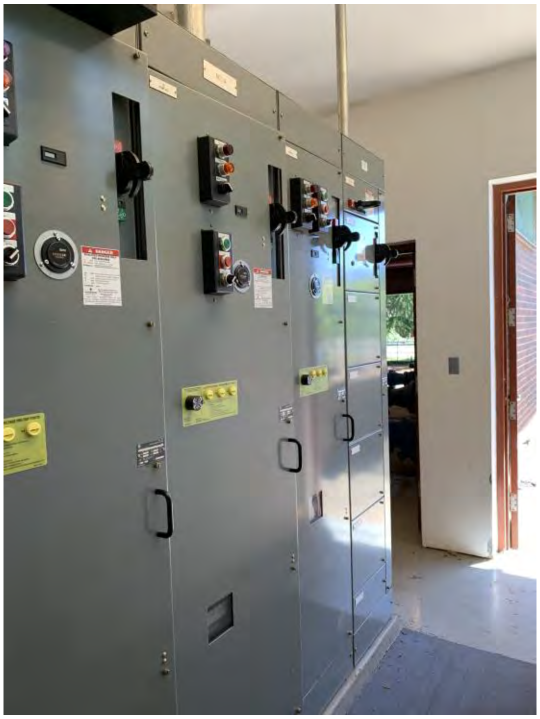


Figure 3.63 Electrical Cabinets with Unknown Anchorage Details

Figure 3.64 Conduits Connecting to Electrical Cabinets without Flexible Connections

Figure 3.65 Pendant Lights without Lens Covers

Figure 3.66 SCADA Antenna

Figure 3.67 Unanchored HVAC Condenser Unit

Figure 3.68 Grating without Clip Connection to Supporting Steel Framing

Figure 3.69 Unrestrained Overhead Bridge Crane

Figure 3.70 Unrestrained Ladder and Rolling Lift

Figure 3.71 Pole-Mounted Electrical Transformers

3.1.8 Limelight Pump Station

The Limelight Pump Station structure (see Figure 3.72) was built in 1998 at NW Van Buren Dr. This structure is an above-grade, single-story, reinforced masonry shear wall structure with a plywood sheathed wood framed roof. The building is rectangular in plan, with approximate dimensions of 20 feet in the east-west direction by 41 feet in the north-south direction.

The Limelight Pump Station houses three pumps that deliver water from the Glen Creek Reservoir to approximately 1,000 nearby homes.

Table 3.8 presents a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.8 the Limelight Pump Station is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Similarly, based on the structural deficiencies identified, the Limelight Pump Station is not currently expected to achieve Life Safety structural performance and represents a potential safety hazard to City staff and contractors.

Table 3.8 Limelight Pump Station Evaluation Summary

Potential Deficiencies	Description
Structural	• Several vertical cracks were observed in all four exterior masonry walls of the pump station (see Figure 3.73). Also, deterioration of the plywood sheathing and support framing was observed adjacent to the pump station entrances (see Figure 3.74). • The roof plywood sheathing nailing schedule was not provided in the available original design drawings. • The roof diaphragm span for east-west oriented seismic forces exceeds the ASCE 41-17 Tier 1 limit for unblocked wood structural panel diaphragms. • A positive connection does not appear to be provided between the sloped truss blocking and masonry wall top plate. Therefore, the load path is incomplete to transfer in-plane shear forces from the roof diaphragm to the masonry walls. Additionally, blocking in approximately every other bay has a long vent slot that limits its capacity to transfer seismic forces from the roof diaphragm to the masonry wall (see Figure 3.75).

Table 3.8 Limelight Pump Station Evaluation Summary (cont.)

Potential Deficiencies	Description
Structural (cont.)	• The available original design drawings do not indicate how the roof diaphragm is connected to the gable end triangular portion wood framed walls and masonry walls below, and do not provide any details for these wood framed walls. Therefore, the load path is potentially inadequate to transfer in-plane shear forces from the roof diaphragm to the masonry shear walls below. • Out-of-plane bracing of perimeter and interior masonry walls is not adequate. The roof trusses do not appear to be attached to the top plate of the perimeter masonry walls with any connection hardware intended to resist wall out-of-plane demands (see Figure 3.76). • Adequate cross ties between diaphragm chords are not provided in both directions.
Nonstructural	• Water system piping within the pump station is not braced (see Figure 3.77). • Valves in line with the water system piping are not braced (see Figure 3.77). • Air relief valves are not braced and are only supported by rigid small diameter piping (see Figure 3.78). • Vertical pump motors are not braced above the center of gravity of the motor (see Figure 3.79). • There does not appear to be adequate flexibility in the piping that is attached to the pumps to accommodate potential relative movement between the piping and the pump, and to prevent the piping from adding additional load to the pump anchorage. • Anchorage of electrical cabinets to the housekeeping pads was not visible from the outside of the cabinets and may not be adequate. Details for how the housekeeping pad is reinforced and connected to the slab on grade are not provided in the available original design drawings and may not be adequate. Additionally, electrical cabinets do not appear to be anchored or braced to the wall near the top of the cabinets to prevent them from tipping over during an earthquake (see Figure 3.80). • The horizontal arm of the SCADA antenna may not be adequately connected to the supporting pole to prevent the antenna from being misaligned or damaged after an earthquake (see Figure 3.81). • No anchorage was observed between the electrical transformer and concrete support pad (see Figure 3.82).

Figure 3.72 Limelight Pump Station

Figure 3.73 Masonry Wall Cracking

Figure 3.74 Sheathing and Framing Deterioration

Figure 3.75 Sloped Blocking Between Wood Trusses

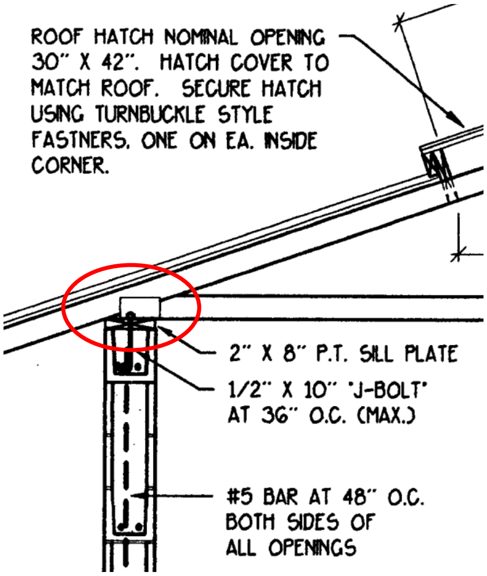


Figure 3.76 Inadequate Wall Out-of-Plane Anchorage Connection (Source: Typical Building Section on Sheet A 2.2 of 1997 design drawings by Multi/Tech Consultants)

Figure 3.77 Unbraced Piping and Valves

Figure 3.78 Unbraced Air Relief Valve

Figure 3.79 Unbraced Pump Motor

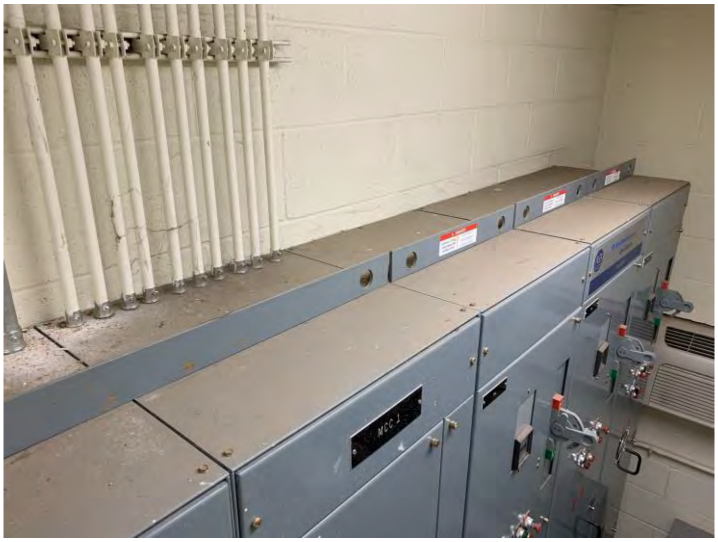


Figure 3.80 Electrical Cabinets with Unknown Anchorage Details

Figure 3.81 SCADA Antenna

Figure 3.82 Unanchored Electrical Transformer

3.1.9 Mountain View Pump Station

The Mountain View Pump Station structure (see Figure 3.83) was built in 1994 at 1616 Schoolhouse Ct NW. This structure is an above-grade, single-story, reinforced masonry shear wall structure with a plywood sheathed wood framed roof. The building is rectangular in plan, with approximate dimensions of 29 feet in the north-south direction by 62 feet in the east-west direction. A significant length of the north wall of the building is inset by approximately four feet. Roof framing at the north edge of the building is supported by a CMU beam that is then supported by three CMU square columns.

The Mountain View Pump Station houses four pumps that deliver water from the G0 Level to the Grice Hill Reservoir (W2 Level). There is a site/retaining wall to the south and west of the pump station that was excluded from the scope of this seismic assessment.

Table 3.9 presents a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.9 the Mountain View Pump Station is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Similarly, based on the structural deficiencies identified, the Mountain View Pump Station is not currently expected to achieve Life Safety structural performance and represents a potential safety hazard to City staff and contractors.

Table 3.9 Mountain View Pump Station Evaluation Summary

Potential Deficiencies	Description
Structural	• The load path is incomplete to deliver seismic forces from the roof diaphragm to the north masonry shear wall. Note that the observed as-built framing configuration is different than shown in the original design drawings (see Figure 3.84). • Out-of-plane bracing of perimeter and interior masonry walls is not adequate. The roof trusses do not appear to be attached to the top plate of the perimeter masonry walls with metal connector hardware specifically designed to resist out-of-plane seismic forces. In the direction perpendicular to the roof trusses, the roof sheathing is used to provide out-of-plane bracing for the masonry walls (see Figure 3.85). • Adequate cross ties between diaphragm chords are not provided in both directions.

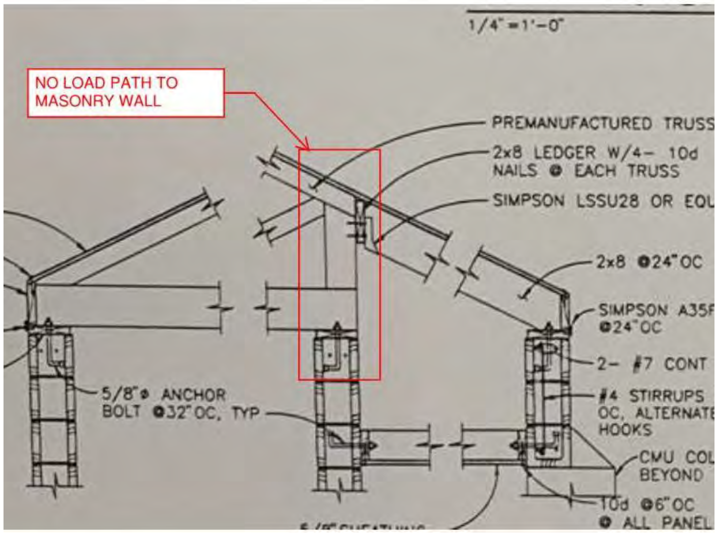
Table 3.9 Mountain View Pump Station Evaluation Summary (cont.)

Potential Deficiencies	Description
Nonstructural	• Water system piping within the pump station is not braced (see Figure 3.86) • Valves in line with the water system piping are not braced (see Figure 3.86). • Air relief valves are not braced and are only supported by rigid small diameter piping (see Figure 3.87). • Vertical pump motors are not braced above the center of gravity of the motor (see Figure 3.88). • There does not appear to be adequate flexibility in the piping that is attached to the pumps to accommodate potential relative movement between the piping and the pump, and to prevent the piping from adding additional load to the pump anchorage. • The piping gravity support stanchions are not anchored to the slab (see Figure 3.89). • The chlorination equipment is not adequately anchored, and the supporting concrete curb is severely damaged (see Figure 3.90 and Figure 3.91). • The fuse protection soft starter cabinets are restrained at the top by a wall mounted strut and spacers. The lag screw expansion shield anchors used to attach the strut to masonry wall are likely not seismically rated and may not provide adequate capacity (see Figure 3.92). Also, the short strut section spacers are not positively connected to the main strut. • The electrical transformer hung from the roof is not adequately braced (see Figure 3.93). • Anchorage of electrical cabinets to the concrete slab was not visible from the outside of the cabinets and may not be adequate. Additionally, electrical cabinets have only one clip angle bracket per cabinet that attaches between the top of the cabinet to the wall, which may not be adequate to prevent them from tipping over during an earthquake (see Figure 3.94). • Electrical conduits, hung from the roof and connected to the top of floor- and wall-mounted electrical panels and cabinets, may not have adequate flexibility to account for differential movement between the floor and the pump station roof (see • Figure 3.95). • The emergency generator starter batteries are not adequately restrained, and the battery bins are not anchored (see Figure 3.96). • The emergency generator muffler is not adequately braced (see Figure 3.97).

Table 3.9 Mountain View Pump Station Evaluation Summary (cont.)

Potential Deficiencies	Description
Nonstructural (cont.)	• Light fixtures in the pump station do not include lens covers (see Figure 3.98). • The horizontal arm of the SCADA antenna may not be adequately connected to the supporting pole to prevent the antenna from being misaligned or damaged after an earthquake (see Figure 3.99). • The overhead trolley chain hoist is not laterally braced and may damage other equipment during an earthquake (see Figure 3.100). • A ladder is unrestrained and may topple into and potentially damage the piping and valves during an earthquake (see Figure 3.100). • No anchorage was observed between the electrical transformer and concrete support pad (see Figure 3.101).

Figure 3.83 Mountain View Pump Station



(a) Detail from Original Design Drawings
(Source: Detail 2 on Sheet S4 of 1994 design drawings by KMC, Inc.)

(b) As-built Framing Configuration Different than Shown on Original Design Drawings

Figure 3.84 Incomplete Load Path at North Wall

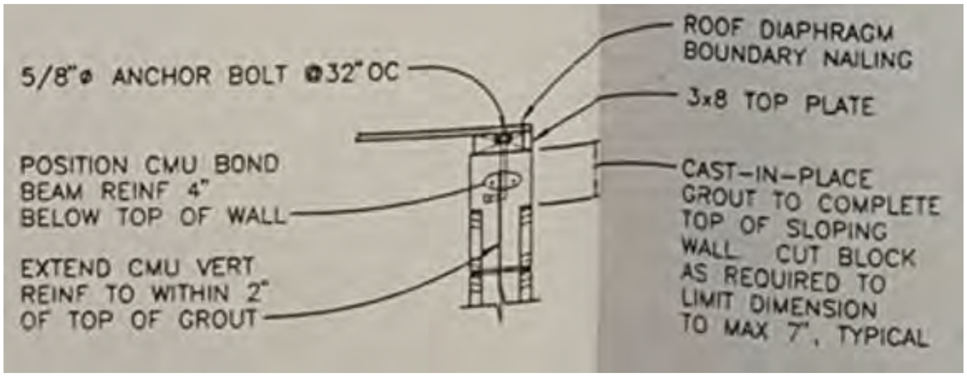


Figure 3.85 Inadequate East and West Wall Out-of-Plane Anchorage (Source: Detail 3 on Sheet S4 of 1994 design drawings by KMC, Inc.)

Figure 3.86 Unbraced Piping and Valves

Figure 3.87 Unbraced Air Relief Valve

Figure 3.88 Unbraced Pump Motor

Figure 3.89 Pipe Support Stanchion Missing Anchors into Floor Slab

Figure 3.90 Chlorine System without Adequate Anchorage

Figure 3.91 Deteriorated Concrete Curb

Figure 3.92 Inadequate Lag Screw and Spacer Strut Connection to Wall

Figure 3.93 Unbraced Elevated Electrical Transformer

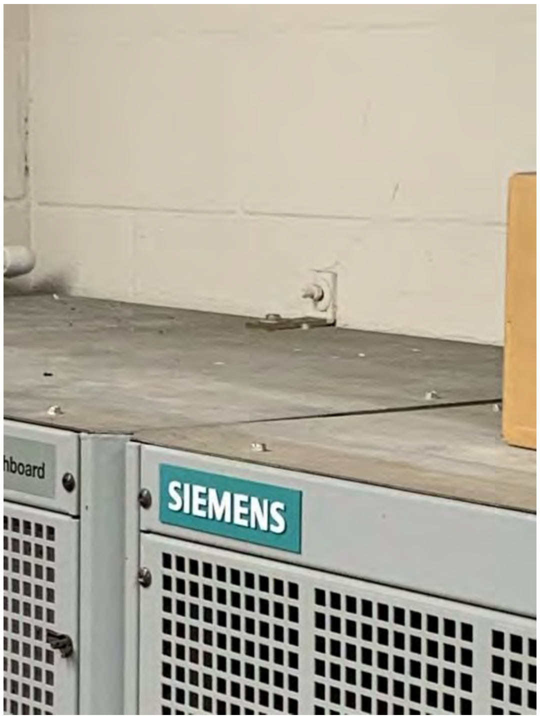


Figure 3.94 Electrical Cabinet with Single Anchor Bracket at Top of Cabinet

Figure 3.95 Conduits Connecting to Electrical Cabinets without Flexible Connections

Figure 3.96 Unrestrained Emergency Generator Starter Batteries

Figure 3.97 Unbraced Emergency Generator Muffler

Figure 3.98 Light Fixtures without Lens Covers

Figure 3.99 SCADA Antenna

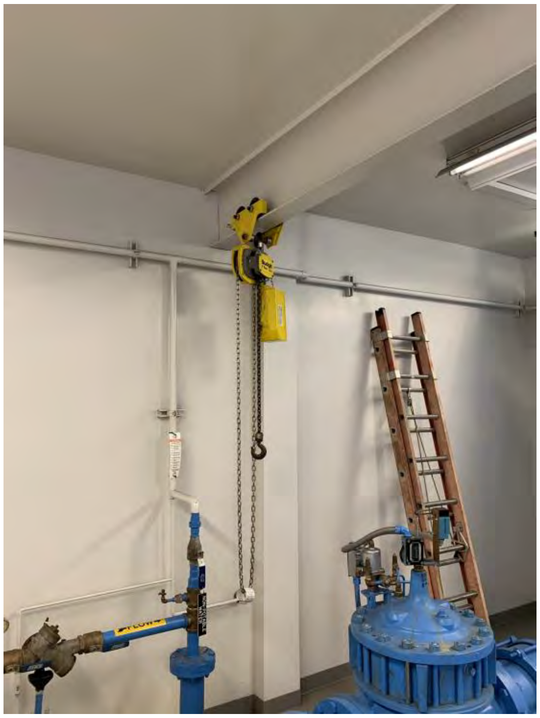


Figure 3.100 Unrestrained Chain Hoist and Ladder

Figure 3.101 Unanchored Electrical Transformer

3.1.10 Salem/Keizer Intertie #1 Pump Station

The Salem/Keizer Intertie #1 Pump Station structure (see Figure 3.102) was built in 2013 at 4110 Cherry Ave NE. This structure is an above-grade, single-story, reinforced masonry shear wall structure with a plywood sheathed wood framed roof. The building is rectangular in plan, with approximate dimensions of 26 feet in the north-south direction by 22 feet in the east-west direction. The pump station is separated from a City of Keizer well building immediately to the east of the pump station by a half-inch gap.

The Salem/Keizer Intertie #1 Pump Station houses one pump and chlorination equipment that can be used as an emergency source to deliver water from the City of Keizer to the City of Salem system. The pump and piping have a capacity of approximately 10 million gallons per day (MGD). However, the City of Keizer is only able to deliver approximately 4 to 5 MGD to the City of Salem at this intertie.

Table 3.10 presents a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.10 the Salem/Keizer Intertie #1 Pump Station is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Similarly, based on the structural deficiencies identified, the Salem/Keizer Intertie #1 Pump Station is not currently expected to achieve Life Safety structural performance and represents a potential safety hazard to City staff and contractors.

Table 3.10 Salem/Keizer Intertie #1 Pump Station Evaluation Summary

Potential Deficiencies	Description
Structural	• Per the Technical Memorandum “Seismic Geohazard Evaluation Report, City of Salem Seismic Resilience Study” by Shannon & Wilson, Inc., there is a potential liquefaction hazard at the site. Liquefaction-induced permanent ground deformation may result in damage to the building structure. • The City of Salem pump station and the adjacent City of Keizer building are only separated by a half-inch seismic joint. This small separation may not be adequate to prevent damage resulting from earthquake shaking-induced pounding between the two buildings. • Roof sheathing is not continuous to the roof ridge line (see Figure 3.103). • A positive connection does not appear to be provided between the truss blocking and masonry wall top plate. Therefore, the load path is incomplete to transfer in-plane shear forces from the roof diaphragm to the masonry walls (see Figure 3.104).

Table 3.10 Salem/Keizer Intertie #1 Pump Station Evaluation Summary (cont.)

Potential Deficiencies	Description
Structural (cont.)	• In the east-west direction, continuous cross ties are not provided between diaphragm chords. Blocking and metal connector straps are provided at 2 feet on center in all but two of the bays between trusses (see Figure 3.105).
Nonstructural	• Water system piping that penetrates through the pump station floor may not have adequate flexibility to accommodate the potential differential movement between the pump station and the surrounding soil at the pipe penetration. • Water system piping within the pump station is not braced (see Figure 3.106). Also, the overflow pipe on the south side of the pump station is not braced (Figure 3.107). • Valves in line with the water system piping are not braced (see Figure 3.106). • There does not appear to be adequate flexibility in the piping that is attached to the pumps to accommodate potential relative movement between the piping and the pump, and to prevent the piping from adding additional load to the pump anchorage. • The chlorination skid is not adequately restrained (see Figure 3.108). Also, the tank is bolted to the floor grid of the chlorination skid, but the floor grid is not positively connected to the skid itself. • Anchorage of electrical cabinets to the concrete housekeeping pads was not visible from the outside of the cabinets and may not be adequate. • The top of the electrical cabinet is restrained with an L-shaped bracket that was fabricated by cutting the flanges of a short section of strut and bending about the web of the strut. The web of the strut may be susceptible to fracture during an earthquake based on how it was fabricated. Also, the vertical position of the anchor bolt between the bracket and wall results in a large eccentricity that will cause additional prying action demand on the anchor (see Figure 3.109). • The horizontal arm of the SCADA antenna may not be adequately connected to the supporting pole to prevent the antenna from being misaligned or damaged after an earthquake (see Figure 3.110). • No anchorage was observed between the electrical transformer and concrete support pad (see Figure 3.111).

Figure 3.102 Salem/Keizer Intertie #1 Pump Station

Figure 3.103 Roof Sheathing not Continuous to Ridge Line

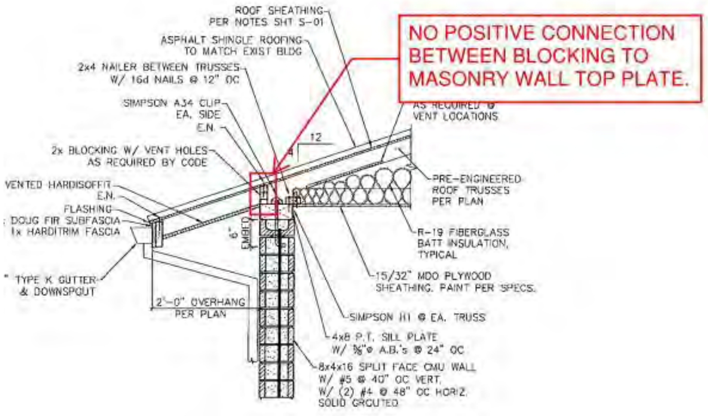


Figure 3.104 Incomplete Load Path
(Source: Detail 3 on Sheet S-06 of 2012 design drawings by Westech Engineering)

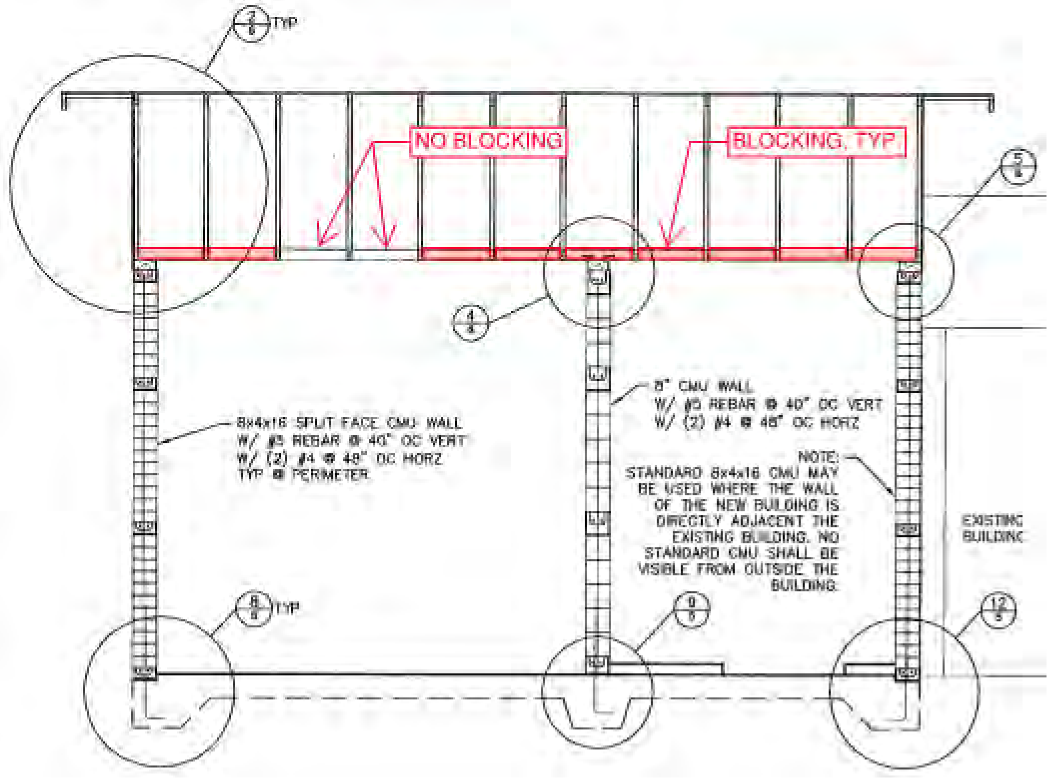


Figure 3.105 Ceiling Level Blocking not Continuous
(Source: Section B on Sheet S-03 of 2012 design drawings by Westech Engineering)

Figure 3.106 Unbraced Piping and Valves

Figure 3.107 Unbraced Overflow Pipe on South Side of Pump Station

Figure 3.108 Unanchored Chlorination Skid

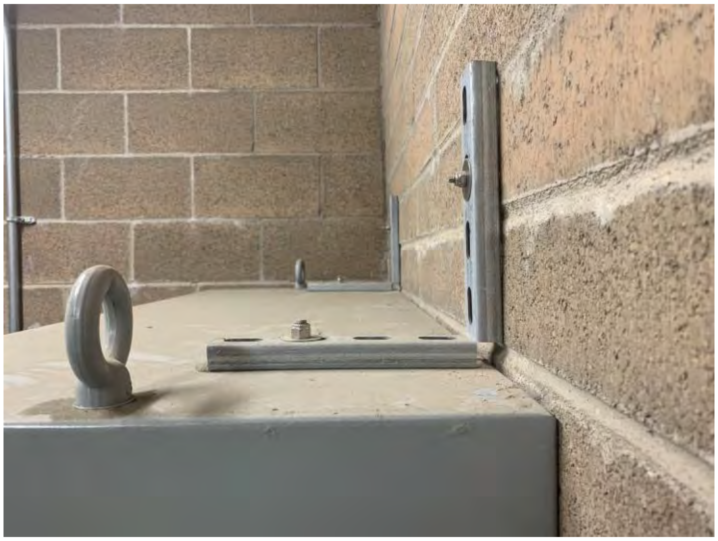


Figure 3.109 Anchorage at Electrical Cabinet with Bent Strut

Figure 3.110 SCADA Antenna

Figure 3.111 Unanchored Electrical Transformer

3.1.11 Turner Control Facility

The original Turner Control Facility at 7100 3rd Street SE in Turner was substantially replaced in 2007. However, a small subgrade portion of the original Turner Control Facility was integrated into the new structure. The Turner Control Facility (see Figure 3.112) is a single-story, above grade reinforced masonry shear wall structure with a plywood sheathed light-gauge metal framed roof. The building is constructed over two sections of concrete basement, where the three water transmission lines and associated valves are located. The building is rectangular in plan, with approximate wall dimensions of 36 feet in the northwest-southeast direction by 52 feet in the northeast-southwest direction.

The Turner Control Facility houses valves that are used to control the flow of water to the G0 Level system from Franzen Reservoir and Geren Island Water Treatment Facility.

Table 3.11 presents a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.11, the Turner Control Facility is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Similarly, based on the structural deficiencies identified, the Turner Control Facility is not currently expected to achieve Life Safety structural performance and represents a potential safety hazard to City staff and contractors.

Table 3.11 Turner Control Facility Evaluation Summary

Potential Deficiencies	Description
Structural	• Per the Technical Memorandum “Seismic Geohazard Evaluation Report, City of Salem Seismic Resilience Study” by Shannon & Wilson, Inc., there is a potential liquefaction hazard at the site. Liquefaction-induced permanent ground deformation may result in damage to the building structure. • The roof diaphragm spans in both directions exceed the ASCE 41-17 Tier 1 limit for unblocked wood structural panel diaphragms. • At the gable end walls, the roof sheathing to blocking and blocking to masonry wall top plate fastener detailing are unclear. The load path may not be adequate to transfer in-plane shear forces from the roof diaphragm to the masonry walls (see Figure 3.113). • At the gable end walls, the outrigger to roof diaphragm connection may not have adequate capacity to resist the expected out-of-plane seismic forces from the masonry walls. • There are no cross ties provided between diaphragm chords in the direction perpendicular to the roof trusses.

Table 3.11 Turner Control Facility Evaluation Summary (cont.)

Potential Deficiencies	Description
Nonstructural	• Water system piping that penetrates through the control facility walls may not have adequate flexibility to accommodate the potential differential movement between the control facility and the surrounding soil at the pipe penetration. • Water system piping within the control facility does not appear to be adequately braced (see Figure 3.114) • Valves in line with the water system piping are not braced (see Figure 3.115). • The valve actuators are not braced (see Figure 3.116). • The control cabinet did not appear to be anchored to the housekeeping pad or wall (see Figure 3.117). • The electrical transformer is only anchored with two anchors at the front of the unit. It is missing two anchors into the concrete slab at the back of the unit (see Figure 3.118). • Backup batteries in the battery cabinet are not adequately restrained (see Figure 3.119). • Pendant lights are not restrained to prevent them from hitting the wall (see Figure 3.120). • Light fixtures in the control facility do not include lens covers (see Figure 3.121). • The horizontal arm of the SCADA antenna may not be adequately connected to the supporting pole to prevent the antenna from being misaligned or damaged after an earthquake (see Figure 3.122). • The ceiling hung inline HVAC fan is not laterally braced (see Figure 3.123). • No anchorage was observed between the HVAC condenser unit and concrete support pad (see Figure 3.124). • Two storage shelving units are not anchored to the floor and/or the wall (see Figure 3.125). • The fire extinguisher is not adequately restrained in its cabinet (see Figure 3.126).

Figure 3.112 Turner Control Facility

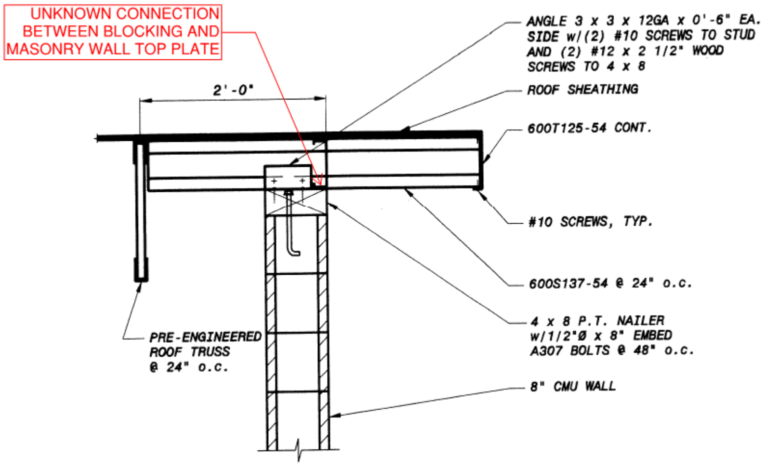


Figure 3.113 Incomplete Load Path at Gable End Walls
(Source: Detail 2 on Sheet S11 of 2007 design drawings by Black & Veatch)

Figure 3.114 Unbraced Pipe

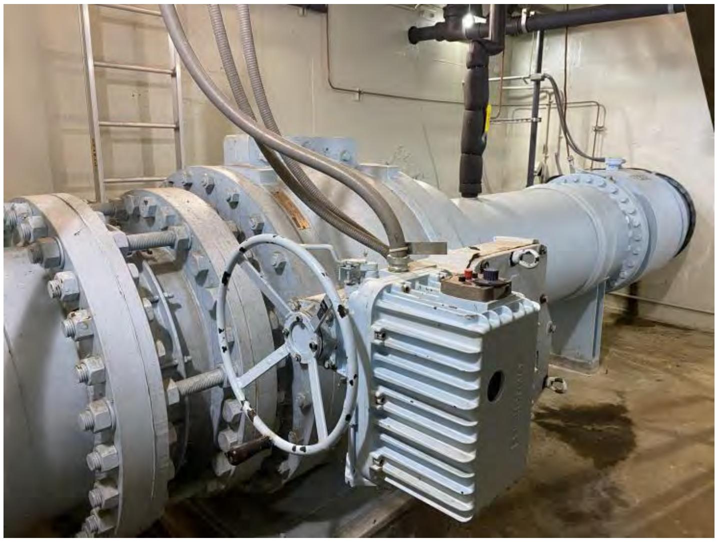
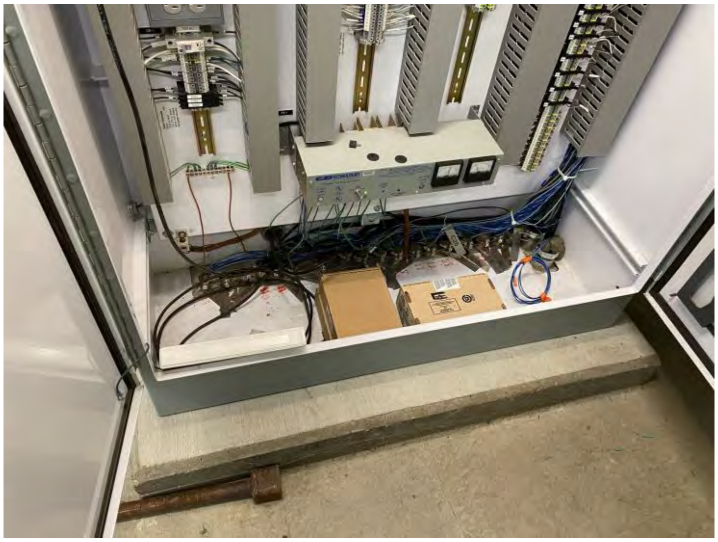


Figure 3.115 Unbraced Valve

Figure 3.116 Unbraced Valve Actuator

(a) Exterior View



(b) Interior View

Figure 3.117 Unanchored Control Cabinet

Figure 3.118 Electrical Transformer Missing Anchors into Floor Slab

Figure 3.119 Unrestrained Backup Batteries in Battery Cabinet

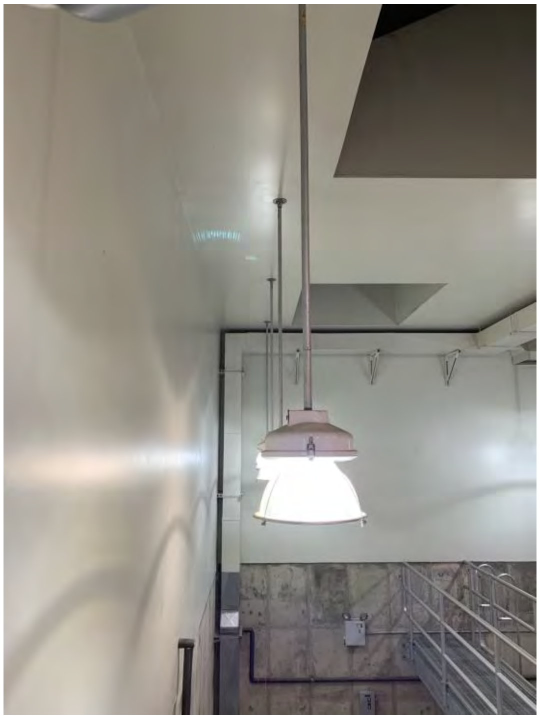


Figure 3.120 Unrestrained Pendant Light Fixtures

Figure 3.121 Light Fixtures without Lens Covers

Figure 3.122 SCADA Antenna

Figure 3.123 Unbraced Inline Fan Unit

Figure 3.124 Unanchored HVAC Condenser Unit

Figure 3.125 Unanchored Shelf

Figure 3.126 Unrestrained Fire Extinguisher

3.2 Reservoirs and Reservoir Control Buildings

The expected structural and nonstructural seismic performance for selected City water reservoirs and associated reservoir control buildings has been evaluated for a M9.0 CSZ scenario earthquake. The following sections provide a short narrative description of each reservoir and associated reservoir control building (where applicable), followed by tables that summarize the potential seismic structural and nonstructural deficiencies identified by the seismic evaluations conducted using the procedures described in Section 2.3. For each reservoir and reservoir control building, selected images from the design drawings and/or site visit photos are provided to help illustrate the identified potential structural and nonstructural deficiencies.

Site visits to these reservoirs and reservoir control buildings were conducted by SEFT on May 11th, 14th, 18th, and 25th, 2021. Site observation was limited to those areas readily accessible to view, and did not include any areas concealed by existing finishes, such as ceilings, soffits, etc. Site observation did not include entry into any permit required confined spaces and did not include any entry or observation inside the reservoirs. A detailed structural condition assessment of these structures was not included in the scope of this project.

3.2.1 Candalaria Reservoir

The Candalaria Reservoir (see Figure 3.127) is located at Candalaria Park, to the north of Candalaria Blvd S. The 0.5-million-gallon (MG) reservoir was originally constructed in 1940. This reservoir is a completely buried rectangular reinforced concrete reservoir with approximate dimensions of 123 feet in the north-south direction by 50 feet in the east-west direction, and a maximum height of retained water of 15 feet. The Candalaria Reservoir serves the City's S1 Level. The City has future plans to construct additional water storage capacity on this site, with a new reservoir located to the south of the existing reservoir.

In 2006 the reservoir was seismically retrofit. The scope of this retrofit included the addition of anchors to connect the roof of the reservoir to the walls. The 2006 retrofit also included the installation of a seismic shutoff valve in a new vault located on the north side of the reservoir. Note that SEFT did not have access to the interior of this valve vault during our site visit. In 2011, Murray, Smith & Associates conducted a condition assessment of Candalaria Reservoir. SEFT reviewed the report associated with the 2011 condition assessment to help inform our seismic assessment.

Table 3.12 presents a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.12, the Candalaria Reservoir is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake.

Table 3.12 Candalaria Reservoir Evaluation Summary

Potential Deficiencies	**Description**
Structural	<u>Reservoir</u> • The column vertical reinforcing lap splice length is less than the ASCE 41-17 Tier 1 specified minimum length for Immediate Occupancy structural performance (i.e., 50 bar diameters). <u>Valve Vault</u> • Per the 2006 design drawings, the valve vault was specified to be cast-in-place concrete or precast concrete, at the contractor's option. If the valve vault was constructed from precast concrete, riser joints of stacked precast components may separate and shift due to seismic lateral earth pressures of the face of the valve vault. • Sand, silt, or groundwater may infiltrate and leak into the valve vault at the precast concrete construction joints.
Nonstructural	<u>Reservoir</u> • Some piping and fittings within the reservoir may be cast-iron, which is a brittle material that may crack when subjected to earthquake shaking-induced forces and/or ground deformation. • The overflow pipe and valve operator riser shafts may not be adequately braced to resist seismic forces (see Figure 3.128). Note that the 2011 condition assessment also indicated that these elements were observed to have significant corrosion deterioration. <u>Valve Vault</u> • Per the 2006 design drawings, the piping and valve inside the valve vault are not independently braced. • Backup batteries in the battery cabinet (for operation of the seismic valve) may not be adequately restrained.

Figure 3.127 Candalaria Reservoir

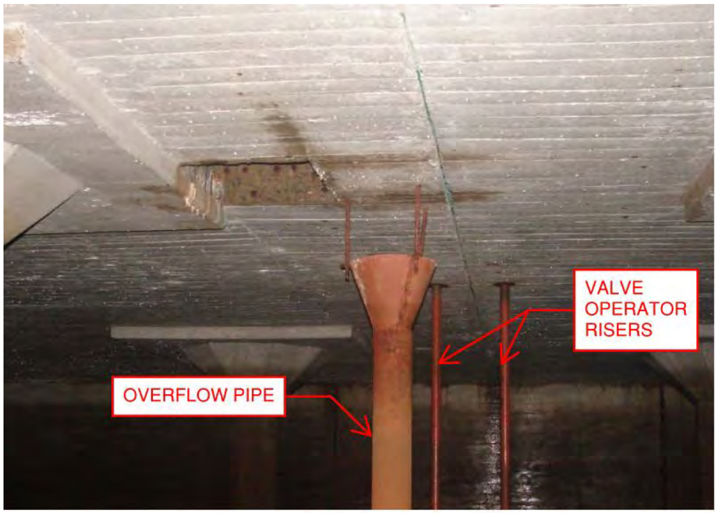


Figure 3.128 Overflow Pipe and Valve Operator Risers Not Adequately Braced (Source: 2011 Reservoir Condition Assessment by Murray, Smith & Associates)

3.2.2 Champion Hill Reservoir and Control Building

The Champion Hill Reservoir (see Figure 3.129) is a 2.2 MG tank built in 2005 at the Champion Hill site off Reservoir Road SE. This reservoir is a strand-wound, circular, prestressed concrete reservoir with a nearly flat roof, an approximate diameter of 140 feet and a maximum height of retained water of 20 feet. The Champion Hill Reservoir serves the City's S3 Level and is supplied by the Creekside Pump Station.

The Champion Hill Reservoir Control Building (see Figure 3.130) is located to the north of the reservoir. The control building is a single-story structure, with reinforced concrete walls below grade, reinforced masonry walls above grade, and a plywood sheathed wood truss roof. The building is rectangular in plan, with approximate dimensions of 38 feet in north-south direction by 46 feet in east-west direction. The building houses piping and valves (including seismic shutoff valves) that support the operation of the reservoir. The original design of the control building included a chlorination room and associated equipment, but the chlorination system is not currently used. Therefore, the chlorination system has not been included in the scope of the nonstructural assessment.

Table 3.13 and Table 3.14 present a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.13, the Champion Hill Reservoir is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Based on potential deficiencies identified in Table 3.14, the Champion Hill Reservoir Control Building is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for the M9.0 CSZ earthquake. Similarly, based on the structural deficiencies identified, the Champion Hill Reservoir Control Building is not currently expected to achieve Life Safety structural performance and represents a potential safety hazard to City staff and contractors.

Table 3.13 Champion Hill Reservoir Evaluation Summary

Potential Deficiencies	Description
Structural	• Per the Technical Memorandum "Seismic Geohazard Evaluation Report, City of Salem Seismic Resilience Study" by Shannon & Wilson, Inc., the reservoir site is potentially founded on silty soil that may be susceptible to liquefaction depending on the groundwater level. • The column vertical reinforcing lap splice length is less than the ASCE 41-17 Tier 1 specified minimum length for Immediate Occupancy structural performance (i.e., 50 bar diameters).

Table 3.13 Champion Hill Reservoir Evaluation Summary (cont.)

Potential Deficiencies	**Description**
Nonstructural	• The pipe support concrete pedestals within the reservoir do not appear to be positively connected to the reservoir floor. Original design drawings show that the vertical reinforcing at the corners of the pedestal terminates at the top of the reservoir floor (see Figure 3.131).

Table 3.14 Champion Hill Reservoir Control Building Evaluation Summary

Potential Deficiencies	**Description**
Structural	• Per the Technical Memorandum "Seismic Geohazard Evaluation Report, City of Salem Seismic Resilience Study" by Shannon & Wilson, Inc., the reservoir site is potentially founded on silty soil that may be susceptible to liquefaction depending on the groundwater level. • The roof diaphragm spans in both directions exceed the ASCE 41-17 Tier 1 limit for unblocked wood structural panel diaphragms. • A positive connection does not appear to be provided between the sloped truss blocking and masonry wall top plate (see Figure 3.132). Therefore, the load path is incomplete to transfer in-plane shear forces from the roof diaphragm to the masonry walls. • A positive connection does not appear to be provided between the gable end wall sheathing and the masonry wall top plate (see Figure 3.133). Instead of the sheathing being edge nailed to the masonry wall top plate, drawings indicate edge nailing to the end truss bottom chord. However, no positive connection is indicated between the end truss bottom chord and masonry wall top plate. Therefore, the load path is incomplete to transfer in-plane shear forces from the roof diaphragm to the masonry walls. • Out-of-plane bracing of the east and west gable end masonry walls is not adequate. Kicker braces are provided between the top of the masonry walls and roof diaphragm (see Figure 3.134). However, no positive connection is indicated between the blocking that the kicker brace frames into and the roof trusses. Therefore, the load path is incomplete to resist the vertical component of the kicker brace force associated with providing out-of-plane bracing for the gable end masonry walls.

Table 3.14 Champion Hill Reservoir Control Building Evaluation Summary (cont.)

Potential Deficiencies	**Description**
Nonstructural	• Water system piping that penetrates through the control building walls may not have adequate flexibility to accommodate the potential differential movement between the control facility and the surrounding soil at the pipe penetration (see Figure 3.135). • Water system piping within the control building is not adequately seismically braced (see Figure 3.136). • Valves in line with the water system piping are not braced (see Figure 3.137). • The motor for the reservoir recirculation pump is not anchored at the base (see Figure 3.138) and the associated piping is not braced. • The pressure tank for the irrigation system appears to be missing an anchor at the base (see Figure 3.139). • Backup batteries in the battery cabinet (for operation of the seismic valves) are not adequately restrained (see Figure 3.140). • The horizontal arm of the SCADA antenna may not be adequately connected to the supporting pole to prevent the antenna from being misaligned or damaged after an earthquake (see Figure 3.141). • The ceiling framing of the chlorine room inside the control building does not appear to provide adequate connections to transfer seismic forces from the ceiling to the masonry walls below. Also, the wood ledger attachment to the masonry wall is not detailed to avoid cross-grain bending (see Figure 3.142). • The temporarily stored electrical cabinets are not anchored and may tip over during an earthquake and potentially damage valves or other components (see Figure 3.143). • The wood pole and anchorage of the pole-mounted transformer to the pole may not be adequately designed to resist the seismic forces generated by the transformer (see Figure 3.144).

Figure 3.129 Champion Hill Reservoir

Figure 3.130 Champion Hill Reservoir Control Building

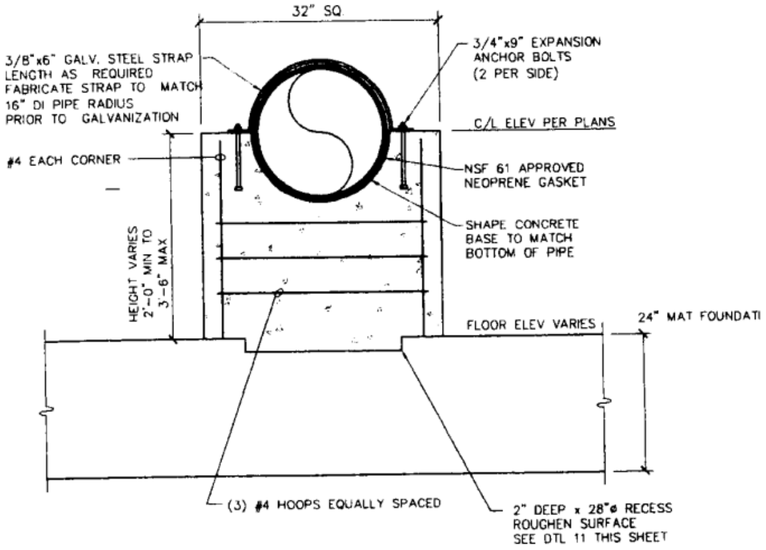


Figure 3.131 Reservoir Pipe Support Detail
(Source: Detail 10 on Sheet S5 of 2005 design drawings by Westech Engineering)

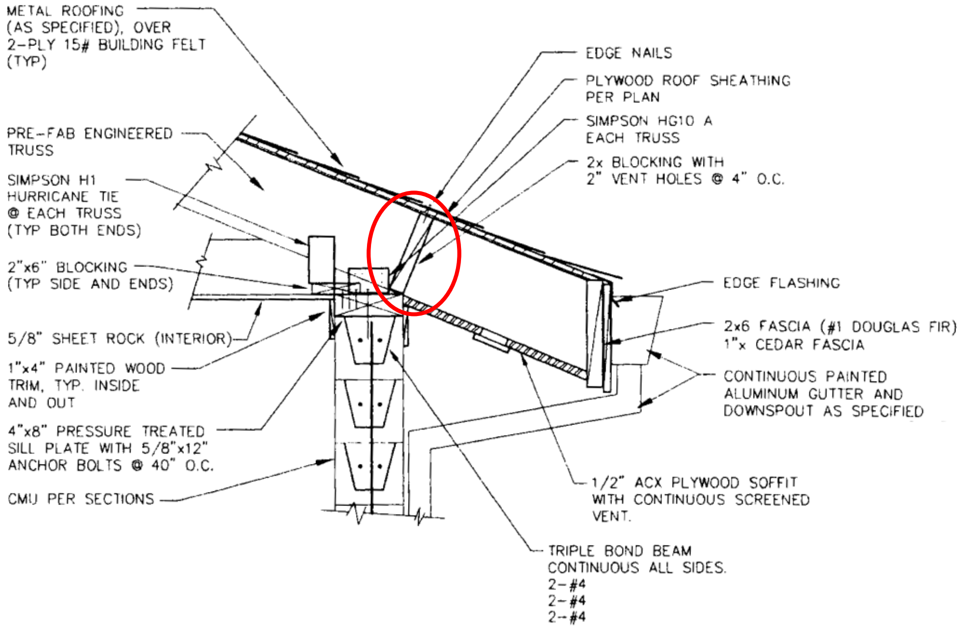


Figure 3.132 Inadequate Connection between Blocking and Masonry Wall Top Plate
(Source: Detail 6 on Sheet S13 of 2005 design drawings by Westech Engineering)

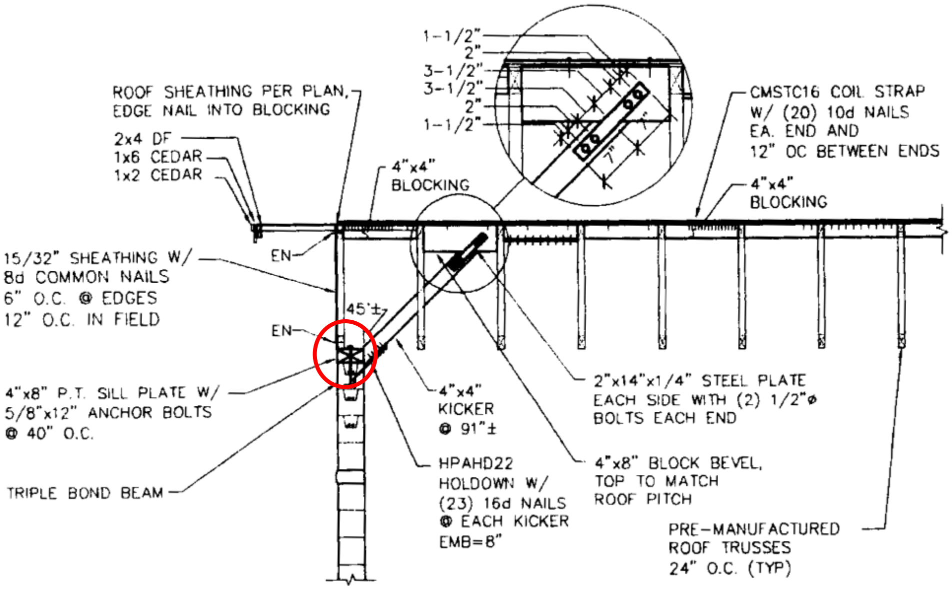


Figure 3.133 Inadequate Connection between Sheathing and Masonry Wall Top Plate (Source: Detail 4 on Sheet S13 of 2005 design drawings by Westech Engineering)

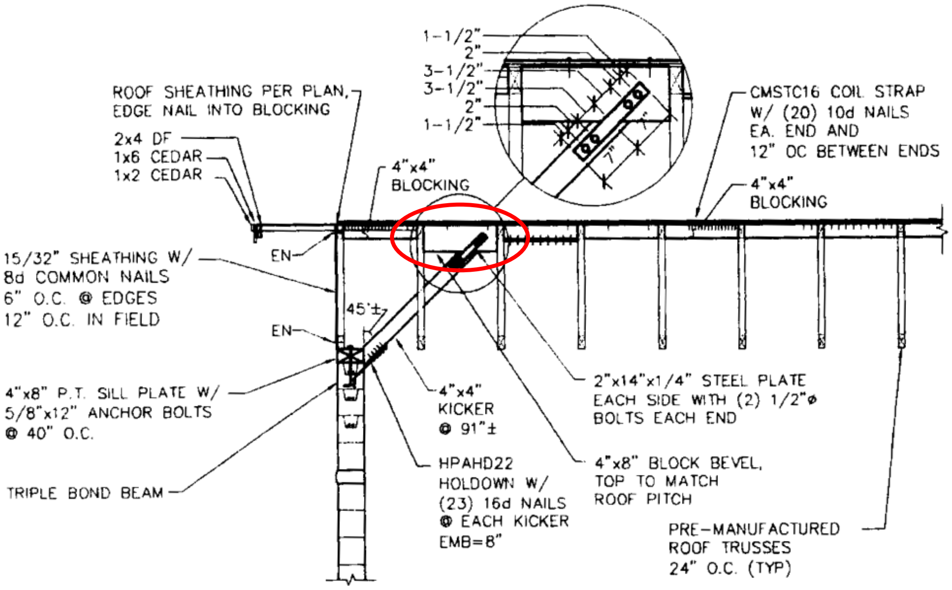


Figure 3.134 Inadequate Connection between Blocking and Roof Truss (Source: Detail 4 on Sheet S13 of 2005 design drawings by Westech Engineering)

Figure 3.135 Rigid Pipe Connection Through Wall

Figure 3.136 Unbraced Piping and Valves

Figure 3.137 Unbraced Seismic Valve

Figure 3.138 Unanchored Recirculation Pump Motor

Figure 3.139 Irrigation Pressure Tank Missing Anchor

Figure 3.140 Unrestrained Backup Batteries

Figure 3.141 SCADA Antenna

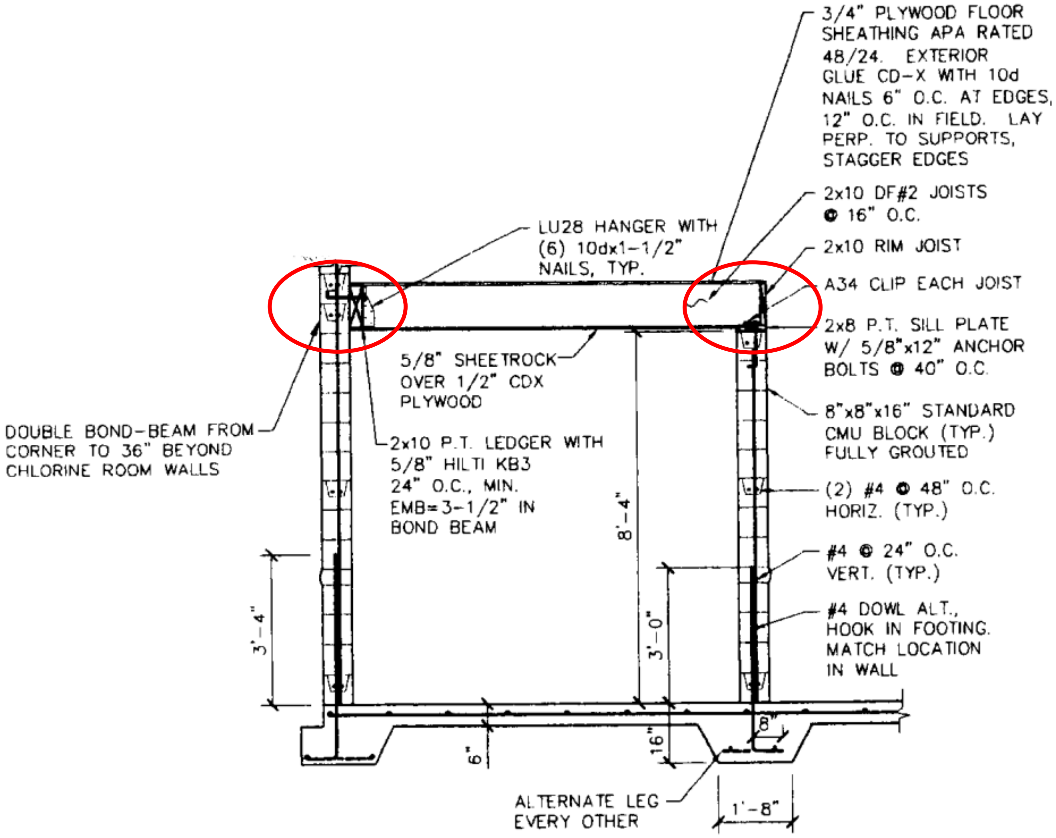


Figure 3.142 Chlorine Room Ceiling Framing
(Source: Section C on Sheet S11 of 2005 design drawings by Westech Engineering)

Figure 3.143 Temporarily Stored Electrical Cabinets

Figure 3.144 Pole-mounted Electrical Transformer

3.2.3 Eola #1B Reservoir

The Eola #1B Reservoir (see Figure 3.145) is a 0.86 million-gallon (MG) reservoir constructed in 1999, at a site west of 35th Avenue NW and north of Eola Drive NW. This reservoir is a circular-shaped reinforced concrete reservoir with an approximate diameter of 92 feet, and a maximum height of retained water of 17 feet. The west side of the reservoir is completely buried and the east side of the reservoir is partially exposed (with a maximum exposed height of approximately 3 feet). The Eola #1B Reservoir serves the City's W3 Level and is supplied by the Gibson Woods Pump Station.

Two, approximately 8-foot diameter, precast concrete valve vaults are located to the southeast of the reservoir. These vaults house piping and valves that support the operation of the reservoir. Note that SEFT did not have access to the interior of these valve vaults during our site visit.

Table 3.15 presents a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.15, the Eola Reservoir #1 is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake.

Table 3.15 Eola #1B Reservoir Evaluation Summary

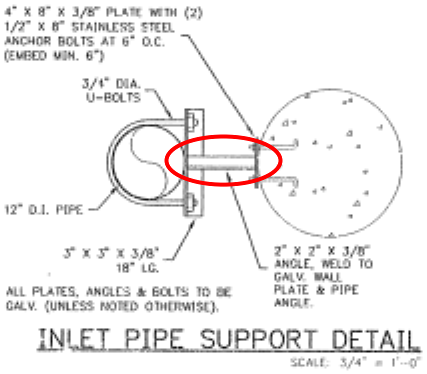
Potential Deficiencies	Description
Structural	<u>Reservoir</u> • Circumferential concrete cracking was observed near the roof to wall interface. The cracking was observed on the east side of the reservoir with a combined length of approximately one-eight the circumference of the reservoir. • The column vertical reinforcing lap splice length is less than the ASCE 41-17 Tier 1 specified minimum length for Immediate Occupancy structural performance (i.e., 50 bar diameters). <u>Valve Vaults</u> • Concrete deterioration was observed near the top of South Valve Vault wall to lid interface (see Figure 3.146) that may impact the seismic performance of the valve vault. • Valve vaults are constructed from precast concrete components. The riser joints of stacked precast components may separate and shift due to seismic lateral earth pressures of the face of the valve vault. • Sand, silt, or groundwater may infiltrate and leak into the valve vaults at the precast concrete construction joints.

Table 3.15 Eola #1B Reservoir Evaluation Summary (cont.)

Potential Deficiencies	Description
Nonstructural	Reservoir • The vertical section of inlet pipe may not be adequately braced as the bracing detail relies on cantilever bending of a relatively small angle section (see Figure 3.147). Valve Vaults • Per the 1999 design drawings, the piping and valves inside the valve vault may not be independently braced.

Figure 3.145 Eola #1B Reservoir

Figure 3.146 Concrete Deterioration at Lid to Wall Connection of South Valve Vault



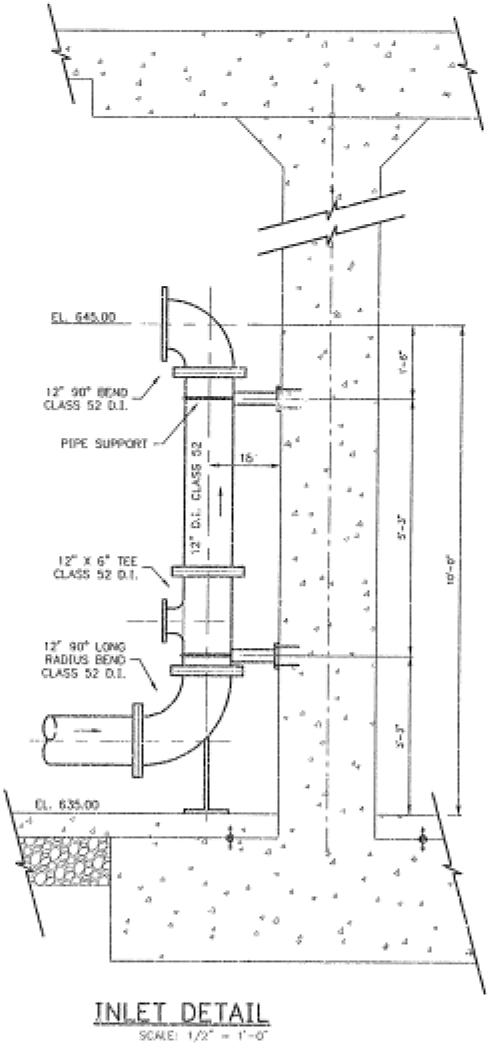


Figure 3.147 Reservoir Inlet Pipe Support Detail
(Source: Sheet 6 of 1999 design drawings by Multi/Tech Consultants)

3.2.4 Fairmount Reservoir and Control Building

The Fairmount Reservoir (see Figure 3.148), is a 10 MG reservoir constructed in 1936, at the Fairmount City Park near the intersection of Rural Avenue S and John Street S. This rectangular-shaped reinforced concrete reservoir is divided into two cells, each with a 5 MG capacity, and has approximate overall dimensions of 384 feet in the east-west direction by 192 feet in the north-south direction, and a maximum height of retained water of 21 feet. The reservoir is partially buried with approximately the top four feet exposed above grade. The Fairmount Reservoir serves the City's G0 Level and is hydraulically connected to both Franzen and Mountain View Reservoirs.

The Fairmount Reservoir Control Building/Pump Station (see Figure 3.149) is located on the south side of the reservoir and adjacent to the division between the two cells. The control building is a single-story above grade with a basement, constructed with reinforced concrete walls, and a reinforced concrete floor and roof. Two walls of the control building were constructed integrally with the Fairmount Reservoir. The building is square in plan, with approximate dimensions of 21 feet by 21 feet. During SEFT's site visit, the City noted that the pumps in this building are very rarely used.

In 2007, Black & Veatch conducted a condition assessment and seismic study of the Fairmount Reservoir and, in 2008, completed a follow-up structural evaluation of the roof. In 2018, Carollo Engineers conducted a seismic study of the Fairmount Reservoir and developed preliminary seismic retrofit concepts, and also developed repair concepts to address observed leaking of roof joints. SEFT reviewed the reports associated with these previous studies as part of our desktop evaluation of the Fairmount Reservoir. It should be noted that these previous studies were preliminary in nature and did not include consideration of the potential interaction between the reservoir and adjacent control building/pump station. The City plans to implement a future seismic retrofit of the Fairmount reservoir based on the recommendations of the 2018 Carollo seismic study.

Table 3.16 presents a summary of the seismic structural deficiencies for the Fairmount Reservoir that were identified in the 2018 reservoir seismic study conducted by Carollo Engineers and additional potential structural and nonstructural deficiencies identified by SEFT as part of this project. Note that based on our desktop evaluation and considering the M9.0 CSZ scenario earthquake seismic hazard parameter data provided by Shannon & Wilson as part of this project, SEFT concurs with the structural seismic deficiencies identified by Carollo.

Table 3.17 presents a summary of potential seismic structural and nonstructural deficiencies for the Fairmount Reservoir Control Building/Pump Station that were identified by this evaluation.

Based on the potential deficiencies identified in Table 3.16, the Fairmount Reservoir is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Based on the potential deficiencies identified in Table 3.17, the Fairmount Reservoir Control Building/Pump Station is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Similarly, based on the structural deficiencies identified, the Fairmount Reservoir Control Building/Pump Station is not currently expected to achieve Life Safety structural performance and represents a potential safety hazard to City staff and contractors.

Table 3.16 Fairmount Reservoir Evaluation Summary

Potential Deficiencies	Description
Structural (based on 2018 seismic study by Carollo Engineers)	• The perimeter walls and footings are overstressed due the tension loads imposed by the bending moment loads caused by hydrodynamic forces. • There is no load path provided to transfer seismic forces from the reservoir roof to the walls. The roof expansion joints (see Figure 3.150) cannot transfer shear forces between adjacent roof panels. Additionally, there is not positive connections between the reservoir roof and walls (see Figure 3.151). This results in reservoir columns being overstressed.
Additional Structural (based on SEFT desktop assessment)	• The 2018 Carollo study considered the BSE-1E seismic hazard level as defined by ASCE 41-13. Chapter 34 of the 2019 Oregon Structural Specialty Code (OSSC) indicates that the BSE-1E hazard level should not be taken as less than 75% of the BSE-1N seismic hazard level as defined by ASCE 41, much higher than what was considered in the 2018 Carollo study. • Previous studies were preliminary in nature and did not include consideration of the potential interaction between the Fairmount Reservoir and adjacent Fairmount Reservoir Control Building/Pump Station. • The column vertical reinforcing lap splice length is less than the ASCE 41-17 Tier 1 specified minimum length for Immediate Occupancy structural performance (i.e., 50 bar diameters).
Nonstructural	• Per the original drawings, some piping and fittings within the reservoir may be cast-iron, which is a brittle material that may crack when subjected to earthquake shaking-induced forces and/or ground deformation.

Table 3.17 Fairmount Reservoir Control Building Seismic Evaluation Summary

Potential Deficiencies	Description
Structural	• The northeast and northwest walls of the control building/pump station were constructed integrally with the reservoir (see Figure 3.152). Evaluation of the potential interaction between these two structures is beyond the scope of this preliminary ASCE 41 Tier 1 check-list based assessment, but should be considered as part of a future detailed seismic evaluation and retrofit design. • Several potential deficiencies are likely associated with detailing requirements for reinforcing steel [reinforcement ratio, maximum spacing limits, reinforcing around openings, reinforcing hooks at slab to wall connections (see Figure 3.153) and foundation dowels (see Figure 3.154)]. • At the operating floor level, large stair openings are located adjacent to three of the four shear walls, limiting the connection length to transfer seismic forces from the floor slab to the concrete walls.
Nonstructural	• Water system piping within the control building is not seismically braced (see Figure 3.155). • Valves and valve actuators in line with the water system piping are not braced (see Figure 3.156). • The vertical pump bells and valve operator riser shafts are not braced (see Figure 3.157). • Per the original drawings and site visit observations, piping and valves within the control building are cast-iron (see Figure 3.158), which is a brittle material that may crack when subjected to earthquake shaking-induced forces and/or ground deformation. • Significant corrosion-induced deterioration was observed for some piping, valves, and pipe connection bolts in the control building (see Figure 3.159). • Pumps do not appear to be anchored at the base (see Figure 3.160). • Vertical pump motors are not braced above the center of gravity of the motor (see Figure 3.160). • There does not appear to be adequate flexibility in the piping that is attached to the pumps to accommodate potential relative movement between the piping and the pump, and to prevent the piping from adding additional load to the pump anchorage. • The air vent vertical pipe adjacent to the east reservoir access stair does not appear to be braced (see Figure 3.161).

Table 3.17 Fairmount Reservoir Control Building Evaluation Summary (cont.)

Potential Deficiencies	Description
Nonstructural (cont.)	• Anchorage of electrical cabinets to the concrete slab was not visible from the outside of the cabinets and may not be adequate. Additionally, electrical cabinets do not appear to be anchored or braced to the wall near the top of the cabinets to prevent them from tipping over during an earthquake (see Figure 3.162) • Electrical conduits, hung from the roof, penetrating the wall and connected to the top of floor-mounted electrical cabinets, may not have adequate flexibility to account for differential movement between the floor, walls, and roof (see Figure 3.163). • The horizontal arm of the SCADA antenna may not be adequately connected to the supporting pole to prevent the antenna from being misaligned or damaged after an earthquake (see Figure 3.164). • The wood pole and anchorage of the pole-mounted transformer to the pole may not be adequately designed to resist the seismic forces generated by the transformer (see Figure 3.165).

Figure 3.148 Fairmount Reservoir

Figure 3.149 Fairmount Reservoir Control Building

Figure 3.150 Fairmount Reservoir Roof Expansion Joints

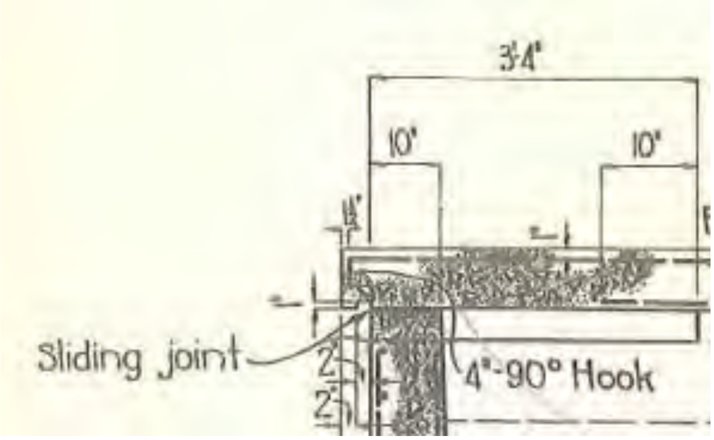


Figure 3.151 Sliding Joint Between Wall and Roof
(Source: Section L-L on Sheet 8 of 1936 design drawings by Stevens & Koon)

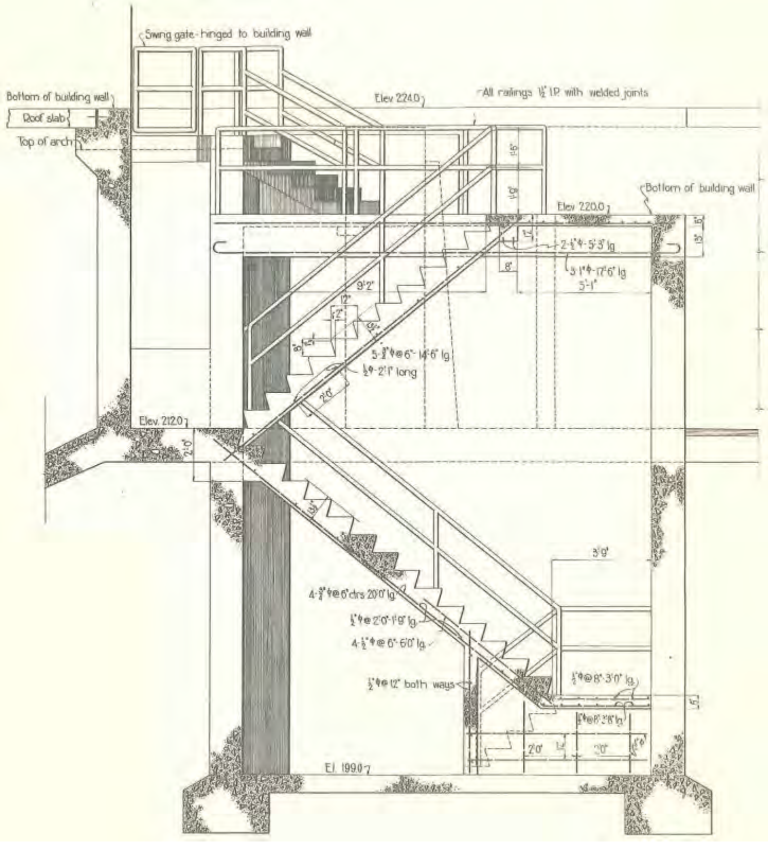


Figure 3.152 Reservoir Adjacent to Control Building/Pump Station
(Source: Section H-H on Sheet 11 of 1936 design drawings by Stevens & Koon)

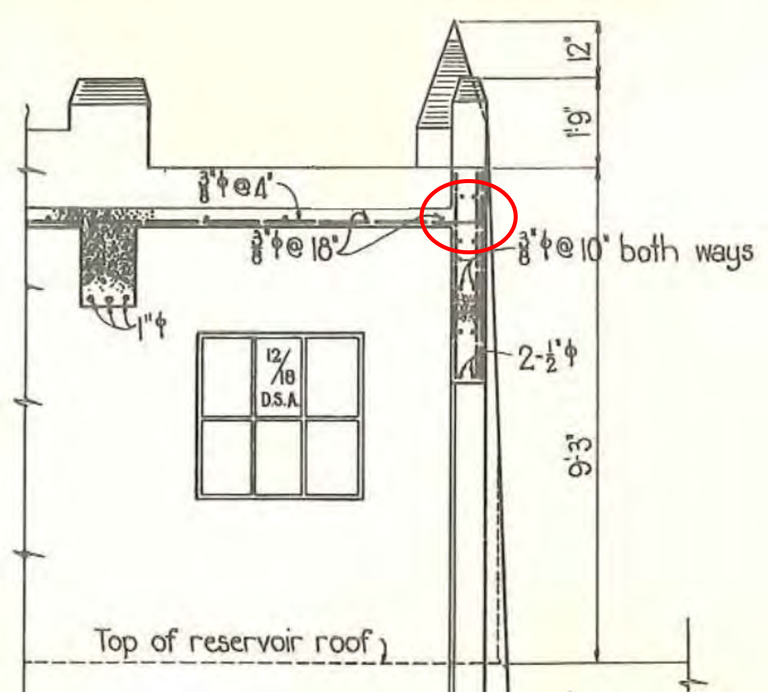


Figure 3.153 Inadequate Shear Wall to Diaphragm Connection
(Source: Section a-a on Sheet 16 of 1936 design drawings by Stevens & Koon)

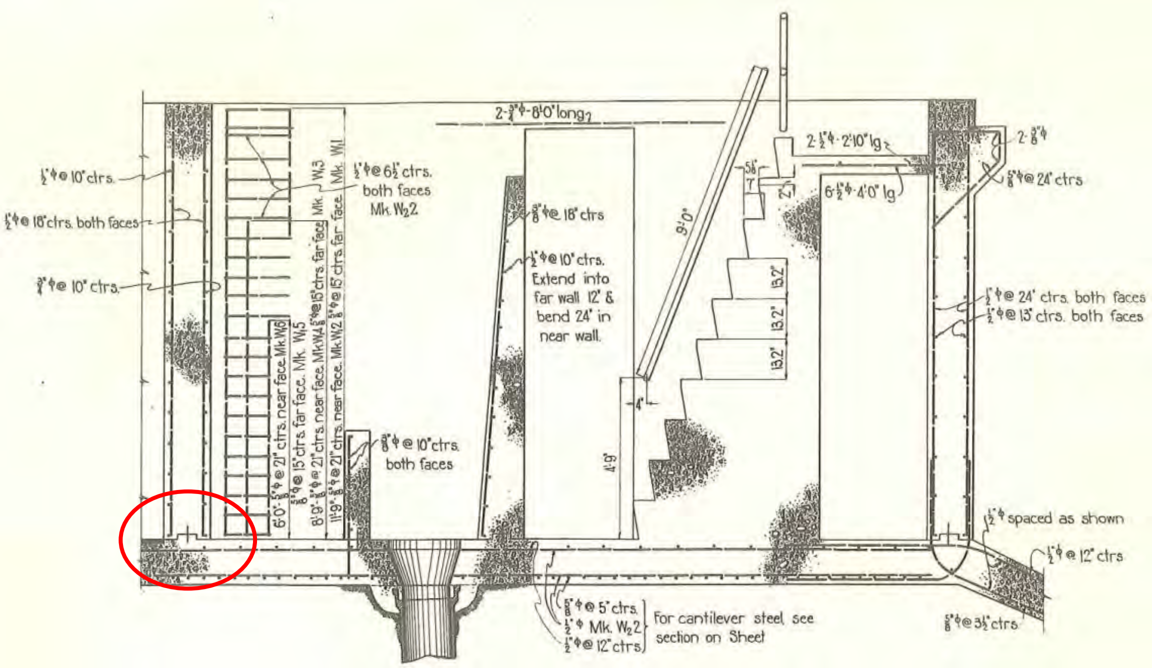


Figure 3.154 Inadequate Shear Wall to Foundation Connection
(Source: Section J-J on Sheet 12 of 1936 design drawings by Stevens & Koon)

Figure 3.155 Unbraced Piping and Valves

Figure 3.156 Unbraced Valve

Figure 3.157 Unbraced Pump Bells and Valve Operator Riser Shafts

Figure 3.158 Cast Iron Valve

Figure 3.159 Corroded Bolts and Pipe

Figure 3.160 Unanchored Pump

Figure 3.161 Unbraced Vent Pipe

Figure 3.162 Electrical Cabinets with Unknown Anchorage Details

Figure 3.163 Conduits Connecting to Electrical Cabinets without Flexible Connections

Figure 3.164 SCADA Antenna

Figure 3.165 Pole-mounted Electrical Transformers

3.2.5 Grice Hill Reservoir and Control Building

The Grice Hill Reservoir (see Figure 3.166) is a 2.2 MG tank built in 2001 at the Grice Hill site off 27th Place NW. This reservoir is a strand-wound, circular, prestressed concrete reservoir with a nearly flat roof, an approximate diameter of 140 feet and a maximum height of retained water of 20 feet. The Grice Hill Reservoir serves the City's W2 Level and is supplied by the Mountain View Pump Station.

The Grice Hill Reservoir Control Building (see Figure 3.167) is located west of the reservoir. The control building is an above-grade, single-story structure, with reinforced masonry shear walls and a plywood sheathed wood truss roof. The building is rectangular in plan, with approximate dimensions of 45 feet in north-south direction by 37 feet in east-west direction. The building houses piping and valves (including seismic shutoff valves) that support the operation of the reservoir. The original design of the control building included a chlorination room and associated equipment, but the chlorination system is not currently used. Therefore, the chlorination system has not been included in the scope of this nonstructural assessment. The SCADA antenna at the Grice Hill Reservoir site is supported by a tall lattice tower (see Figure 3.168) and has not been included in the scope of the nonstructural assessment.

Table 3.18 and Table 3.19 present a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.18, the Grice Hill Reservoir is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Based on the potential deficiencies identified in Table 3.19, the Grice Hill Reservoir Control Building is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Similarly, based on the structural deficiencies identified, the Grice Hill Reservoir Control Building is not currently expected to achieve Life Safety structural performance and represents a potential safety hazard to City staff and contractors.

Table 3.18 Grice Hill Reservoir Evaluation Summary

Potential Deficiencies	Description
Structural	• The column vertical reinforcing lap splice length is less than the ASCE 41-17 Tier 1 specified minimum length for Immediate Occupancy structural performance (i.e., 50 bar diameters).
Nonstructural	• The pipe support concrete pedestals within the reservoir do not appear to be positively connected to the reservoir floor. Original design drawings show that the vertical reinforcing at the corners of the pedestal terminates at the top of the reservoir floor (see Figure 3.169).

Table 3.19 Grice Hill Reservoir Control Building Seismic Evaluation Summary

Potential Deficiencies	Description
Structural	• The roof diaphragm spans in both directions exceed the ASCE 41-17 Tier 1 limit for unblocked wood structural panel diaphragms. • The configuration of the toenail connection provided between the sloped truss blocking and masonry wall top plate (see Figure 3.170) may have resulted in splitting of the blocking, corner of the top plate, or both. Therefore, the load path may not be adequate to transfer in-plane shear forces from the roof diaphragm to the masonry walls. • Out-of-plane bracing of the north and south gable end masonry walls is not adequate. Kicker braces are provided between the top of the masonry walls and roof diaphragm (see Figure 3.171). However, no positive connection is indicated between the blocking that the kicker braces frames into and the roof trusses. Therefore, the load path is incomplete to resist the vertical component of the kicker brace force associated with providing out-of-plane bracing for the gable end masonry walls.
Nonstructural	• Water system piping within the control building is not adequately seismically braced (see Figure 3.172). • Valves in line with the water system piping are not braced (see Figure 3.173). • One of the seismic valves has a note attached indicating that the valve is out of service (see Figure 3.174). • The pressure tank for the irrigation system is not anchored at the base (see Figure 3.175). • Backup batteries in the battery cabinet (for operation of the seismic valves) are not adequately restrained (see Figure 3.176). • The ceiling framing of the chlorine room inside the control building does not appear to provide adequate connections to transfer seismic forces from the ceiling to the masonry walls below (see Figure 3.177). • The temporarily stored electrical cabinet (see Figure 3.178), emergency generator, etc. may tip over or slide during an earthquake and potentially damage valves or other components. • A ladder is unrestrained (see Figure 3.179) and may topple into and potentially damage valves or other components during an earthquake. • No anchorage was observed between the electrical transformer and concrete support pad (see Figure 3.180).

Figure 3.166 Grice Hill Reservoir

Figure 3.167 Grice Hill Reservoir Control Building

Figure 3.168 SCADA Antenna Supported by Lattice Tower

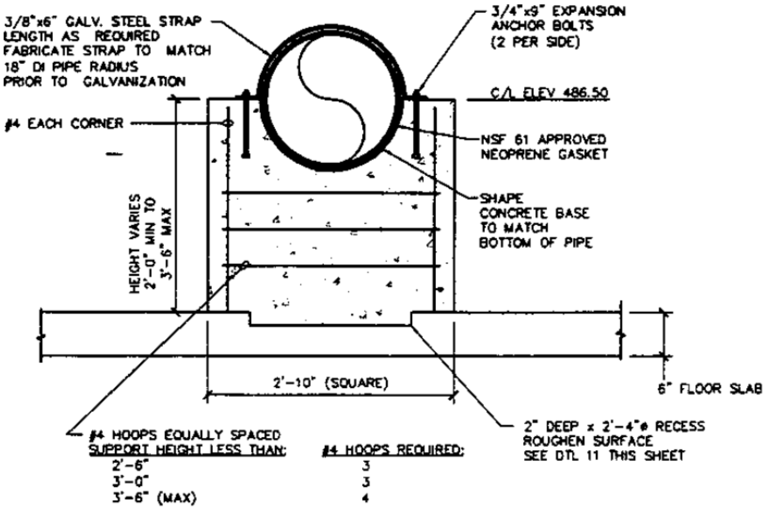


Figure 3.169 Reservoir Pipe Support Detail
(Source: Detail 10 on Sheet S6 of 2001 design drawings by Westech Engineering)

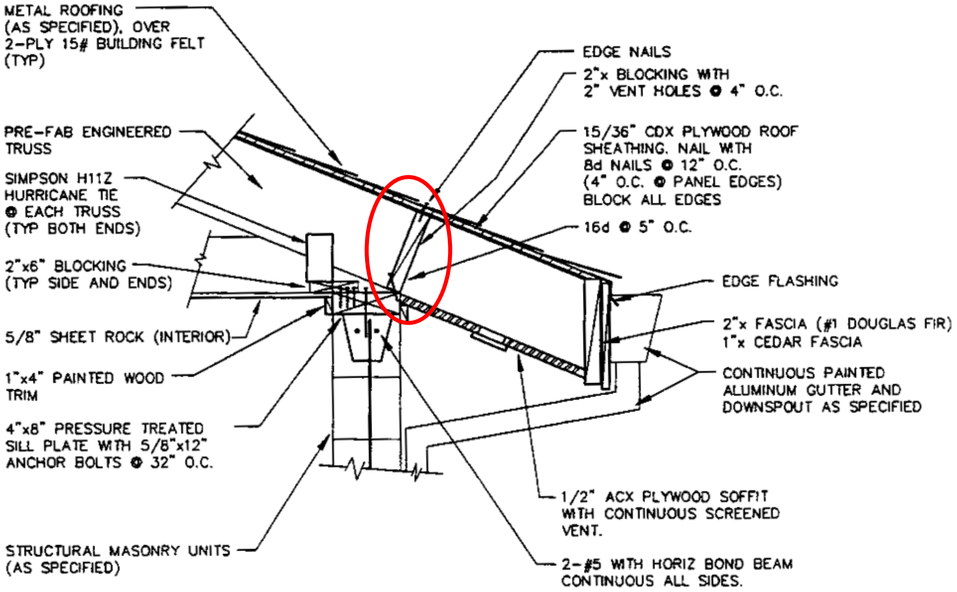


Figure 3.170 Inadequate Connection between Blocking and Masonry Wall Top Plate (Source: Detail 6 on Sheet B-5 of 2001 design drawings by Westech Engineering)

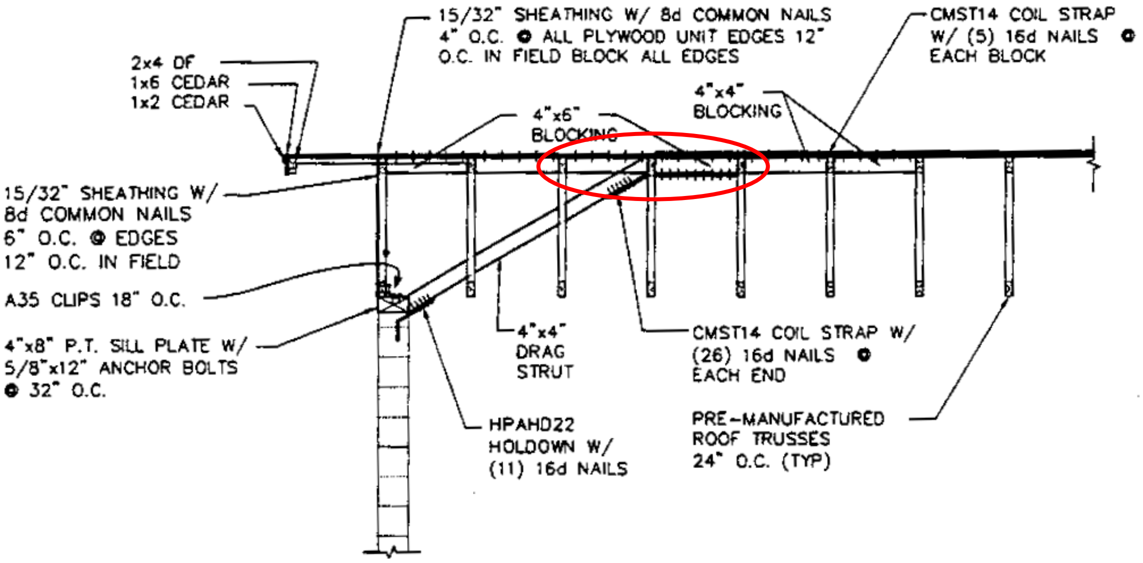


Figure 3.171 Inadequate Connection between Blocking and Roof Truss (Source: Detail 4 on Sheet B-5 of 2001 design drawings by Westech Engineering)

Figure 3.172 Unbraced Piping and Valves

Figure 3.173 Unbraced Seismic Valve

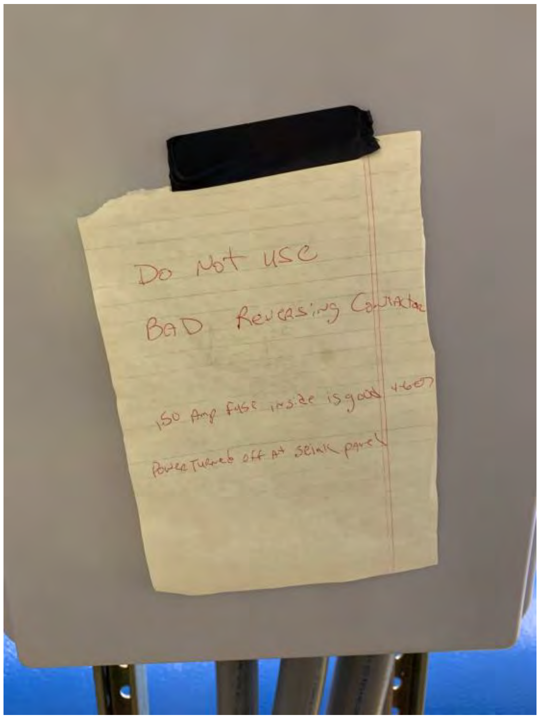


Figure 3.174 Note about Inoperable Seismic Valve

Figure 3.175 Unanchored Irrigation Pressure Tank

Figure 3.176 Backup Batteries without Adequate Restraint

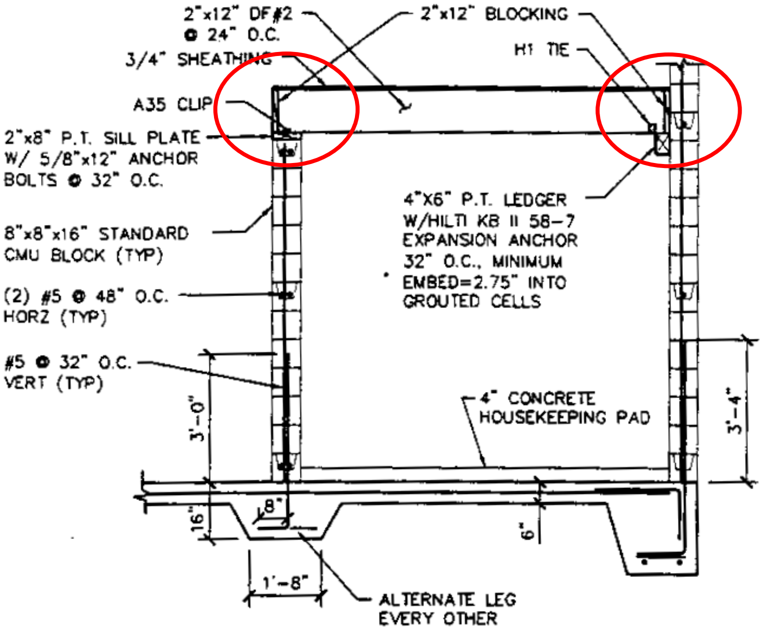


Figure 3.177 Chlorine Room Ceiling Framing
(Source: Detail 1 on Sheet B-5 of 2001 design drawings by Westech Engineering)

Figure 3.178 Temporarily Stored Electrical Cabinet

Figure 3.179 Unrestrained Ladder

Figure 3.180 Unanchored Electrical Transformer

3.2.6 Lone Oak Reservoir and Control Building

The Lone Oak Reservoir (see Figure 3.181) is a 5.6 MG tank built in 2003 near the intersection of Lone Oak Road SE and Midred Lane SE. This reservoir is a strand-wound, circular, prestressed concrete reservoir with a nearly flat roof with an approximate diameter of 196 feet and a maximum height of retained water of 26 feet. The Lone Oak Reservoir serves the City’s S2 Level.

The Lone Oak Reservoir Control Building (see Figure 3.182) is located west of the reservoir. The control building is a single-story building with reinforced concrete walls below grade, reinforced masonry walls above grade, and a plywood sheathed wood truss roof. The building is rectangular in plan, with approximate dimensions of 39 feet in north-south direction by 29 feet in east-west direction. The building houses piping and valves (including a seismic shutoff valve) that support the operation of the reservoir. The original design of the control building included a chlorination room and associated equipment, but the chlorination system is not currently used. Therefore, the chlorination system has not been included in the scope of the nonstructural assessment (with the exception of the large hot water heater).

Table 3.20 and Table 3.21 present a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.20, the Lone Oak Reservoir is expected to achieve Immediate Occupancy structural performance and Operational nonstructural performance for a M9.0 CSZ earthquake. Based on the potential deficiencies identified in Table 3.21, the Lone Oak Reservoir Control Building may not achieve Immediate Occupancy structural performance and is not expected to achieve Operational nonstructural performance for a M9.0 CSZ earthquake. Similarly, based on the structural deficiencies identified, the Lone Oak Reservoir Control Building may not achieve Life Safety structural performance and may represent a potential safety hazard to City staff and contractors.

Table 3.20 Lone Oak Reservoir Evaluation Summary

Potential Deficiencies	**Description**
Structural	• None identified.
Nonstructural	• None identified.

Table 3.21 Lone Oak Reservoir Control Building Evaluation Summary

Potential Deficiencies	Description
Structural	• The original design drawings indicate that the design of the Lone Oak Reservoir Control Building masonry walls and roof structure was a deferred submittal item (see Figure 3.183). The deferred submittal drawings/calculations were not available for review as part of this project and the roof framing was not visible during SEFT's site visit. Based on the limited information available, the expected structural performance of the Lone Oak Reservoir Control Building could not be quantified.
Nonstructural	• Water system piping within the control building is not seismically braced (see Figure 3.184). • Valves in line with the water system piping are not braced (see Figure 3.185). • The pressure tank for the irrigation system is not anchored at the base (see Figure 3.186). • Anchorage of the control cabinet to the housekeeping pads was not visible from the outside of the cabinets and may not be adequate. Additionally, the control cabinet does not appear to be anchored or braced to the wall near the top of the cabinet to prevent it from tipping over during an earthquake (see Figure 3.187). • Anchorage of the battery cabinet to the top of the control cabinet (see Figure 3.187) was not visible from the outside of the cabinet and may not be adequate. Also, backup batteries in the battery cabinet may not be adequately restrained. • The horizontal arm of the SCADA antenna may not be adequately connected to the supporting pole to prevent the antenna from being misaligned or damaged after an earthquake (see Figure 3.188). • The suspended HVAC unit may not be adequately braced (see Figure 3.189). Potential bracing deficiencies include the bracing angle for one pair of cable braces is near vertical (resulting in a significant decrease in the capacity of the braces to resist horizontal seismic forces), some braces appear to load the bottom chord of the roof truss in the out-of-plane direction (blocking or other detailing to deliver these seismic forces to the roof diaphragm are unknown) and only a single cable clamp is used for the braces (no redundance if the single clamp were to loosen) • The base skid for the large water heater and safety shower in the chlorine room does not appear to be adequately anchored (see Figure 3.190).

Table 3.21 Lone Oak Reservoir Control Building Evaluation Summary (cont.)

Potential Deficiencies	Description
Nonstructural (cont.)	• The original design drawings indicate that the design of the chlorine room masonry walls and top of wall bracing was a deferred submittal item (see Figure 3.191). The deferred submittal drawings/calculations were not available for review as part of this project and the bracing at the top of these masonry walls was not visible during SEFT's site visit. Therefore, the adequacy of the masonry wall bracing is unknown. • A ladder is unrestrained (see Figure 3.192) and may topple into and potentially damage valves or other components during an earthquake. • No anchorage was observed between the electrical transformer and concrete support pad (see Figure 3.193).

Figure 3.181 Lone Oak Reservoir

Figure 3.182 Lone Oak Reservoir Control Building

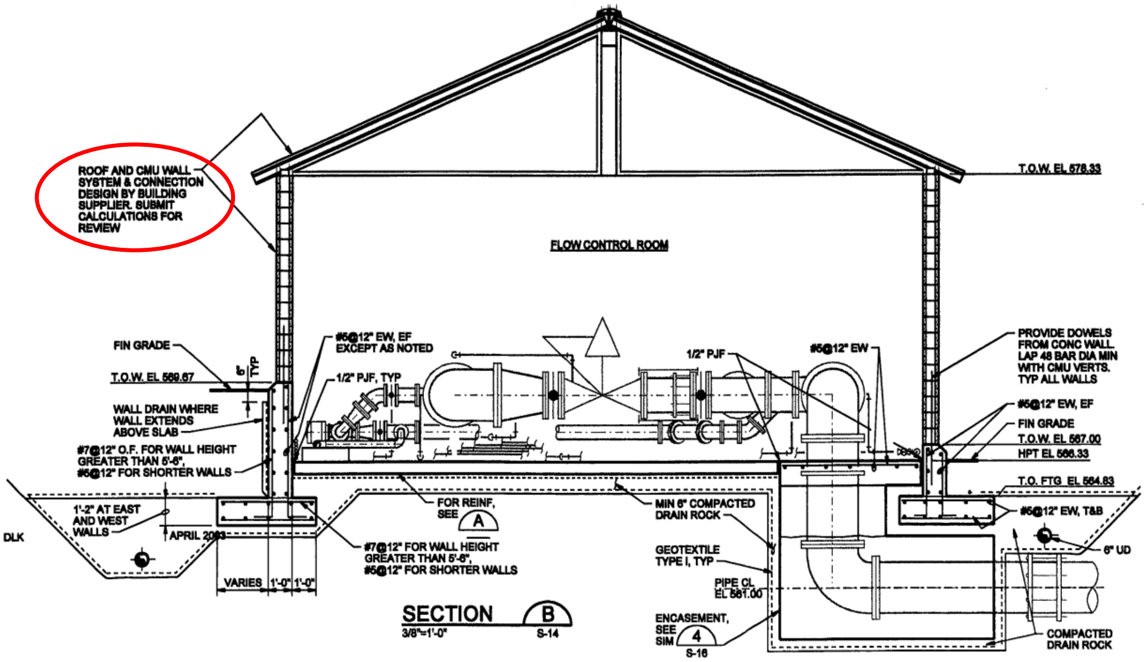


Figure 3.183 Design of Lone Oak Reservoir Control Building was a Deferred Submittal (Source: Section B on Sheet B-15 of 2003 design drawings by CH2M Hill)

Figure 3.184 Unbraced Piping and Valves

Figure 3.185 Unbraced Seismic Valve

Figure 3.186 Unanchored Pressure Tank

Figure 3.187 Control Cabinet with Unknown Anchorage Details

Figure 3.188 SCADA Antenna

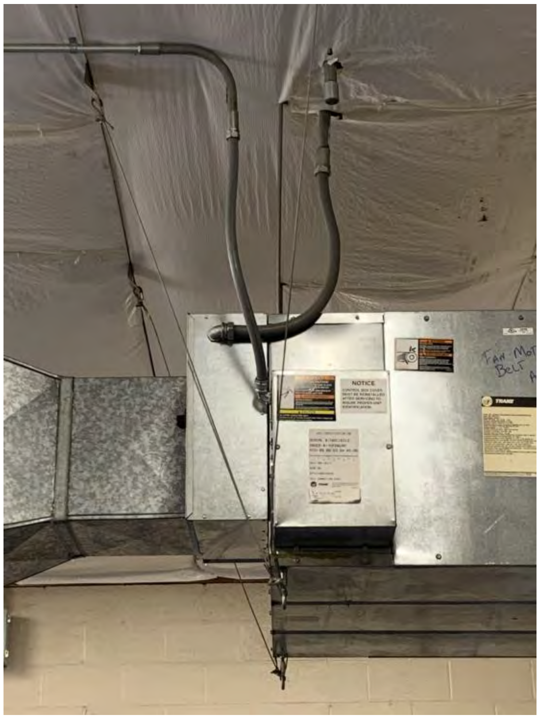


Figure 3.189 Suspended HVAC Unit

Figure 3.190 Inadequate Overturning Anchorage of Water Heater

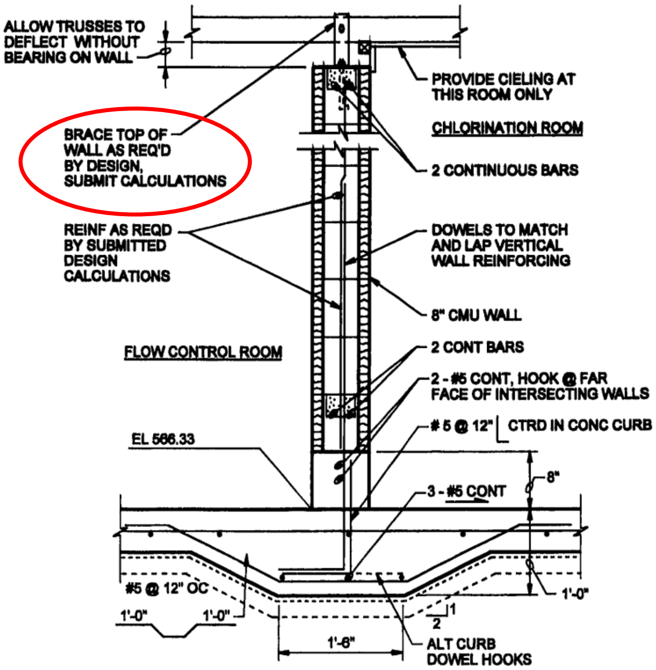


Figure 3.191 Chlorine Room Ceiling Framing
(Source: Section C on Sheet B-15 of 2003 design drawings by CH2M Hill)

Figure 3.192 Unrestrained Step Ladder

Figure 3.193 Unanchored Electrical Transformer

3.2.7 Mill Creek #1 Reservoir and Control Building

The Mill Creek #1 Reservoir (see Figure 3.194) is a 2.2 MG tank built in 2013 at the Mill Creek #1 site off Deer Park Drive SE. This reservoir is a strand-wound, circular, prestressed concrete reservoir with a nearly flat roof, an approximate diameter of 140 feet and a maximum height of retained water of 20 feet. The Mill Creek #1 Reservoir serves the City's S1 Level and is supplied by the Deer Park Pump Station.

The Mill Creek #1 Reservoir Control Building (see Figure 3.195) is located southwest of the reservoir. The control building is a single-story building with reinforced concrete walls below grade, reinforced masonry walls above grade, and a plywood sheathed wood truss roof. The building is rectangular in plan, with approximate dimensions of 46 feet in north-south direction by 42 feet in east-west direction. The building houses piping and valves (including seismic shutoff valves) that support the operation of the reservoir and a small pump station that supports the City's College Reservoir (steel tank). The original design of the control building included a chlorination room and associated equipment, but the chlorination system is not currently used. Therefore, the chlorination system has not been included in the scope of the nonstructural assessment.

Table 3.22 and Table 3.23 present a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.22, the Mill Creek #1 Reservoir is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Based on the potential deficiencies identified in Table 3.23, the Mill Creek #1 Reservoir Control Building is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake. Similarly, based on the structural deficiencies identified, the Mill Creek #1 Reservoir Control Building is not currently expected to achieve Life Safety structural performance and represents a potential safety hazard to City staff and contractors.

Table 3.22 Mill Creek #1 Reservoir Evaluation Summary

Potential Deficiencies	Description
Structural	• The column vertical reinforcing lap splice length is less than the ASCE 41-17 Tier 1 specified minimum length for Immediate Occupancy structural performance (i.e., 50 bar diameters).

Table 3.22 Mill Creek #1 Reservoir Evaluation Summary (cont.)

Potential Deficiencies	Description
Nonstructural	• The pipe support concrete pedestals within the reservoir do not appear to be positively connected to the reservoir floor. Original design drawings show that the vertical reinforcing at the corners of the pedestal terminates at the top of the reservoir floor (see Figure 3.196). • The steel framed roof access stair located on the southeast side of the reservoir is relatively flexible (see Figure 3.197). During an earthquake, the stair will likely pound against the reservoir and may damage the stair or locally damage the concrete reservoir.

Table 3.23 Mill Creek #1 Reservoir Control Building Evaluation Summary

Potential Deficiencies	Description
Structural	• The roof diaphragm spans in both directions exceed the ASCE 41-17 Tier 1 limit for unblocked wood structural panel diaphragms. • The original design drawings indicate that the truss manufacturer was to provide truss blocking capable of transferring shear loads from the roof diaphragm to the masonry wall top plate (see Figure 3.198). The deferred submittal drawings/calculations were not available for review as part of this project and this area was not visible during SEFT's site visit. Therefore, the adequacy of the truss blocking is unknown. • Out-of-plane bracing of the north and south gable end masonry walls is not adequate. Three bays of blocking are provided to transfer out-of-plane wall bracing forces to the ceiling level plywood sheathed diaphragm (see Figure 3.199). This blocking does not engage an adequate depth of the ceiling level diaphragm.

Table 3.23 Mill Creek #1 Reservoir Control Building Evaluation Summary (cont.)

Potential Deficiencies	Description
Nonstructural	• Water system piping within the control building is not adequately seismically braced (see Figure 3.200). • Valves in line with the water system piping are not braced (see Figure 3.201). • The pressure tank for the irrigation system is not anchored at the base (see Figure 3.202). • Vertical pump motors are not braced above the center of gravity of the motors (see Figure 3.203). • There does not appear to be adequate flexibility in the piping that is attached to the pumps to accommodate potential relative movement between the piping and the pump, and to prevent the piping from adding additional load to the pump anchorage. • Backup batteries in the battery cabinet (for operation of the seismic valves) may not be adequately restrained. • The horizontal arm of the SCADA antenna may not be adequately connected to the supporting pole to prevent the antenna from being misaligned or damaged after an earthquake (see Figure 3.204). • The ceiling framing of the chlorine room inside the control building does not appear to provide adequate connections to transfer seismic forces from the ceiling to the masonry walls below. Also, the wood ledger attachment to the masonry wall is not detailed to avoid cross-grain bending (see Figure 3.205). • The electrical “room” partial height masonry walls are not laterally braced (see Figure 3.206). • Two ladders are unrestrained (see Figure 3.207) and may topple into and potentially damage valves or other components during an earthquake. • No anchorage was observed between the electrical transformer and concrete support pad (see Figure 3.208).

Figure 3.194 Mill Creek #1 Reservoir

Figure 3.195 Mill Creek #1 Reservoir Control Building

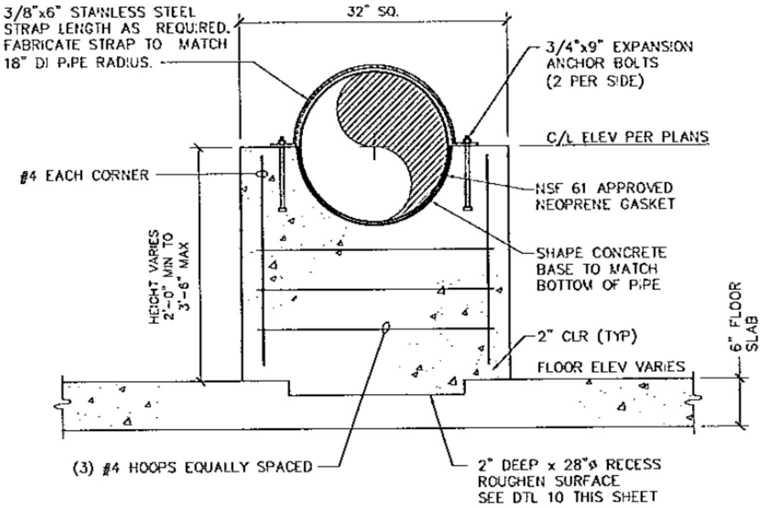


Figure 3.196 Reservoir Pipe Support Detail
(Source: Detail 9 on Sheet S4 of 2014 design drawings by Westech Engineering)

Figure 3.197 Mill Creek #1 Reservoir Roof Access Stair

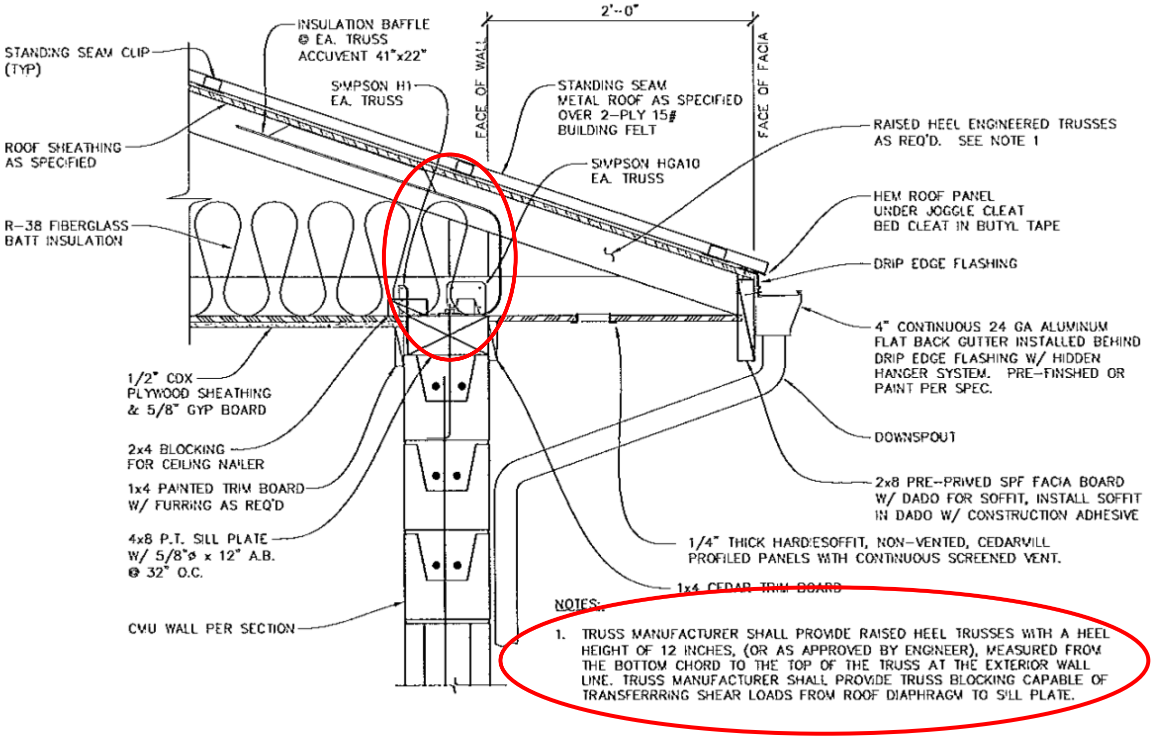


Figure 3.198 Inadequate Connection between Blocking and Masonry Wall Top Plate (Source: Detail 5 on Sheet S-20 of 2014 design drawings by Westech Engineering)

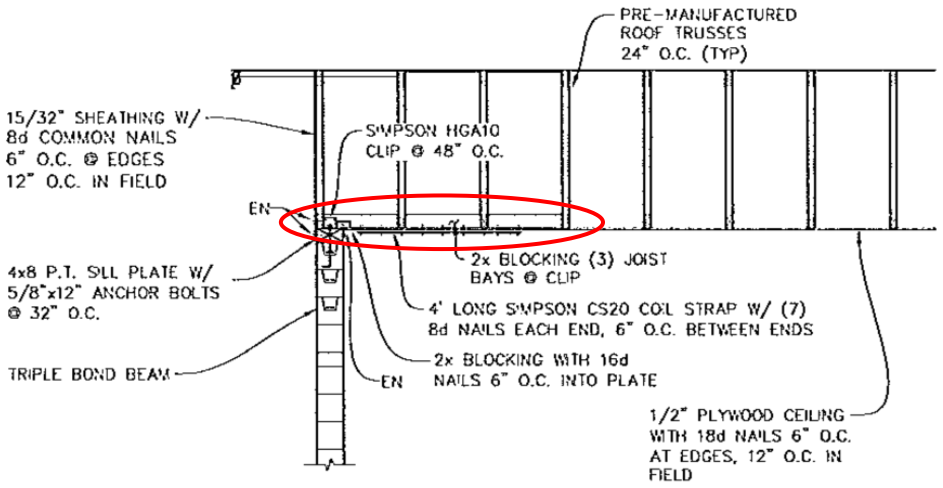


Figure 3.199 Inadequate Transfer Length between Blocking and Ceiling Diaphragm (Source: Detail 1 on Sheet S-20 of 2014 design drawings by Westech Engineering)

Figure 3.200 Unbraced Piping and Valves

Figure 3.201 Unbraced Seismic Valves

Figure 3.202 Unanchored Pressure Tank

Figure 3.203 Unanchored Pump Motor

Figure 3.204 SCADA Antenna

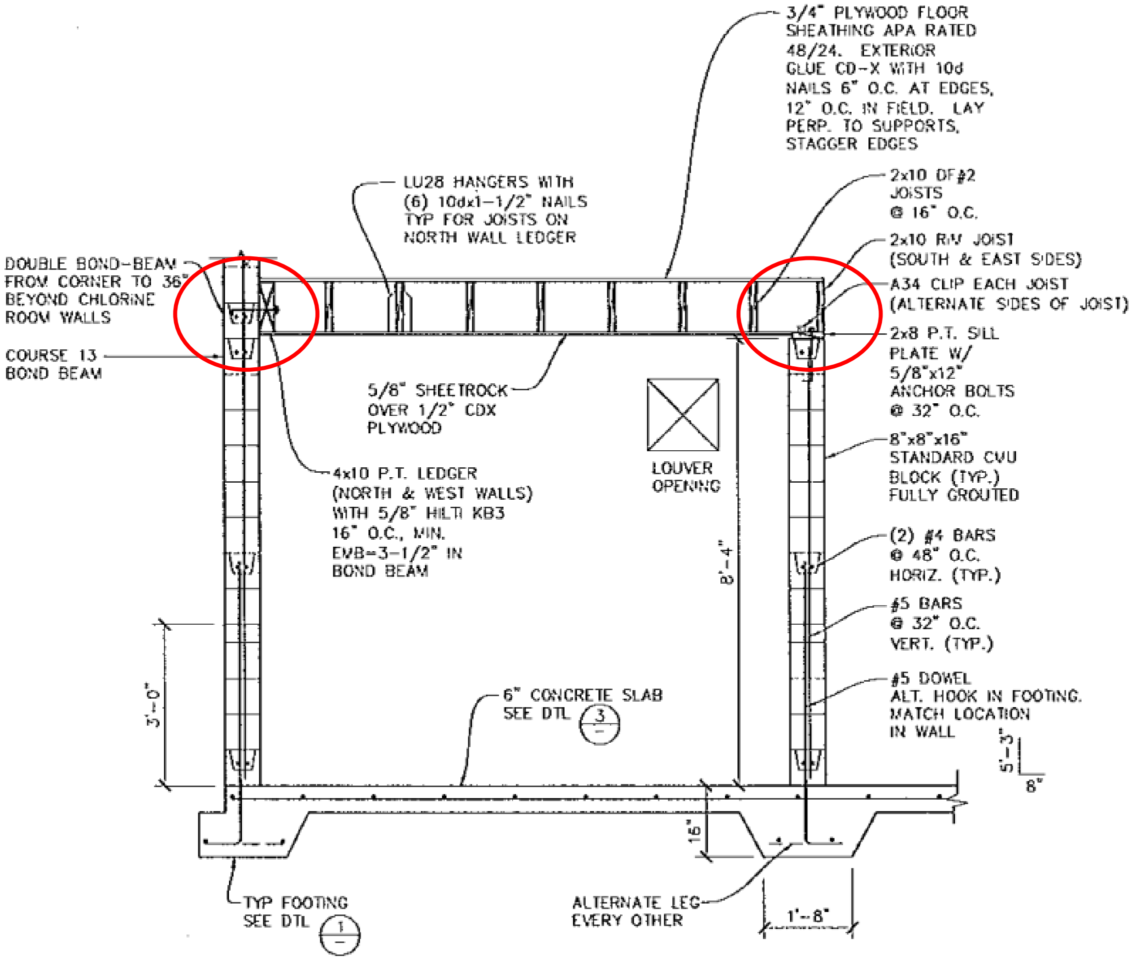


Figure 3.205 Chlorine Room Ceiling Framing
(Source: Section C on Sheet S-17 of 2014 design drawings by Westech Engineering)

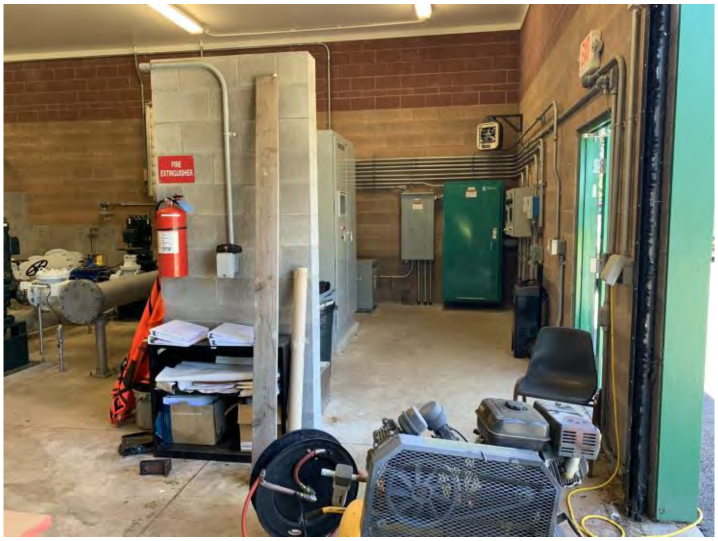


Figure 3.206 Unbraced Partial Height CMU Walls in Electrical Room

Figure 3.207 Unrestrained Ladders

Figure 3.208 Unanchored Electrical Transformer

3.2.8 Mountain View Reservoir

The Mountain View Reservoir (see Figure 3.209) is a 10 MG tank built in 1971 near the intersection of Wallowa Avenue NW and Orchard Heights Road NW. This reservoir is a completely buried strand-wound, circular, prestressed concrete reservoir with a nearly flat roof, an approximate diameter of 292 feet and a maximum height of retained water of 20 feet. The Mountain View Reservoir serves the City's G0 Level and is hydraulically connected to both Franzen and Fairmount Reservoirs.

In 2008, Black & Veatch conducted a condition assessment and seismic evaluation of Mountain View Reservoir. SEFT reviewed the report associated with the 2008 condition assessment and seismic evaluation to help inform our seismic assessment.

Table 3.24 presents a summary of potential seismic structural and nonstructural deficiencies identified by this evaluation. Based on the potential deficiencies identified in Table 3.24, the Mountain View Reservoir is not expected to achieve Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ earthquake.

Table 3.24 Mountain View Reservoir Evaluation Summary

Potential Deficiencies	Description
Structural	• No seismic cables or dowels were used to connect the base of the wall to the foundation (see Figure 3.210). Shear forces are only transferred from the wall to foundation by friction, which is likely inadequate to resist the earthquake-induced lateral force. • The existing capacity of the horizontal prestressing strands on the wall of the reservoir is inadequate to resist the combination of hydrostatic and expected hydrodynamic hoop forces during an earthquake. • The column vertical reinforcing lap splice length and tie spacing is less than the ASCE 41-17 Tier 1 specified values for Immediate Occupancy structural performance (i.e., minimum lap splice length of 50 bar diameters and minimum tie spacing of 8 bar diameters)
Nonstructural	• Per the original drawings, some piping and fittings within the Reservoir may be cast-iron, which is a brittle material that may crack when subjected to earthquake shaking-induced forces and/or ground deformation.

Figure 3.209 Mountain View Reservoir

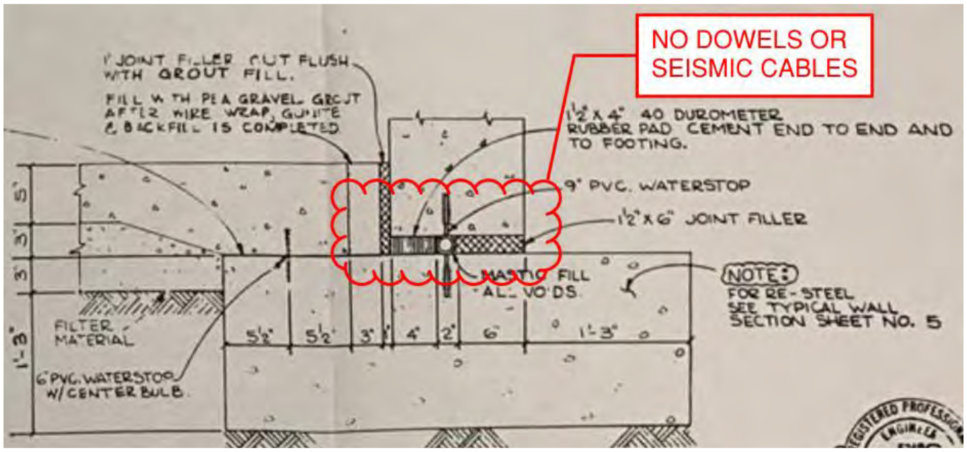


Figure 3.210 Base of Wall to Foundation Connection without Dowels or Seismic Cables (Source: Section C on Sheet 6 of 1971 design drawings by Stevens, Thomsen & Runyan)

4.0 Preliminary Seismic Structural and Nonstructural Mitigation Concepts

4.1 Pump Stations and Control Facilities

This section provides summary tables that describe preliminary seismic retrofit concepts to mitigate the potential structural and nonstructural seismic deficiencies identified for selected City pump stations and control facilities, described in Section 3.1. Where appropriate, these tables also provide recommendations for further investigation and/or analysis to potentially mitigate deficiencies through more detailed structural calculations or to infill gaps in the data that was available for this study.

4.1.1 ASR #1 and #2 Pump Station

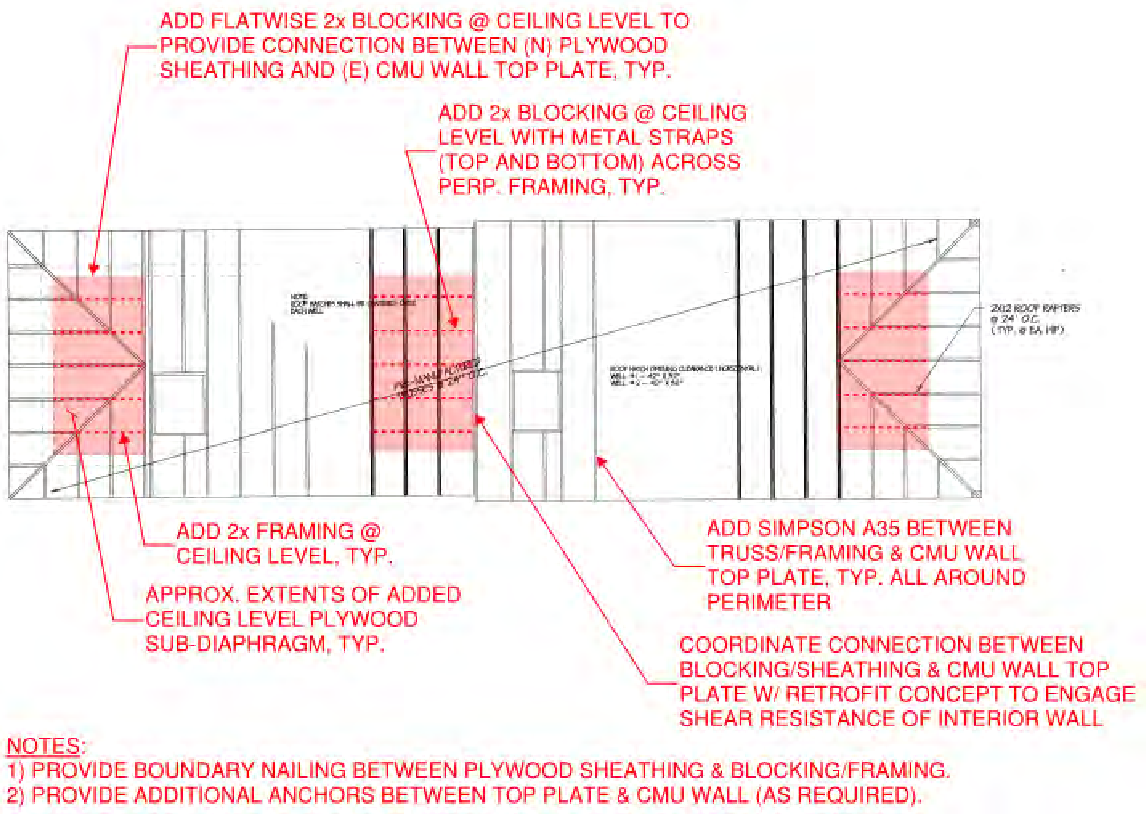
Table 4.1 summarizes preliminary seismic retrofit concepts to mitigate the potential structural and nonstructural deficiencies described in Section 3.1.1 for the ASR #1 and #2 Pump Station.

Table 4.1 ASR #1 and #2 Pump Station Preliminary Mitigation Concepts

Potential Deficiencies	Description
Structural	• Install vertical steel angles where the east-west oriented CMU walls of the ASR #2 addition interface with the west wall of the original ASR #1 structure. • Remove existing gypsum board interior finish to investigate the load path to transfer seismic roof diaphragm forces at the roof step between the ASR #1 and ASR #2 portions of the structure to the masonry wall below. Likely add a combination of plywood sheathing, blocking, and metal connector hardware to provide an adequate load path. • Remove a small section of existing roofing to verify the adequacy of the existing roof sheathing to truss nailing. • Install new shaped blocking between the roof sheathing and masonry wall top plate. Provided boundary nailing between the roof sheathing and blocking, and Simpson A35 clip angles between the blocking and top plate and blocking and trusses. • Install Simpson A35 clips between truss bottom chord and masonry wall top plate in conjunction with cross tie retrofit described in next bullet item.

Table 4.1 ASR #1 and #2 Pump Station Preliminary Mitigation Concepts (cont.)

Potential Deficiencies	Description
Structural (cont.)	• Install a combination of plywood sheathing, blocking, steel straps, and metal connector hardware to provide adequate out-of-plane support for CMU walls (see Figure 4.1). • Conduct nondestructive scanning to verify the size and location of masonry wall vertical reinforcing. If vertical trim reinforcing is not provided adjacent to door openings, install face mounted steel plates through-bolted to masonry wall and anchored to foundation
Nonstructural	• Provide bracing for the piping. • Provide independent bracing for the valves. • Provide independent support and bracing for the air relief valve. • Perform additional analysis to evaluate the adequacy of the existing connection of the motor to the top of the steel motor support, the motor support, and the connection of the motor support to the steel well casing. • Add flexible couplings between the pumps and the connected piping. • Provide anchorage/bracing between the top of the electrical cabinets and wall. • Provide lens covers for light fixtures with safety devices that prevent the lens cover and light bulbs from falling. • Provide flexible couplings where conduits connect to the top of wall-mounted electrical panels and cabinets. • Verify the adequacy of the connection between the horizontal antenna and the supporting pole. • Add helical wall ties between the masonry veneer and the ASR #1 masonry walls. • Conduct nondestructive scanning to verify the size and spacing of reinforcing in the architectural concrete pillars and perform calculations to verify the adequacy of the existing reinforcing. If reinforcing is found to be inadequate, remove existing architectural concrete pillars. • Coordinate with electrical utility to anchor the utility owned electrical transformer to the concrete pad, as required.



ADD FLATWISE 2x BLOCKING @ CEILING LEVEL TO PROVIDE CONNECTION BETWEEN (N) PLYWOOD SHEATHING AND (E) CMU WALL TOP PLATE, TYP.

ADD 2x BLOCKING @ CEILING LEVEL WITH METAL STRAPS (TOP AND BOTTOM) ACROSS PERP. FRAMING, TYP.

ADD 2x FRAMING @ CEILING LEVEL, TYP.

APPROX. EXTENTS OF ADDED CEILING LEVEL PLYWOOD SUB-DIAPHRAGM, TYP.

ADD SIMPSON A35 BETWEEN TRUSS/FRAMING & CMU WALL TOP PLATE, TYP. ALL AROUND PERIMETER

COORDINATE CONNECTION BETWEEN BLOCKING/SHEATHING & CMU WALL TOP PLATE W/ RETROFIT CONCEPT TO ENGAGE SHEAR RESISTANCE OF INTERIOR WALL

NOTES:
1) PROVIDE BOUNDARY NAILING BETWEEN PLYWOOD SHEATHING & BLOCKING/FRAMING.
2) PROVIDE ADDITIONAL ANCHORS BETWEEN TOP PLATE & CMU WALL (AS REQUIRED).

Figure 4.1 Sub-diaphragm Retrofit Concept
(Adapted From: Roof Plan on Sheet A1 of 1997 design drawings by Stettler Company)

4.1.2 ASR #4 Pump Station

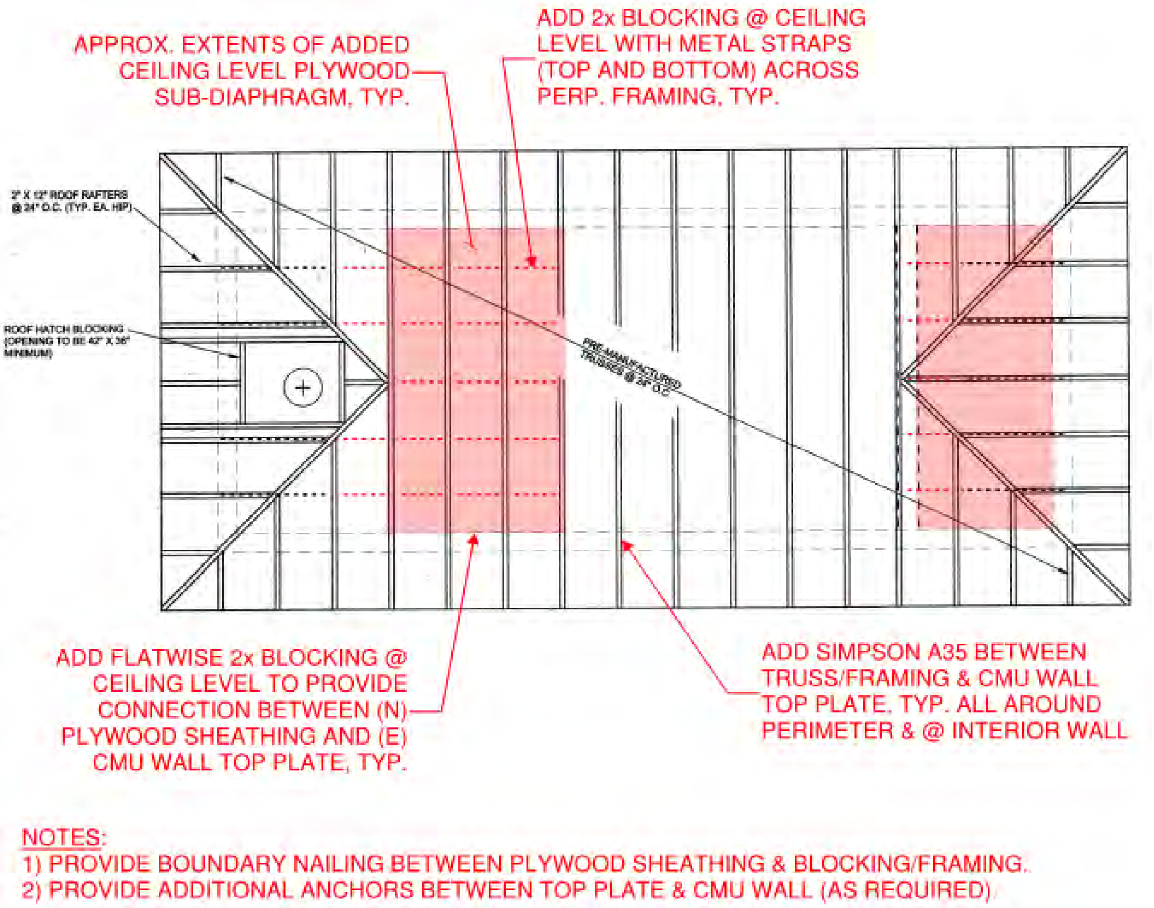
Table 4.2 summarizes preliminary seismic retrofit concepts to mitigate the potential structural and nonstructural deficiencies described in Section 3.1.2 for the ASR #4 Pump Station.

Table 4.2 ASR #4 Pump Station Preliminary Mitigation Concepts

Potential Deficiencies	Description
Structural	• Remove a small section of existing roofing to verify the adequacy of the existing roof sheathing to truss nailing. • Install new shaped blocking between the roof sheathing and masonry wall top plate. Provided boundary nailing between the roof sheathing and blocking, and Simpson A35 clip angles between the blocking and top plate and blocking and trusses. • Perform additional analysis to investigate if the diaphragm has adequate capacity to transfer seismic forces from the roof diaphragm to the east masonry shear wall, considering the impact of the hatch opening adjacent to the wall. Note that this analysis will require information about the existing roof sheathing to truss nailing (i.e., size, and spacing). • Install Simpson A35 clips between truss bottom chord and masonry wall top plate in conjunction with cross tie retrofit described in next bullet item. • Install a combination of plywood sheathing, blocking, steel straps, and metal connector hardware to provide adequate out-of-plane support for CMU walls (see Figure 4.2). • Conduct nondestructive scanning to verify the size and location of masonry wall vertical reinforcing. If vertical trim reinforcing is not provided adjacent to door openings, install face mounted steel plates through-bolted to masonry wall and anchored to foundation.
Nonstructural	• Provide bracing for the piping. • Provide independent bracing for the valves. • Provide independent support and bracing for the air relief valve. • Perform additional analysis to evaluate the adequacy of the existing connection of the motor to the top of the steel motor support, the motor support, and the connection of the motor support to the steel well casing. • Add flexible couplings between the pump and the connected piping.

Table 4.2 ASR #4 Pump Station Preliminary Mitigation Concepts (cont.)

Potential Deficiencies	Description
Nonstructural (cont.)	• Provide anchorage/bracing of pump station control cabinet to floor and wall. • Provide lens covers for light fixtures with safety devices that prevent the lens cover and light bulbs from falling. • Coordinate with electrical utility to anchor the utility owned electrical transformer to the concrete pad, as required.



NOTES:
1) PROVIDE BOUNDARY NAILING BETWEEN PLYWOOD SHEATHING & BLOCKING/FRAMING.
2) PROVIDE ADDITIONAL ANCHORS BETWEEN TOP PLATE & CMU WALL (AS REQUIRED)

Figure 4.2 Sub-diaphragm Retrofit Concept
(Adapted From: Roof Plan on Sheet A2 of 1998 design drawings by Stettler Company)

4.1.3 ASR #5 Pump Station

Table 4.3 summarizes preliminary seismic retrofit concepts to mitigate the potential structural and nonstructural deficiencies described in Section 3.1.3 for the ASR #5 Pump Station.

Table 4.3 ASR #5 Pump Station Preliminary Mitigation Concepts

Potential Deficiencies	Description
Structural	• Add shaped blocking or framing/sheathing with metal connector hardware to provide a load path between the roof of the steel framed pavilion and masonry shear walls of the pump station structure. • Perform an investigation to determine if the ceiling diaphragm is adequately connected to the interior masonry walls to engage the walls as part of the seismic force resisting system. If not, add blocking/framing to provide a load path between the ceiling diaphragm and interior masonry shear walls. • Perform an investigation of the existing ceiling nail size and spacing to verify the adequacy of the existing ceiling sheathing to joist nailing. • Perform an investigation to determine if adequate blocking and connections are provided between the ceiling sheathing and masonry wall top plate. Install new blocking with boundary nailing between the ceiling sheathing and blocking, and Simpson A35 clip angles between the blocking and top plate and blocking and ceiling joists, as appropriate. • Install Simpson A35 clips between ceiling joists and masonry wall top plate in conjunction with cross tie retrofit described in next bullet item. • Install a combination of blocking, steel straps, and metal connector hardware to provide adequate out-of-plane support for CMU walls (see Figure 4.3). • Conduct nondestructive scanning to verify the size and location of masonry wall vertical reinforcing. If vertical trim reinforcing is not provided adjacent to door openings, install face mounted steel plates through-bolted to masonry wall and anchored to foundation. • Perform additional analysis to investigate the adequacy of the free-standing masonry wall to resist seismic forces without additional bracing.

Table 4.3 ASR #5 Pump Station Preliminary Mitigation Concepts (cont.)

Potential Deficiencies	Description
Structural (cont.)	• Investigate extent and severity of the corrosion damage to the steel column, repair damage (as appropriate), and mitigate cause of moisture to prevent similar future damage.
Nonstructural	• Provide bracing for the piping. • Provide independent bracing for the valves. • Provide independent support and bracing for the air relief valve. • Perform additional analysis to evaluate the adequacy of the existing connection of the motor to the top of the steel motor support, the motor support, and the connection of the motor support to the steel well casing. • Add flexible couplings between the pumps and the connected piping. • Provide flexible couplings where conduits connect to the top of floor- and wall-mounted electrical panels and cabinets. • Provide anchorage/bracing of pump station control cabinet to floor and wall. • Provide lens covers for light fixtures with safety devices that prevent the lens cover and light bulbs from falling. • Coordinate with electrical utility to anchor the utility owned electrical transformer to the concrete pad, as required.

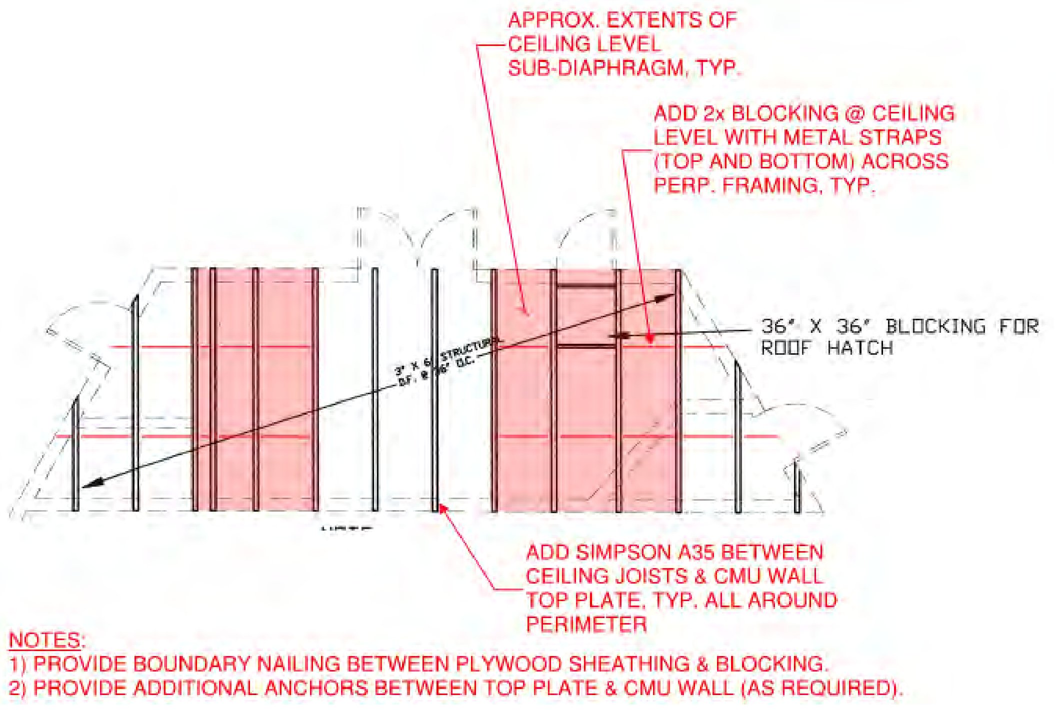


Figure 4.3 Sub-diaphragm Retrofit Concept
(Adapted From: Detail 6 on Sheet A3 of 1997 design drawings by Stettler Company)

4.1.4 Boone Road Pump Station

Table 4.4 summarizes preliminary seismic retrofit concepts to mitigate the potential structural and nonstructural deficiencies described in Section 3.1.4 for the Boone Road Pump Station.

Table 4.4 Boone Road Pump Station Preliminary Mitigation Concepts

Potential Deficiencies	Description
Structural	• Perform an investigation to determine if the gable end framing, sheathing nailing, and connection details are adequate to deliver seismic forces from the upper roof to the lower roof. If not, add supplemental nailing, framing, blocking, and/or metal connector hardware, as appropriate. • Install a wood structural panel overlay on top of the existing straight sheathing. The joints of the wood structural panels should be placed so that they are near the center of the existing sheathing boards or at a 45-degree angle to the joints between existing sheathing boards. • Install a combination of sub-diaphragm framing and connection hardware at the roof level to provide adequate out-of-plane support for CMU walls (see Figure 4.4).
Nonstructural	• Provide bracing for the piping. • Provide independent bracing for the valves. • Perform additional analysis to evaluate the adequacy of the existing connection of the motor to the top of the steel motor support, the motor support, the connection of the motor support to the concrete pedestal, and the concrete pedestal. • Add flexible couplings between the pumps and the connected piping. • Provide longitudinal bracing for the cable tray. Reconfigure the anchorage of the transverse bracing strut to avoid anchorage into the masonry head joints. • Provide lens covers for light fixtures with safety devices that prevent the lens cover and light bulbs from falling. • Provide grating clip connections between the grating and steel support framing. • Verify the adequacy of the antenna connection to the supporting pole.

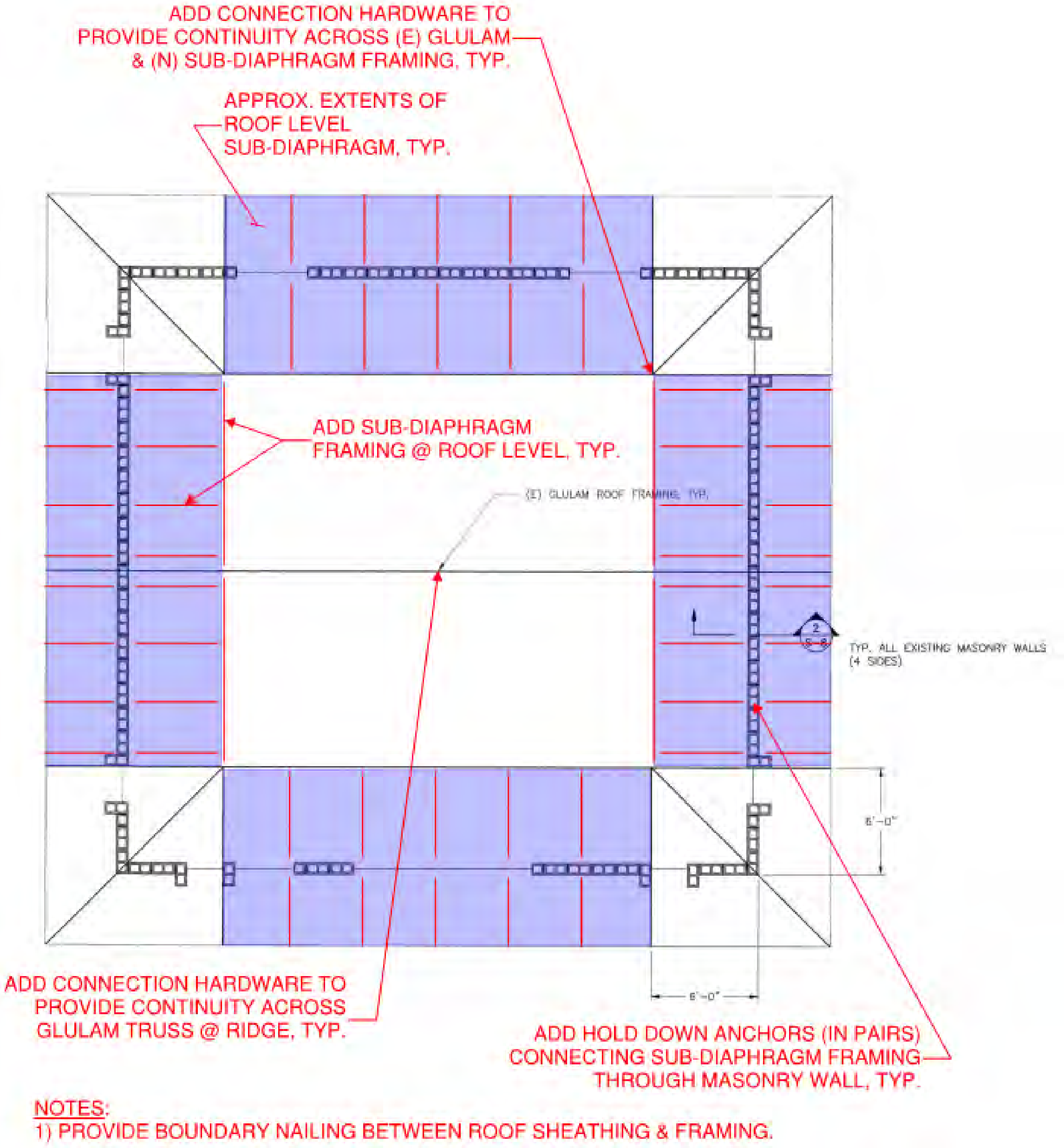


Figure 4.4 Sub-diaphragm Retrofit Concept
(Adapted From: Detail 1 on Sheet S-8 of 2018 design drawings by Murraysmith)

4.1.5 Creekside Pump Station

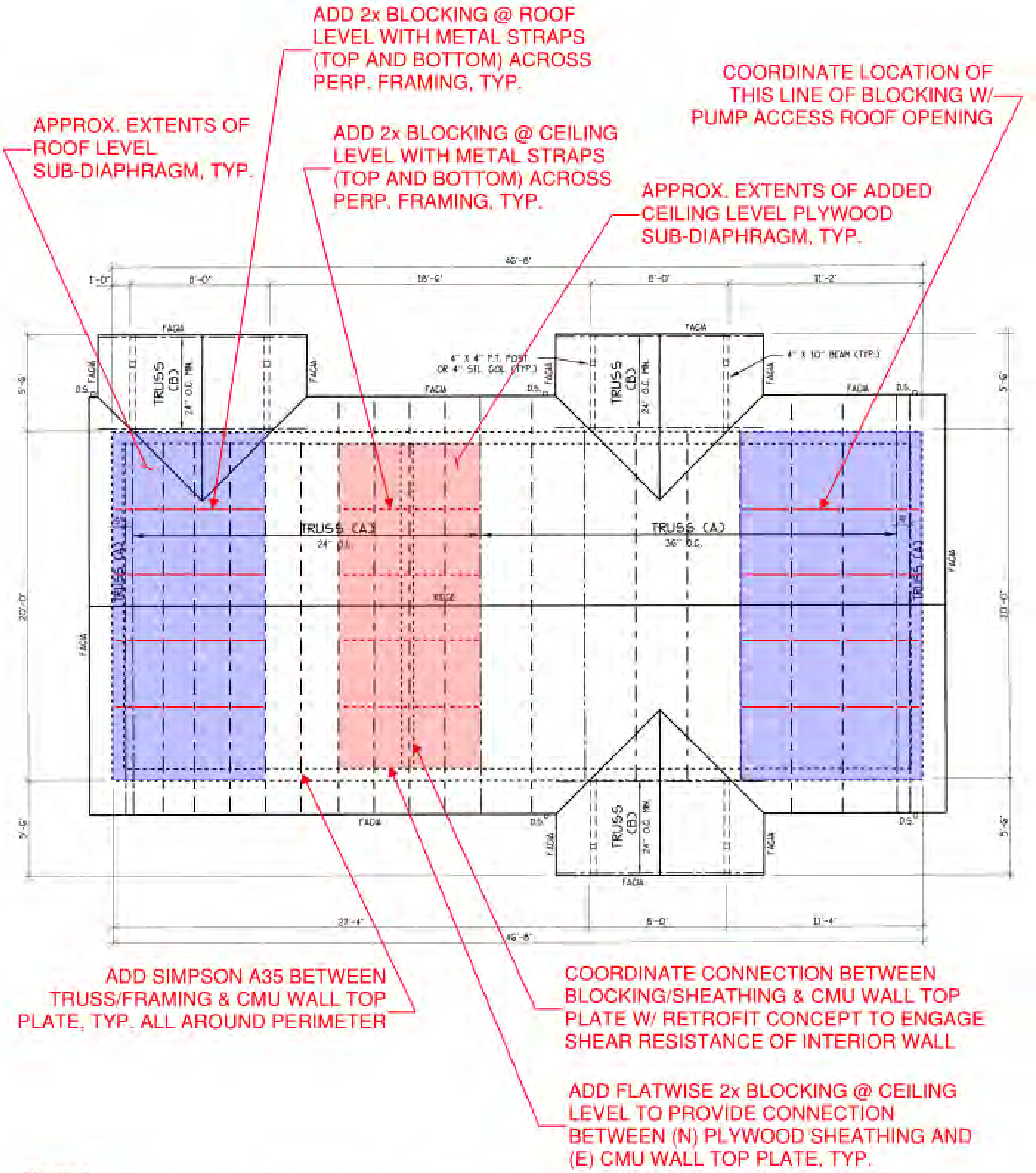
Table 4.5 summarizes preliminary seismic retrofit concepts to mitigate the potential structural and nonstructural deficiencies described in Section 3.1.5 for the Creekside Pump Station.

Table 4.5 Creekside Pump Station Preliminary Mitigation Concepts

Potential Deficiencies	Description
Structural	• Remove a small section of existing roofing to verify the adequacy of the existing roof sheathing to truss nailing. • Add blocking to support the edges of the roof sheathing panels and provided boundary nailing between the roof sheathing and blocking. • Perform an investigation to determine if adequate blocking and connections are provided between the roof sheathing and masonry wall top plate. Install new shaped blocking between the roof sheathing and masonry wall top plate. Provided boundary nailing between the roof sheathing and blocking, and Simpson A35 clip angles between the blocking and top plate and blocking and trusses, as appropriate. Also, suggest engaging the shear resistance of the interior masonry wall by adding a combination of plywood sheathing, framing/blocking, and metal connector hardware to provide a load path between the roof diaphragm and the interior masonry wall. • Perform an investigation to verify the adequacy of the connection between the roof sheathing and gable end masonry wall top plate. • Install Simpson A35 clips between trusses and masonry wall top plate in conjunction with cross tie retrofit described in next bullet item. • Install a combination of plywood, blocking, steel straps, and metal connector hardware to provide adequate out-of-plane support for CMU walls (see Figure 4.5).
Nonstructural	• Provide bracing for the piping. • Provide independent bracing for the valves. • Provide independent support and bracing for the air relief valves. • Perform additional analysis to evaluate the adequacy of the existing connection of the motor to the top of the steel motor support, the motor support, the connection of the motor support to the concrete pedestal, and the concrete pedestal. • Add flexible couplings between the pumps and the connected piping.

Table 4.5 Creekside Pump Station Preliminary Mitigation Concepts (cont.)

Potential Deficiencies	Description
Nonstructural (cont.)	• Perform an investigation to evaluate the adequacy of the anchorage of electrical cabinets to housekeeping pads and housekeeping pads to slab on grade, and supplement anchorage (as required). Also, provide anchorage/bracing between the top of the electrical cabinets and masonry wall. • Provide anchorage/bracing for emergency generator air intake support frame, muffler, and exhaust pipe. • Coordinate with electrical utility to anchor the utility owned electrical transformer to the concrete pad, as required.



ADD 2x BLOCKING @ ROOF LEVEL WITH METAL STRAPS (TOP AND BOTTOM) ACROSS PERP. FRAMING, TYP.

COORDINATE LOCATION OF THIS LINE OF BLOCKING W/ PUMP ACCESS ROOF OPENING

APPROX. EXTENTS OF ROOF LEVEL SUB-DIAPHRAGM, TYP.

ADD 2x BLOCKING @ CEILING LEVEL WITH METAL STRAPS (TOP AND BOTTOM) ACROSS PERP. FRAMING, TYP.

APPROX. EXTENTS OF ADDED CEILING LEVEL PLYWOOD SUB-DIAPHRAGM, TYP.

ADD SIMPSON A35 BETWEEN TRUSS/FRAMING & CMU WALL TOP PLATE, TYP. ALL AROUND PERIMETER

COORDINATE CONNECTION BETWEEN BLOCKING/SHEATHING & CMU WALL TOP PLATE W/ RETROFIT CONCEPT TO ENGAGE SHEAR RESISTANCE OF INTERIOR WALL

ADD FLATWISE 2x BLOCKING @ CEILING LEVEL TO PROVIDE CONNECTION BETWEEN (N) PLYWOOD SHEATHING AND (E) CMU WALL TOP PLATE, TYP.

NOTES:
1) PROVIDE BOUNDARY NAILING BETWEEN PLYWOOD SHEATHING & BLOCKING/FRAMING.
2) PROVIDE ADDITIONAL ANCHORS BETWEEN TOP PLATE & CMU WALL (AS REQUIRED).

Figure 4.5 Sub-diaphragm Retrofit Concept
(Adapted From: Roof Plan on Sheet A1.3 of 1997 design drawings by Multi/Tech Consultants)

4.1.6 Deer Park Pump Station

Table 4.6 summarizes preliminary seismic retrofit concepts to mitigate the potential structural and nonstructural deficiencies described in Section 3.1.6 for the Deer Park Pump Station.

Table 4.6 Deer Park Pump Station Preliminary Mitigation Concepts

Potential Deficiencies	Description
Structural	• Conduct nondestructive scanning to verify the size and location of masonry wall reinforcing and evaluate the adequacy of the masonry walls. • Based on the number of potential deficiencies identified that are associated with the wood framed roof, suggest removing existing roof and replacing with new plywood sheathed wood truss roof with appropriate seismic detailing (including consideration of cross ties between diaphragm chords and out of plane bracing for perimeter and interior masonry walls).
Nonstructural	• Provide bracing for the piping. • Provide independent bracing for the valves. • Install angles all around the perimeter of the pump support concrete pedestal with anchors into the floor slab. On two opposing sides, add a pair of steel straps that are welded to the angle and anchored up the face of the pedestal. • Add flexible couplings between the pumps and the connected piping. • Add anchors through the pipe support stanchion base plates into the slab-on-grade at locations with missing anchors. • Perform an investigation to evaluate the adequacy of the anchorage of electrical cabinets to housekeeping pads and housekeeping pads to slab on grade, and supplement anchorage (as required). Also, provide anchorage/bracing between the top of the electrical cabinets and masonry wall. • Re-install the restrainer bracket for the emergency generator starter batteries, as appropriate. • Verify the adequacy of the antenna connection to the supporting pole. • Coordinate with electrical utility to conduct further evaluation to validate the seismic performance of pole-mounted transformer and utility pole.

4.1.7 Edwards Pump Station

Table 4.7 summarizes preliminary seismic retrofit concepts to mitigate the potential structural and nonstructural deficiencies described in Section 3.1.7 for the Edwards Pump Station.

Table 4.7 Edwards Pump Station Preliminary Mitigation Concepts

Potential Deficiencies	Description
Structural	• Perform a site-specific geotechnical study to further evaluate the potential earthquake-induced liquefaction and lateral spreading hazard at the site. If required, mitigate the potential permanent ground deformation hazard at the site with geotechnical improvement, or other appropriate techniques. • Based on the age of the structure and the number of potential deficiencies identified, it is recommended that the City consider replacing the Edwards Pump Station structure.
Nonstructural	• Mitigation of potential nonstructural deficiencies is dependent on the selected approach to mitigate structural deficiencies. If the pump stations is replaced, it is anticipated that new components would be installed satisfying current seismic design and detailing requirements. • Provide appropriate flexible joints where water system piping penetrates through the pump station floor to accommodate potential differential movement between the structure and the surrounding soil at or near the pipe penetration. • Provide bracing for the piping. • Provide independent bracing for the valves. • Perform additional analysis to evaluate the adequacy of the existing connection of the motor to the top of the steel motor support, the motor support, the connection of the motor support to the concrete pedestal, and the concrete pedestal. • Install Z-shaped brackets (fabricated from welded channel sections) anchored to the concrete slab on grade and bearing against top surface of the pump support steel base plate. Provide two brackets on each side of the concrete pedestal near the existing steel base plate anchors. • Add flexible couplings between the pumps and the connected piping.

Table 4.7 Edwards Pump Station Preliminary Mitigation Concepts (cont.)

Potential Deficiencies	Description
Nonstructural (cont.)	• Perform an investigation to evaluate the adequacy of the anchorage of electrical cabinets to housekeeping pads and housekeeping pads to slab on grade, and supplement anchorage (as required). Also, provide anchorage/bracing between the top of the electrical cabinets and masonry wall. • Provide flexible couplings where conduits connect to the top of floor- and wall-mounted electrical panels and cabinets. • Provide restraint for the emergency generator starter batteries. • Provide lens covers for light fixtures with safety devices that prevent the lens cover and light bulbs from falling. • Verify the adequacy of the antenna connection to the supporting pole. • Provide anchorage of the HVAC unit to the concrete pad. • Provide grating clip connections between the grating and steel support framing. • Provide restraint for the overhead bridge crane, when not in use. • Provide restraint for rolling lifts, when not in use. • Provide restraint for ladders (using straps to the wall, etc.), when not in use. • Coordinate with electrical utility to conduct further evaluation to validate the seismic performance of pole-mounted transformer and utility pole.

4.1.8 Limelight Pump Station

Table 4.8 summarizes preliminary seismic retrofit concepts to mitigate the potential structural and nonstructural deficiencies described in Section 3.1.8 for the Limelight Pump Station.

Table 4.8 Limelight Pump Station Preliminary Mitigation Concepts

Potential Deficiencies	Description
Structural	• Perform an evaluation of the potential impact of the vertical cracks in the masonry shear walls on the seismic performance of the pump station and implement an appropriate repair concept. Implement repairs of localized deterioration of plywood sheathing and framing to restore these components to their original strength. • Remove a small section of existing roofing to verify the adequacy of the existing roof sheathing to truss nailing. • Add blocking to support the edges of the roof sheathing panels and provided boundary nailing between the roof sheathing and blocking. • Install new shaped blocking between the roof sheathing and masonry wall top plate. Provided boundary nailing between the roof sheathing and blocking, and Simpson A35 clip angles between the blocking and top plate and blocking and trusses. Also, suggest engaging the shear resistance of the interior masonry wall by adding a combination of plywood sheathing, framing/blocking, and metal connector hardware to provide a load path between the roof diaphragm and the interior masonry wall. • Perform an investigation to verify the adequacy of the connection between the roof sheathing, gable end triangular portion wood framed shear walls and masonry wall top plate below. • Install Simpson A35 clips between trusses and masonry wall top plate in conjunction with cross tie retrofit described in next bullet item. • Install a combination of plywood sheathing, blocking, steel straps, and metal connector hardware to provide adequate out-of-plane support for CMU walls. The concept is similar to that shown in Figure 4.5, for the Creekside Pump Station, except that all three sub-diaphragms should be installed with added plywood at the at the ceiling level, since the masonry portion of the gable end walls does not extend all the way to the roof level.
Nonstructural	• Provide bracing for the piping. • Provide independent bracing for the valves.

Table 4.8 Limelight Pump Station Preliminary Mitigation Concepts (cont.)

Potential Deficiencies	Description
Nonstructural (cont.)	• Provide independent support and bracing for the air relief valves. • Perform additional analysis to evaluate the adequacy of the existing connection of the motor to the top of the steel motor support, the motor support, the connection of the motor support to the concrete pedestal, and the concrete pedestal. • Add flexible couplings between the pumps and the connected piping. • Perform an investigation to evaluate the adequacy of the anchorage of electrical cabinets to housekeeping pads and housekeeping pads to slab on grade, and supplement anchorage (as required). Also, provide anchorage/bracing between the top of the electrical cabinets and masonry wall. • Verify the adequacy of the antenna connection to the supporting pole. • Coordinate with electrical utility to anchor the utility owned electrical transformer to the concrete pad, as required.

4.1.9 Mountain View Pump Station

Table 4.9 summarizes preliminary seismic retrofit concepts to mitigate the potential structural and nonstructural deficiencies described in Section 3.1.9 for the Mountain View Pump Station.

Table 4.9 Mountain View Pump Station Preliminary Mitigation Concepts

Potential Deficiencies	Description
Structural	• Reconfigure roof trusses and framing/blocking to provide plywood shear wall between roof diaphragm and top plate of north masonry wall. Also, suggest engaging the shear resistance of the interior north-south oriented masonry wall by adding a combination of plywood sheathing, framing/blocking, and metal connector hardware to provide a load path between the roof diaphragm and the interior masonry wall. • Install Simpson A35 clips between trusses and masonry wall top plate in conjunction with cross tie retrofit described in next bullet item. • Install a combination of plywood sheathing, blocking, steel straps, and metal connector hardware to provide adequate out-of-plane support for CMU walls (see Figure 4.6).
Nonstructural	• Provide bracing for the piping. • Provide independent bracing for the valves. • Provide independent support and bracing for the air relief valves. • Perform additional analysis to evaluate the adequacy of the existing connection of the motor to the top of the steel motor support, the motor support, the connection of the motor support to the concrete pedestal, and the concrete pedestal. • Add flexible couplings between the pumps and the connected piping. • Add anchors through the pipe support stanchion base plates into the slab-on-grade at locations with missing anchors. • Provide additional anchors for chlorination equipment and repair/replace damaged curb. • Provide adequate anchorage between the strut and the masonry shear wall for seismic demands and provide positive connection between spacers and main strut. • Provide bracing for transformer hung from roof.

Table 4.9 Mountain View Pump Station Preliminary Mitigation Concepts (cont.)

Potential Deficiencies	Description
Nonstructural (cont.)	• Perform an investigation to evaluate the adequacy of the anchorage of electrical cabinets to the slab on grade, and supplement anchorage (as required). Also, supplement the existing anchorage between the top of the electrical cabinets and masonry wall. • Provide flexible couplings where conduits connect to the top of floor- and wall-mounted electrical panels and cabinets. • Provide restraint for the emergency generator starter batteries within the battery bins (e.g., strap) and anchorage of the battery bins. • Provide bracing for the emergency generator muffler. • Provide lens covers for light fixtures with safety devices that prevent the lens cover and light bulbs from falling. • Verify the adequacy of the antenna connection to the supporting pole. • Provide restraint for overhead trolley chain hoist, when not in use. • Provide restraint for ladders (using straps to the wall, etc.), when not in use. • Coordinate with electrical utility to anchor the utility owned electrical transformer to the concrete pad, as required.

INVESTIGATE FRAMING CONFIGURATION IN THIS AREA & MODIFY SUB-DIAPHRAGM APPROACH, AS REQUIRED

ADD FLATWISE 2x BLOCKING @ CEILING LEVEL TO PROVIDE CONNECTION BETWEEN (N) PLYWOOD SHEATHING AND (E) CMU WALL TOP PLATE, TYP.

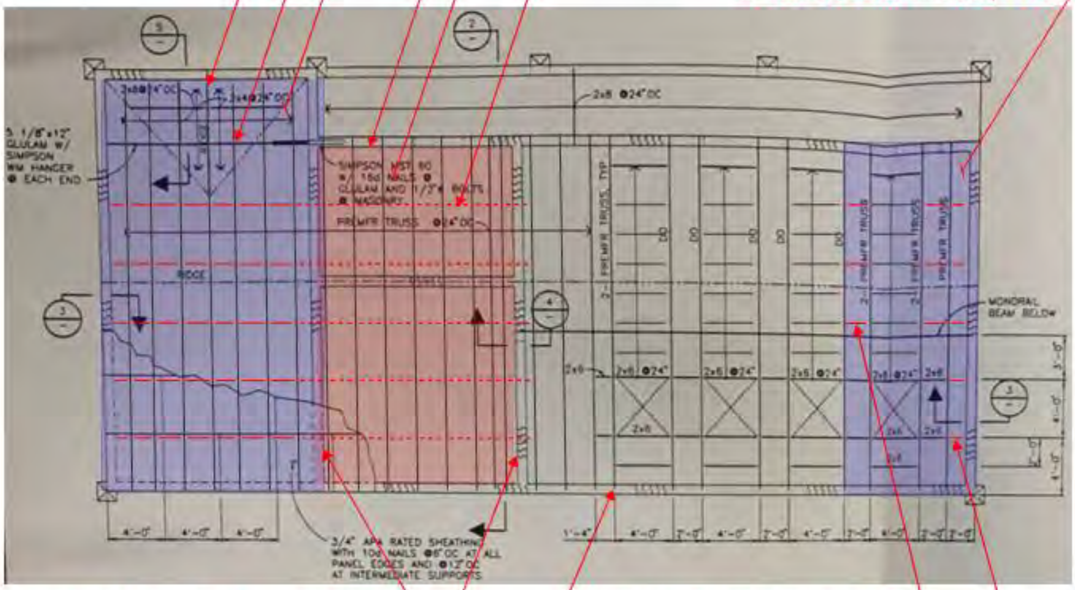
ADD HOLD DOWN ANCHORS (IN PAIRS) BETWEEN JOIST & TRUSS, TYP.

APPROX. EXTENTS OF ADDED CEILING LEVEL PLYWOOD SUB-DIAPHRAGM, TYP.

ADD HOLD DOWN ANCHORS (IN PAIRS) BETWEEN JOIST & CMU WALL, TYP.

ADD 2x BLOCKING @ CEILING LEVEL WITH METAL STRAPS (TOP AND BOTTOM) ACROSS PERP. FRAMING, TYP.

APPROX. EXTENTS OF ROOF LEVEL SUB-DIAPHRAGM, TYP.



COORDINATE CONNECTION BETWEEN BLOCKING/SHEATHING & CMU WALL TOP PLATE W/ RETROFIT CONCEPT TO ENGAGE SHEAR RESISTANCE OF INTERIOR WALL

ADD 2x BLOCKING @ ROOF LEVEL WITH METAL STRAPS (TOP AND BOTTOM) ACROSS PERP. FRAMING, TYP.

ADD SIMPSON A35 BETWEEN TRUSS & CMU WALL TOP PLATE, TYP.

ADD HOLD DOWN ANCHORS (IN PAIRS) BETWEEN BLOCKING & CMU WALL, TYP.

NOTES:
1) PROVIDE BOUNDARY NAILING BETWEEN PLYWOOD SHEATHING & BLOCKING/FRAMING.
2) PROVIDE ADDITIONAL ANCHORS BETWEEN TOP PLATE & CMU WALL (AS REQUIRED).

Figure 4.6 Sub-diaphragm Retrofit Concept
(Adapted From: Detail 1 on Sheet S4 of 1994 design drawings by KMC, Inc.)

4.1.10 Salem/Keizer Intertie #1 Pump Station

Table 4.10 summarizes preliminary seismic retrofit concepts to mitigate the potential structural and nonstructural deficiencies described in Section 3.1.10 for the Salem/Keizer Intertie #1 Pump Station.

Table 4.10 Salem/Keizer Intertie #1 Pump Station Preliminary Mitigation Concepts

Potential Deficiencies	Description
Structural	• Perform a site-specific geotechnical study to confirm the expected liquefaction-induced settlement. If required, mitigate the potential permanent ground deformation hazard at the site with geotechnical improvement, or other appropriate techniques. • Perform additional analysis to investigate the adequacy of the gap between the City of Salem pump station and the adjacent City of Keizer building. • Install shaped blocking at the ridge line to bridge over the existing gap in the roof sheathing. Provide boundary nailing between the roof sheathing and new blocking. Coordinate with architect for any necessary modifications to roof venting. • Install Simpson A35 clip angles between the blocking and top plate and blocking and trusses. • Install a combination of blocking and steel straps between truss bottom chord members in the two truss bays where blocking is not currently installed to provide continuous cross ties in the east-west direction.
Nonstructural	• Provide appropriate flexible joints where water system piping penetrates through the pump station floor to accommodate potential differential movement between the structure and the surrounding soil at or near the pipe penetration. • Provide bracing for the piping. • Provide independent bracing for the valves. • Add flexible couplings between the pumps and the connected piping. • Provide anchorage of the chlorination skid to the concrete slab on grade and anchorage of chlorination system components to the skid. • Perform an investigation to evaluate the adequacy of the anchorage of electrical cabinets to the concrete housekeeping pads, and supplement anchorage (as required).

Table 4.10 Salem/Keizer Intertie #1 Pump Station Preliminary Mitigation Concepts (cont.)

Potential Deficiencies	Description
Nonstructural (cont.)	• Remove the existing L-shaped strut brackets at the top of the electrical cabinets and replace with a more appropriate steel bracket. • Verify the adequacy of the antenna connection to the supporting pole. • Coordinate with electrical utility to anchor the utility owned electrical transformer to the concrete pad, as required.

4.1.11 Turner Control Facility

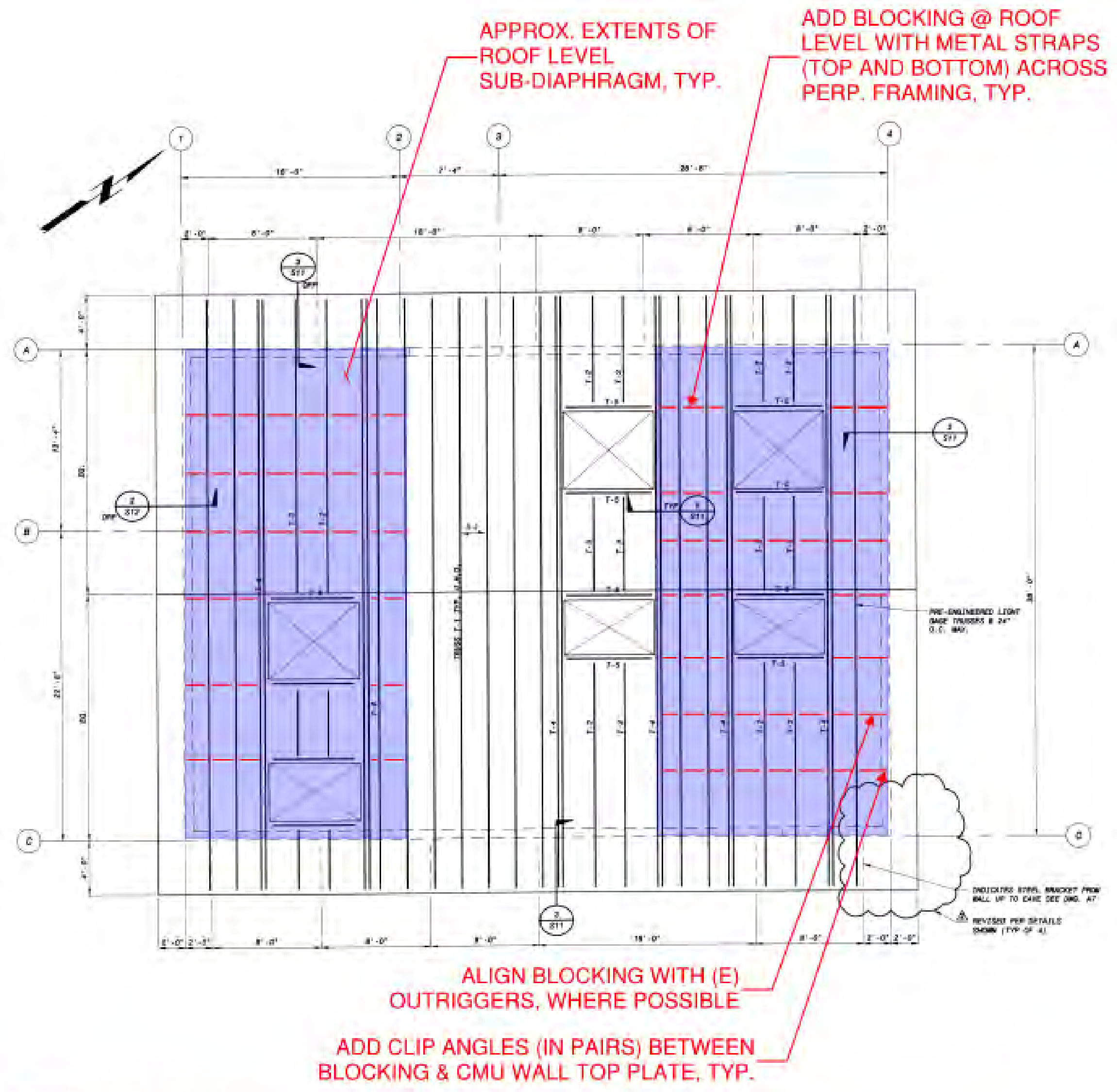
Table 4.11 summarizes preliminary seismic retrofit concepts to mitigate the potential structural and nonstructural deficiencies described in Section 3.1.11 for the Turner Control Facility.

Table 4.11 Turner Control Facility Preliminary Mitigation Concepts

Potential Deficiencies	Description
Structural	• Perform a site-specific geotechnical study to confirm the expected liquefaction-induced settlement. If required, mitigate the potential permanent ground deformation hazard at the site with geotechnical improvement, or other appropriate techniques. • Add blocking to support the edges of the roof sheathing panels and provided boundary fasteners between the roof sheathing and blocking. • Perform an investigation to determine if adequate fasteners are provided for the roof sheathing to blocking and blocking to masonry wall top plate connections. Provide supplemental fasteners, as required. • Install additional fasteners between the roof sheathing and outriggers in conjunction with cross tie retrofit described in next bullet item. • Install a combination of blocking, steel straps, and metal connector hardware to provide adequate out-of-plane support for CMU walls in the direction perpendicular to the roof trusses (see Figure 4.7).
Nonstructural	• Provide appropriate flexible joints where water system piping penetrates through the control facility wall to accommodate potential differential movement between the structure and the surrounding soil at or near the pipe penetration. • Provide bracing for the piping. • Provide independent bracing for the valves. • Provide independent bracing for the valve actuators. • Provide anchorage of the control cabinet to the housekeeping pad. • Provide supplemental anchorage of the electrical transformer to the concrete slab on grade. • Provide restraint for backup batteries inside the battery cabinet. • Provide restraint for pendant supported lights to prevent excessive swing. • Provide lens covers for light fixtures with safety devices that prevent the lens cover and light bulbs from falling.

Table 4.11 Turner Control Facility Preliminary Mitigation Concepts (cont.)

Potential Deficiencies	Description
Nonstructural (cont.)	• Verify the adequacy of the antenna connection to the supporting pole. • Provide seismic bracing for the ceiling hung inline HVAC fan. • Provide anchors between the HVAC condenser unit and concrete support pad. • Provide anchorage or bracing for the storage shelving to the floor and/or the wall. • Provide appropriate restraint for the fire extinguisher in its cabinet.



APPROX. EXTENTS OF ROOF LEVEL SUB-DIAPHRAGM, TYP.

ADD BLOCKING @ ROOF LEVEL WITH METAL STRAPS (TOP AND BOTTOM) ACROSS PERP. FRAMING, TYP.

ALIGN BLOCKING WITH (E) OUTRIGGERS, WHERE POSSIBLE

ADD CLIP ANGLES (IN PAIRS) BETWEEN BLOCKING & CMU WALL TOP PLATE, TYP.

NOTES:
1) PROVIDE BOUNDARY FASTENERS BETWEEN PLYWOOD SHEATHING & BLOCKING/FRAMING.
2) PROVIDE ADDITIONAL ANCHORS BETWEEN TOP PLATE & CMU WALL (AS REQUIRED).

Figure 4.7 Sub-diaphragm Retrofit Concept
(Adapted From: Roof Plan on Sheet S6 of 2007 design drawings by Black & Veatch Corporation)

4.2 Reservoirs and Reservoir Control Buildings

This section provides summary tables that describe preliminary seismic retrofit concepts to mitigate the potential structural and nonstructural seismic deficiencies identified for selected City reservoirs and reservoir control buildings, described in Section 3.2. Where appropriate, these tables also provide recommendations for further investigation and/or analysis to potentially mitigate deficiencies through more detailed structural calculations or to infill gaps in the data that was available for this study.

4.2.1 Candalaria Reservoir

Table 4.12 summarizes preliminary seismic retrofit concepts to mitigate the potential structural and nonstructural deficiencies described in Section 3.2.1 for the Candalaria Reservoir.

Table 4.12 Candalaria Reservoir Preliminary Mitigation Concepts

Retrofit Recommendations	Description
Structural	Reservoir • Perform a future ASCE 41 Tier 2 assessment to evaluate the adequacy of the provided column reinforcing lap splice length. Valve Vault • Install stainless steel plates and/or angles to connect riser, base, and lid precast components at the precast concrete construction joints. • Repair any leaking precast joints with polyurethane resin injection or other similar method after an earthquake, as required.
Nonstructural	Reservoir • Verify that piping, fittings, and valve bodies are constructed of steel or ductile iron. Replace any components that are suspected to be cast iron. • Perform additional analysis to evaluate the adequacy of the overflow pipe and valve operator riser shafts to resist seismic forces. Provide lateral bracing of the overflow pipe and valve operator riser shafts, as required.

Table 4.12 Candalaria Reservoir Preliminary Mitigation Concepts (cont.)

Retrofit Recommendations	Description
Nonstructural (cont.)	Valve Vault • Perform additional analysis to evaluate the adequacy of the piping and valve to resist seismic forces (e.g., span between vault walls). Alternatively, provide bracing for the pipe and valve inside the valve vault. • Provide restraint of backup batteries, as required.

4.2.2 Champion Hill Reservoir

Table 4.13 and Table 4.14 summarizes preliminary seismic retrofit concepts to mitigate the potential structural and nonstructural deficiencies described in Section 3.2.2 for the Champion Hill Reservoir and the Champion Hill Reservoir Control Building, respectively

Table 4.13 Champion Hill Reservoir Preliminary Mitigation Concepts

Retrofit Recommendations	Description
Structural	• Perform a site-specific geotechnical study to further evaluate the potential earthquake-induced liquefaction hazard at the site. If required, mitigate the potential permanent ground deformation hazard at the site with geotechnical improvement, or other appropriate techniques. • Perform a future ASCE 41 Tier 2 assessment to evaluate the adequacy of the provided column reinforcing lap splice length.
Nonstructural	• Install connection brackets at the corners of the pipe support pedestals that are anchored to both the concrete pedestal and concrete floor slab.

Table 4.14 Champion Hill Reservoir Control Building Preliminary Mitigation Concepts

Retrofit Recommendations	Description
Structural	• Perform a site-specific geotechnical study to further evaluate the potential earthquake-induced liquefaction hazard at the site. If required, mitigate the potential permanent ground deformation hazard at the site with geotechnical improvement, or other appropriate techniques. • Add blocking to support the edges of the roof sheathing panels and provided boundary nailing between the roof sheathing and blocking. • Install Simpson roof boundary clips (RBCs) between blocking and masonry wall top plate and Simpson A35 clip angles between the blocking and trusses. • Install Simpson A35 clip angles, at approximately 2-feet on center, between gable end truss bottom chord and masonry wall top plate.

Table 4.14 Champion Hill Reservoir Control Building Preliminary Mitigation Concepts (cont.)

Retrofit Recommendations	Description
Structural (cont.)	• Install appropriate metal connector hardware to provide a vertical connection between the blocking that the kicker brace frames into and the adjacent roof trusses. Provide local strengthening of trusses, as appropriate.
Nonstructural	• Provide appropriate flexible joints where water system piping penetrates through the control building wall to accommodate potential differential movement between the structure and the surrounding soil at or near the pipe penetration. • Provide bracing for the piping. • Provide independent bracing for the valves. • Provide independent bracing for the recirculation pump and associated piping. • Provide an additional anchor at the base of the pressure tank. • Provide restraint for backup batteries inside the battery cabinet. • Verify the adequacy of the antenna connection to the supporting pole. • Install a combination of blocking and metal connector hardware to provide adequate connections to transfer seismic forces from the ceiling to the masonry walls and eliminate the potential for cross-grain bending of wood ledgers. • Provide restraint for temporarily stored electrical cabinets and other components (using straps to the wall, etc.). • Coordinate with electrical utility to conduct further evaluation to validate the seismic performance of pole-mounted transformer and utility pole.

4.2.3 Eola #1B Reservoir

Table 4.15 summarizes preliminary seismic retrofit concepts to mitigate the potential structural and nonstructural deficiencies described in Section 3.2.3 for the Eola #1B Reservoir.

Table 4.15 Eola #1B Reservoir Preliminary Mitigation Concepts

Retrofit Recommendations	Description
Structural	<u>Reservoir</u> • Perform an evaluation of the potential impact of the circumferential concrete cracks adjacent to the roof to wall interface on the seismic performance of the reservoir. • Perform a future ASCE 41 Tier 2 assessment to evaluate the adequacy of the provided column reinforcing lap splice length. <u>Valve Vaults</u> • Investigate concrete deterioration near top of South Valve Vault wall to lid connection and develop appropriate repair concepts. • Install stainless steel plates and/or angles to connect riser, base, and lid precast components at the precast concrete construction joints. • Repair any leaking precast joints with polyurethane resin injection or other similar method after an earthquake, as required.
Nonstructural	<u>Reservoir</u> • Supplement existing bracing for vertical section of inlet pipe. <u>Valve Vaults</u> • Perform additional analysis to evaluate the adequacy of the piping and valves to resist seismic forces (e.g., span between vault walls). Alternatively, provide bracing for the piping and valves inside the valve vaults.

4.2.4 Fairmount Reservoir

Table 4.16 and Table 4.17 summarizes preliminary seismic retrofit concepts to mitigate the potential structural and nonstructural deficiencies described in Section 3.2.4 for the Fairmount Reservoir and the Fairmount Reservoir Control Building/Pump Station, respectively.

Table 4.16 Fairmount Reservoir Preliminary Mitigation Concepts

Retrofit Recommendations	Description
Structural (based on 2018 seismic study by Carollo Engineers)	• Add a 6-inch layer of shotcrete at the inside face of the perimeter walls and footings. • Provide stainless steel connections along the roof expansion joints to transfer shear forces between roof panels. Also, provide anchors between the roof slab and the walls to transfer roof seismic loads to the perimeter walls.
Additional Structural (based on SEFT Desktop assessment)	• It is recommended that if the future seismic retrofit of Fairmount Reservoir is designed for a reduced seismic hazard level (i.e., BSE-1E hazard level), the 2019 OSSC Chapter 34 exception that the seismic hazard level should not be taken as less than 75% of the BSE-1N seismic hazard level should be considered. • Perform a future structural assessment to evaluate the potential impact of the interaction between the Fairmount Reservoir and the integrally constructed Fairmount Reservoir Control Building/Pump Station. • Perform a future ASCE 41 Tier 2 assessment to evaluate the adequacy of the provided column reinforcing lap splice length.
Nonstructural	• Verify that piping, fittings, and valve bodies are constructed of steel or ductile iron. Replace any components that are suspected to be cast iron.

Table 4.17 Fairmount Reservoir Control Building/Pump Station Preliminary Mitigation Concepts

Retrofit Recommendations	Description
Structural	• In coordination with the future detailed design for the seismic retrofit of the Fairmount Reservoir, perform a detailed structural seismic assessment of the Fairmount Reservoir Control Building/Pump Station and develop seismic mitigation concept recommendations for consideration by the City.
Nonstructural	• Provide bracing for the piping. • Provide independent bracing for the valves and valve actuators. • Provide bracing/restraint for vertical pump bells and valve operator riser shafts. • Replace any piping and valve components that are suspected to be cast iron. • Replace any corrosion damaged piping and valve components and connection hardware not already replaced by the bullet item above. • Provide anchorage between pump bases and concrete slab. • Perform additional analysis to evaluate the adequacy of the existing connection of the motor to the top of the steel motor support and the motor support. • Add flexible couplings between the pumps and the connected piping. • Provide bracing for the air vent vertical pipe. • Perform an investigation to evaluate the adequacy of the anchorage of electrical cabinets to concrete floor slab, and supplement anchorage (as required). Also, provide anchorage/bracing between the top of the electrical cabinets and concrete wall. • Provide flexible couplings where conduits connect to the top of floor-mounted electrical cabinets. • Verify the adequacy of the antenna connection to the supporting pole. • Coordinate with electrical utility to conduct further evaluation to validate the seismic performance of pole-mounted transformer and utility pole.

4.2.5 Grice Hill Reservoir

Table 4.18 and Table 4.19 summarizes preliminary seismic retrofit concepts to mitigate the potential structural and nonstructural deficiencies described in Section 3.2.5 for Grice the Hill Reservoir and the Grice Hill Reservoir Control Building, respectively.

Table 4.18 Grice Hill Reservoir Preliminary Mitigation Concepts

Retrofit Recommendations	Description
Structural	• Perform a future ASCE 41 Tier 2 assessment to evaluate the adequacy of the provided column reinforcing lap splice length.
Nonstructural	• Install connection brackets at the corners of the pipe support pedestals that are anchored to both the concrete pedestal and concrete floor slab.

Table 4.19 Grice Hill Reservoir Control Building Preliminary Mitigation Concepts

Retrofit Recommendations	Description
Structural	• Add blocking to support the edges of the roof sheathing panels and provided boundary nailing between the roof sheathing and blocking. • Install Simpson roof boundary clips (RBCs) between blocking and masonry wall top plate and Simpson A35 clip angles between the blocking and trusses. • Install appropriate metal connector hardware to provide a vertical connection between the blocking that the kicker brace frames into and the adjacent roof trusses. Provide local strengthening of trusses, as appropriate.
Nonstructural	• Provide bracing for the piping. • Provide independent bracing for the valves. • Repair seismic valve so that is operational in the event of an earthquake. • Provide anchorage of the pressure tank. • Provide additional restraint for backup batteries inside the battery cabinet. • Install a combination of blocking and metal connector hardware to provide adequate connections to transfer seismic forces from the ceiling to the masonry walls and eliminate the potential for cross-grain bending of wood ledgers.

Table 4.19 Grice Hill Reservoir Control Building Preliminary Mitigation Concepts (cont.)

Retrofit Recommendations	Description
Nonstructural (cont.)	• Provide restraint for temporarily stored electrical cabinets and other components (using straps to the wall, etc.). • Provide restraint for ladder (using straps to the wall, etc.), when not in use. • Coordinate with electrical utility to anchor the utility owned electrical transformer to the concrete pad, as required.

4.2.6 Lone Oak Reservoir

Table 4.20 and Table 4.21 summarizes preliminary seismic retrofit concepts to mitigate the potential structural and nonstructural deficiencies described in Section 3.2.6 for the Lone Oak Reservoir and the Lone Oak Reservoir Control Building, respectively.

Table 4.20 Lone Oak Reservoir Preliminary Mitigation Concepts

Retrofit Recommendations	Description
Structural	• No potential deficiencies were identified that require mitigation.
Nonstructural	• No potential deficiencies were identified that require mitigation.

Table 4.21 Lone Oak Reservoir Control Building Preliminary Mitigation Concepts

Retrofit Recommendations	Description
Structural	• Coordinate with the City to attempt to locate deferred submittal design drawings and calculations from original construction. Supplement available drawings with a detailed field investigation (including localized removal of architectural finishes) to observe and document details of original construction. Once additional details of original construction are available, complete a follow-up ASCE 41 Tier 1 evaluation and develop preliminary concepts to mitigate the identified deficiencies.
Nonstructural	• Provide bracing for the piping. • Provide independent bracing for the valves. • Provide anchorage of the pressure tank. • Perform an investigation to evaluate the adequacy of the anchorage of control cabinet to housekeeping pad, and supplement anchorage (as required). Also, provide anchorage/bracing between the top of the control cabinet and masonry wall. • Perform an investigation to evaluate the adequacy of the anchorage of the battery cabinet to the control cabinet, and supplement anchorage (as required). Also, provide restraint for backup batteries inside the battery cabinet.

Table 4.21 Lone Oak Reservoir Control Building Preliminary Mitigation Concepts (cont.)

Retrofit Recommendations	Description
	• Verify the adequacy of the antenna connection to the supporting pole. • Perform an investigation to evaluate the adequacy of the bracing of the suspended HVAC unit, and supplement bracing (as required). • Perform an investigation to evaluate the adequacy of the anchorage of the water heater/safety shower base skid to the concrete slab, and supplement anchorage (as required). • Perform an investigation of the original deferred submittal design details for the chlorine room masonry wall reinforcing and top of wall bracing. Provide supplemental bracing of masonry walls, as required. • Provide restraint for ladder (using straps to the wall, etc.), when not in use. • Coordinate with electrical utility to anchor the utility owned electrical transformer to the concrete pad, as required.

4.2.7 Mill Creek #1 Reservoir

Table 4.22 and Table 4.23 summarizes preliminary seismic retrofit concepts to mitigate the potential structural and nonstructural deficiencies described in Section 3.2.7 for the Mill Creek #1 Reservoir and the Mill Creek #1Reservoir Control Building, respectively.

Table 4.22 Mill Creek #1 Reservoir Preliminary Mitigation Concepts

Retrofit Recommendations	Description
Structural	• Perform a future ASCE 41 Tier 2 assessment to evaluate the adequacy of the provided column reinforcing lap splice length.
Nonstructural	• Install connection brackets at the corners of the pipe support pedestals that are anchored to both the concrete pedestal and concrete floor slab. • Provide additional seismic separation between the steel framed stair landing platform and reservoir concrete roof and/or provide diagonal bracing between stair landing support posts.

Table 4.23 Mill Creek #1 Reservoir Control Building Preliminary Mitigation Concepts

Retrofit Recommendations	Description
Structural	• Add blocking to support the edges of the roof sheathing panels and provided boundary nailing between the roof sheathing and blocking. • Coordinate with the City to attempt to locate deferred submittal design drawings and calculations from original construction. Supplement available drawings with a detailed field investigation (including potential localized removal of architectural finishes) to observe and document details of the truss blocking and associated connections. Once additional details of original construction are available, evaluate the adequacy of the load path to transfer seismic forces from the roof diaphragm to the masonry wall top plate and develop mitigation concepts, as appropriate. • Install a combination of blocking and steel straps between truss bottom chord members in four additional truss bays per line of blocking.
Nonstructural	• Provide bracing for the piping. • Provide independent bracing for the valves. • Provide anchorage of the pressure tank.

Table 4.23 Mill Creek #1 Reservoir Control Building Preliminary Mitigation Concepts (cont.)

Retrofit Recommendations	Description
Nonstructural (cont.)	• Perform additional analysis to evaluate the adequacy of the existing connection of the motor to the top of the steel motor support, the motor support, and the connection of the motor support to the concrete slab. • Add flexible couplings between the pumps and the connected piping. • Provide restraint for backup batteries inside the battery cabinet. • Verify the adequacy of the antenna connection to the supporting pole. • Install a combination of blocking and metal connector hardware to provide adequate connections to transfer seismic forces from the ceiling to the masonry walls and eliminate the potential for cross-grain bending of wood ledgers. • Perform additional analysis to evaluate the adequacy of the electrical room unbraced partial height masonry walls. • Provide restraint for ladders (using straps to the wall, etc.), when not in use. • Coordinate with electrical utility to anchor the utility owned electrical transformer to the concrete pad, as required.

4.2.8 Mountain View Reservoir

Table 4.24 summarizes preliminary seismic retrofit concepts to mitigate the potential structural and nonstructural deficiencies described in Section 3.2.8 for the Mountain View Reservoir.

Table 4.24 Mountain View Reservoir Preliminary Mitigation Concepts

Retrofit Recommendations	Description
Structural	• Install seismic restraint between the reservoir walls and foundation. Potential concepts include using brackets and high-strength rods installed from inside the reservoir or installing new seismic cables in a thickened wall section from the exterior of the reservoir. Both options would likely require modifying/enlarging the existing foundation ring. • Operate the reservoir at a lower maximum elevation to reduce hydrodynamic forces to a level that makes the seismic performance of the prestressing strands adequate without further retrofit. (Note that this option may not be practical due to how the water level in the Mountain View Reservoir is hydraulically connected to the level in Franzen and Fairmount Reservoirs. OR Re-wrap the core wall with additional circumferential prestressing strands encased with shotcrete to provide additional capacity to resist the combination of hydrostatic and expected hydrodynamic hoop forces during an earthquake. • Provide fiber reinforced polymer (FRP) wrapping of columns.
Nonstructural	• Verify that piping, fittings, and valve bodies are constructed of steel or ductile iron. Replace any components that are suspected to be cast iron.

5.0 Next Steps

This technical memorandum summarizes the results of SEFT's preliminary seismic structural and nonstructural evaluation of selected City of Salem water system facilities (10 pump stations, Turner Control Facility, 8 reservoirs, and 5 reservoir control buildings). Based on the potential structural and nonstructural deficiencies identified, only one reservoir is expected to achieve Immediate Occupancy structural performance and Operational nonstructural performance. None of the other structures evaluated are expected to achieve either Immediate Occupancy structural performance or Operational nonstructural performance for a M9.0 CSZ scenario earthquake.

Due to project budget limitations, not all City of Salem water system structures were included in the scope of the preliminary seismic structural and nonstructural evaluations conducted as part of this project. It is recommended that the City conducts seismic evaluations of the remaining inventory of water system structures (e.g., pump stations, reservoirs, communications towers, etc.) as part of a future project.

The seismic evaluation findings presented in this report should be integrated with the findings of previous seismic studies of other water system components and future seismic assessments of the remaining water system components, to develop a holistic view of the expected seismic performance of the water system. This knowledge can be leveraged in developing a comprehensive long-term plan for implementing water system seismic resilience improvements. In the near-term, the City is strongly encouraged to implement a seismic retrofit program to address Life Safety seismic deficiencies for water system structures that are frequently accessed by City staff and contractors.

During this project it was observed that the City has installed seismic isolation valves on many reservoirs. These seismically activated valves are designed to close when they detect earthquake shaking and are intended to help prevent all the water stored in these reservoirs from leaking out of transmission and distribution system pipelines that may be damaged by the earthquake. The significant volume of water that will be preserved in the reservoirs that have seismic isolation valves will help to meet community water needs (e.g., firefighting, drinking, sanitation, etc.) after a major earthquake. However, once the seismic isolation valves shut, accessing the water stored in the reservoirs may be challenging. There does not appear to be hydrants (or other connection points) installed between the reservoirs and seismic isolation valves. In the near-term, the City should consider installing hydrants (or other connection points) between the reservoirs and seismic isolation valves, so that the stored water can be easily accessed by City staff and the City of Salem Fire Department. These hydrants and associated piping should be designed to accommodate the expected level of permanent ground deformation that may occur at the reservoirs. Also, in the near-term, the City should consider installing seismic isolation valves and associated hydrant connections for reservoirs that do not currently have seismic isolation valves.

If replacement of existing or construction of new water system structures is considered in the future to meet water demand or operational goals, then this would provide an opportunity to build more seismically resilient structures and associated support infrastructure that are capable of achieving the City's post-earthquake LOS goals. The selection of the location of any new water system structures and the foundation design for those structures should include appropriate consideration of potential earthquake-induced permanent ground deformation and related mitigation strategies to achieve the City's resilience goals.

In order to continue to advance the City's water system resilience planning process, we recommend that a follow-up study be conducted to identify and understand dependency relationships and develop appropriate strategies to manage them to minimize any associated cascading effects. Planning for and addressing issues such as where the City will get fuel for trucks and generators, how suppliers and contractors will be rapidly engaged and compensated, etc. will help improve resilience and speed the return to normalcy after a major disaster. The City of Salem should also continue to evaluate and implement alternative options to provide water to customers in the event that the water system is significantly damaged by a major earthquake and could take months to repair for more recently constructed structures to years to rebuild older structures.

6.0 Limitations

The opinions and recommendations presented in this report were developed with the care commonly used as the state of practice of the profession. No other warranties are included, either expressed or implied, as to the professional advice included in this report. This report has been prepared for the City of Salem to be used solely in its evaluation of the seismic safety of the water system components referenced. This report has not been prepared for use by other parties and may not contain sufficient information for purposes of other parties or uses.

7.0 References

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Appendix D. Facility Vulnerability Assessment Summary

The following sections provide a brief description of each of the facilities and summaries of seismic assessments for each of these facilities. The seismic assessments focus on structural and geotechnical issues. Nonstructural deficiencies are not discussed in this section. Detailed descriptions of the structural and nonstructural assessment results are presented in the SEFT report in Appendix C.

ASR # 1 and #2 Pump Station

ASR #2 Pump Station was built in 1998 as an addition to the ASR #1 Pump Station which was constructed in 1995. The combined structure of the two pump stations is an above-grade, single-story, reinforced masonry shear wall structure with a plywood-sheathed wood framed roof. The single-story building has an approximate footprint of 12 feet by 54 feet.

Structural deficiencies comprise the following items:

- Design or construction drawings for the structure were not available, so the masonry connections of ASR #2 to the original structure #1 could not be verified.
- Structurally, the roof poses seismic concerns as roof anchorage and wall bracing could not be identified.
- The load path is incomplete to transfer in-plane shear forces from the roof diaphragm to the masonry walls.

ASR #4 Pump Station

The ASR #4 Pump Station structure is an above-grade, single-story, reinforced masonry shear wall structure that was built in 1998. It has a plywood sheathed wood framed roof. The single-story building has an approximate footprint of 12 feet by 30 feet.

Structural concerns comprise the following items:

- A positive connection does not appear to be provided between the masonry walls and the roof.
- An access hatch in the roof creates an incomplete load path, reducing the capacity to transfer seismic forces to the shear wall below.
- Additionally, wall bracing is inadequate for this structure.

ASR #5 Pump Station

The ASR #5 Pump Station structure was built in 1998 and consists of an above-grade, single-story, reinforced masonry shear wall structure with a plywood ceiling diaphragm. It has a footprint of approximately 40 feet by 12 feet. The pump station's masonry shear wall structure is integrated with a premanufactured, hexagonal steel framed visitor-pavilion. The City Parks Department uses the room at the south end of the pump station structure for storage. This room is out of the scope of the seismic assessment.

Structural deficiencies comprise the following items:

- There does not appear to be either a) an adequate load path to transfer the seismic forces generated by the steel framed pavilion to the masonry shear wall structure or b) an adequate seismic separation to prevent unintended interaction between the steel framed pavilion and masonry shear wall structure.
- The north-south horizontal span for the ceiling diaphragm exceeds the ASCE 41-17 Tier 1 limit.
- No ceiling plywood sheathing nailing schedule was available in the drawings; therefore, it was unable to verify the nailing system adequacy,
- It is unclear if blocking is provided between the ceiling sheathing and masonry wall top plate. Therefore, there may be an incomplete load path to transfer in-plane shear forces from the ceiling diaphragm to the masonry walls.
- There is inadequate out-of-plane bracing for the perimeter and interior masonry walls.
- There are inadequate crossties between diaphragm chords.
- Vertical trim reinforcing is missing at the sides of door and other openings.
- The free-standing masonry wall to the north of the pump station is unbraced.
- Corrosion damage was observed at the base of the northern-most steel tube section columns of the pavilion.

Boone Road Pump Station

The original Boone Road Pump Station structure is an above-grade, single-story, reinforced masonry shear wall structure with a straight-sheathed wood framed roof that was built in 1977. The building has an approximate footprint of 34 feet by 36 feet. The structure received a partial seismic retrofit as part of a recent expansion project at the Boone Road Pump Station site.

The new electrical building that services the pump station is excluded from the scope of this study.

Structural deficiencies comprise the following items:

- The roof is slightly offset from the masonry shear walls on the north and south ends of the structure. The current load path may not be adequate for withstand seismic forces.
- The roof diaphragm span and aspect ratio exceed the ASCE 41-17 Tier 1 limits.
- The crossties between diaphragm chords are inadequate to resist seismic forces.

Creekside Pump Station

The Creekside Pump Station is an above-grade, single-story, reinforced masonry shear wall structure with a plywood sheathed wood framed roof. The facility as built in 1998 and has an approximate footprint of 20 feet by 47 feet.

Structural deficiencies comprise the following items:

- The original design drawings did not provide the roof plywood sheathing nailing schedule; therefore, the adequacy of the nailing system could not be verified.
- The roof diaphragm span for east-west oriented seismic forces exceeds the ASCE 41-17 Tier 1 limit for unblocked wood structural panel diaphragms.
- The load path is likely inadequate to transfer in-plane shear forces from the roof diaphragm to the masonry walls.
- Out-of-plane bracing of perimeter and interior masonry walls is inadequate.
- Adequate cross ties between diaphragm chords are not provided in both directions.

Deer Park Pump Station

The Deer Park Pump Station is an above-grade, single-story, reinforced masonry shear wall structure with a plywood sheathed wood framed roof. It was originally constructed in 1982, with an electrical room addition (located to the south of the pump station) added between 2008 and 2010. Roll-up doors and associated modifications were added in 2013. The building has an approximate footprint of approximate 44 feet by 20 feet.

Structural deficiencies comprise the following items:

- Design drawings were unavailable for the original construction of the structure the additions. Sizing, spacing, and detailing of the structure is unknown, and could result in further structural deficiencies.
- The roof diaphragm span for east-west oriented seismic forces exceeds the ASCE 41-17 Tier 1 limit for unblocked wood structural panel diaphragms.
- A positive connection between the roof and the masonry walls was not observed, resulting in inadequate load path to transfer in-plane shear forces from the roof to the walls.
- Out-of-plane bracing of perimeter and interior masonry walls is inadequate.
- There are inadequate cross ties between diaphragm chords in both directions.

Edwards Pump Station

The Edwards Pump Station structure was built in 1961, and structural and piping modifications were completed in 1966. This structure is an above-grade, single-story structure with a straight-sheathed wood framed roof. Structural clay research (SCR) brick shear walls are located at the perimeter of the building. Roof straight-sheathing is supported by a combination of steel frames, wood framing, and masonry walls. The L-shaped building has an overall footprint of 51 feet by 39 feet.

Structural deficiencies comprise the following items:

- There is evidence of soil settlement resulting from past uncontrolled water releases at the pump stations and uncertainty associated with the liquefaction potential of the soil in the area around the pump station.
- Design or construction drawings were not available for the original construction of the structure or for the additions. The structural detailing for the facility could not be ascertained; therefore, additional structural deficiencies may be revealed in the Tier 2 investigation. It is assumed that the brick walls are unreinforced. Therefore, the load path may be incomplete or inadequate to transfer seismic forces from the roof diaphragm to the masonry walls and/or steel frames.
- Cracking was observed in the masonry walls at the southwest corner of the building.
- Many components of the structure do not meet ASCE 41-17 Tier 1 limits. This includes the shear stress in the masonry walls, the height-to-thickness ratio for the masonry walls, the flexural stress in the steel moment frame beams, the steel moment frames, and the roof diaphragm.

Limelight Pump Station

The Limelight Pump Station is an above-grade, single-story, reinforced masonry shear wall structure with a plywood-sheathed wood framed roof. The structure was built in 1998 and has an approximate footprint of 20 feet by 41 feet.

Structural deficiencies comprise the following items:

- Vertical cracks were observed in all four exterior masonry walls.
- The roof diaphragm span for east-west oriented seismic forces exceeds the ASCE 41-17 Tier 1 limit.
- A positive connection between the roof and the masonry walls does appear to be provided. This may result in an inadequate load path to transfer in-plane shear forces.
- There is inadequate out-of-plane bracing of perimeter and interior masonry walls.
- Adequate crossties between diaphragm chords are not provided in both directions.

Mountain View Pump Station

The Mountain View Pump Station was built in 1995 and comprises an above-grade, single-story, reinforced masonry shear wall structure with a plywood-sheathed wood framed roof. The facility has an approximate footprint of 29 feet by 62 feet. A significant length of the north wall of the building is inset by approximately 4 feet. Roof framing at the north edge of the building is supported by a CMU beam that is then supported by three CMU square columns.

Structural deficiencies comprise the following items:

- The load path of the roof of this structure is incomplete to deliver seismic forces from the roof to the masonry walls.
- There is inadequate out-of-plane bracing of perimeter and interior walls.
- Adequate cross ties between diaphragm chords are not provided in both directions.

Salem/Keizer Intertie #1 Pump Station

The Salem/Keizer Intertie #1 Pump Station was built in 2013 and comprises an above-grade, single-story, reinforced masonry shear wall structure with a plywood-sheathed wood framed roof. The structure has an approximate footprint of 26 feet by 22 feet.

Structural and geotechnical deficiencies comprise the following items:

- There is a potential liquefaction hazard at the site. Liquefaction-induced permanent ground deformation may result in damage to the building structure.
- The pump station and the adjacent City of Keizer building are only separated by a 1/2-inch seismic joint. The two buildings are susceptible to earthquake-induced pounding because of this small separation.
- Roof sheathing is not continuous to the roof ridge line.
- The load path is incomplete to transfer in-plane shear forces from the roof diaphragm to the masonry walls, because a positive connection does not appear to be provided between the truss blocking and masonry wall top plate.
- Adequate crossties between diaphragm chords are not provided in both directions.

Turner Control Facility

The Turner Control Facility is mostly a new structure, as the original Turner Control Facility was substantially replaced in 2007. Only a small subgrade portion of the original structure integrated into the new structure. The facility is a single-story, above-grade reinforced masonry shear wall structure with a plywood-sheathed light-gauge metal framed roof. The building is constructed over two sections of concrete basement where the three water transmission lines and associated valves are located. The building has an approximate footprint of 36 feet by 52 feet.

Structural deficiencies comprise the following items:

- There is a potential liquefaction hazard at the site. Liquefaction-induced permanent ground deformation may result in damage to the building structure.
- The roof diaphragm spans in both directions do not meet ASCE 41-17 Tier 1 limits.
- At the gable end locations, the load path may be inadequate to transfer in-plane shear forces from the roof diaphragm to the masonry walls.
- Also at the gable end walls, the outrigger to roof diaphragm connection may not have adequate capacity to resist out-of-plane seismic forces from the masonry walls.
- There are no crossties between diaphragm chords in the direction perpendicular to the roof trusses.

Candalaria Reservoir

The 0.5 MG Candalaria Reservoir is a completely buried rectangular reinforced concrete reservoir, located at Candalaria Park, to the north of Candalaria Blvd S. This reservoir is approximately 123 feet in by 50 feet, with a maximum height of retained water of 15 feet. The reservoir was originally constructed in 1940 and was seismically retrofit in 2006. The scope of this retrofit included the addition of anchors to connect the roof of the reservoir to the walls. The 2006 retrofit also included the installation of a seismic shutoff valve in a new vault located on the north side of the reservoir. The assessment of the Candalaria Reservoir did not include the interior of the reservoir valve vault.

Structural deficiencies comprise the following items:

- The column vertical reinforcing lap splice length is less than the ASCE 41-17 Tier 1 specified minimum length for Immediate Occupancy.
- The valves may have structural deficiencies if they were constructed from precast concrete.
- Riser joints may separate and shift due to seismic forces, and sand, silt, and groundwater could infiltrate at these compromised locations.

Champion Hill Reservoir and Control Building

The Champion Hill Reservoir is a strand-wound, circular, prestressed concrete reservoir with a nearly flat roof. It has an approximate diameter of 140 feet and a maximum height of retained water of 20 feet.

The 2.2 MG tank and control building were built in 2005. The control building is a single-story structure that is approximately 37 feet by 46 feet in footprint. It has reinforced concrete walls below grade, reinforced masonry walls above grade, and a plywood sheathed wood truss roof.

Structural deficiencies comprise the following items:

- In the reservoir, the column vertical reinforcing lap splice length is less than the ASCE 41-17 Tier 1 specified minimum length for Immediate Occupancy structural performance.
- The control building does not show a positive connection between the gable end wall sheathing and the masonry wall top plate.
- There is an incomplete load path to transfer in-plane shear forces from the roof to the walls.
- The masonry walls indicate inadequate bracing.

Eola #1B Reservoir

The Eola #1B Reservoir was constructed in 1999 and has a capacity of 0.86 MG. The tank is 92 feet in diameter with a maximum water depth of 17 feet. The reservoir is partially buried: the west side is completely buried, whereas the east side of the reservoir is partially exposed. Two precast concrete valve vaults are located to the southeast of the reservoir.

Structural deficiencies comprise the following items:

- Around the circumference of the reservoir, concrete cracking was observed on the east side of the structure.
- In the reservoir, the column vertical reinforcing lap splice length is less than the ASCE 41-17 Tier 1 specified minimum length for Immediate Occupancy structural performance.
- The valve vaults show concrete deterioration.
- The vaults were constructed using precast concrete, which can result in water infiltration from shifted riser joints due to lateral earth pressures.

Fairmount Reservoir and Control Building

The Fairmount Reservoir and Control Building were constructed in 1936. The reservoir is a rectangular, reinforced concrete structure with two cells, each with a 5 MG capacity. The 10 MG reservoir is approximately 384 feet by 192 feet and has a maximum water depth of 21 feet. The reservoir is partially buried.

The Control Building/Pump Station is located on the south side of the reservoir and consists of a single-story above grade structure with a basement, constructed with reinforced concrete walls, and a reinforced concrete floor and roof. Two walls of the control building were constructed integrally with the Fairmount Reservoir. The building is approximately 21 feet by 21 feet.

Structural deficiencies comprise the following items (from 2018 seismic study by Carollo Engineers):

- Overstressed perimeter walls and footings, resulting from tension loads imposed by the bending moment loads caused by hydrodynamic forces.
- Lack of load path to transfer seismic forces from roof to walls.
- Shear forces cannot be transferred adequately between roof panels due to expansion joints
- Lack of positive connections between the roof and walls resulting in columns being overstressed.

In addition to these issues (which appear to be unmitigated), the following additional issues were noted about the reservoir in SEFT's Tier 1 assessment:

- "The 2018 Carollo study considered the BSE-1E seismic hazard level as defined by ASCE 41-13. Chapter 34 of the 2019 Oregon Structural Specialty Code (OSSC) indicates that the BSE-1E hazard level should not be taken as less than 75 percent of the BSE-1N seismic hazard level as defined by ASCE 41, much higher than what was considered in the 2018 Carollo study.
- Previous studies were preliminary in nature and did not include consideration of the potential interaction between the Fairmount Reservoir and adjacent Fairmount Reservoir Control Building/Pump Station.
- The column vertical reinforcing lap splice length is less than the ASCE 41-17 Tier 1 specified minimum length for Immediate Occupancy structural performance (i.e., 50 bar diameters)."

SEFT also noted the following structural issues related to the Control Building:

- The northeast and northwest walls of the control building/pump station were constructed integrally with the reservoir. Evaluation of the potential interaction between these two structures is beyond the scope of this preliminary ASCE 41 Tier 1 check-list based assessment but should be considered as part of a future detailed seismic evaluation and retrofit design.
- Several potential deficiencies are likely associated with detailing requirements for reinforcing steel (reinforcement ratio, maximum spacing limits, reinforcing around openings, reinforcing hooks at slab to wall connections, and foundation dowels).
- At the operating floor level, large stair openings are located adjacent to three of the four shear walls, limiting the connection length to transfer seismic forces from the floor slab to the concrete walls.

Grice Hill Reservoir and Control Building

The Grice Hill Reservoir is a strand-wound, circular, prestressed concrete reservoir with a nearly flat roof. The 2.2 MG reservoir has an approximate diameter of 140 feet and a maximum depth of 20 feet. The facility was constructed in 2001.

The Control Building is located to the west of the reservoir, comprising an above grade, single-story structure, with reinforced masonry shear walls and a plywood sheathed wood truss roof. The building has an approximate footprint of 45 feet by 37 feet.

The only structural issue identified with the reservoir is that the column vertical reinforcing lap splice length is less than the ASCE 41-17 Tier 1 specified minimum length for Immediate Occupancy structural performance.

Structural deficiencies comprise the following items:

- The ASCE 41-17 Tier 1 limit (unblocked wood structural panel diaphragms) was exceeded for the roof diaphragm spans in both directions.
- The sloped roof truss blocking and/or corners of top plate may be split as a result of the configuration of the toenail connection between the blocking and masonry wall top plate. This may have caused a reduced and inadequate load path to transfer in-plane shear forces from the roof diaphragm to the walls.
- The north and south gable end walls have inadequate out-of-plane bracing.

Lone Oak Reservoir and Control Building

The Lone Oak Reservoir is a strand-wound, circular, prestressed concrete reservoir with a nearly flat roof. The 5.6 MG reservoir has an approximate diameter of 196 feet and a maximum water depth of 26 feet.

The Lone Oak Reservoir Control Building is located to the west of the reservoir. The Control Building is a single-story building with reinforced concrete walls below grade, reinforced masonry walls above grade, and a plywood-sheathed wood truss roof. The building has an approximate footprint of 39 feet by 29 feet.

Structural deficiencies comprise the following items:

- No structural deficiencies were identified for the reservoir.
- Original design drawings for the control building were not available for review; therefore, an analysis of the structural deficiencies could not be completed.

Mill Creek #1 Reservoir and Control Building

The Mill Creek #1 Reservoir is a strand-wound, circular, prestressed concrete reservoir with a nearly flat roof. The 2.2 MG reservoir is approximately 140 feet in diameter, with a maximum water depth of 20 feet. The facility was built in 2013.

The Control Building is located to the southwest of the reservoir. The structure is a single-story building with reinforced concrete walls below grade, reinforced masonry walls above grade, and a plywood-sheathed wood truss roof. The building has an approximate footprint of 46 feet by 42 feet.

The only structural deficiency noted for the reservoir was that the minimum lap slice length according to ASCE 41-17 Tier 1 criteria are exceeded for reinforcing in the support columns.

Structural deficiencies comprise the following items:

- The ASCE 41-17 Tier 1 limit (unblocked wood structural panel diaphragms) was exceeded for the roof diaphragm spans in both directions.
- The adequacy of the truss blocking is unknown because submittal drawings/calculations from the roof truss manufacturer were not available for review. The trusses were not visible during SEFT's site visit.
- There is inadequate out-of-plane bracing for the north and south gable end masonry walls.

Mountain View Reservoir

The Mountain View Reservoir is a completely buried, strand-wound, circular, prestressed concrete structure with a flat roof. It has a capacity of 10 MG and was built in 1971. The reservoir has an approximate diameter of 292 feet and a maximum water depth of 20 feet.

Structural deficiencies comprise the following items:

- Seismic cables or dowels were not used to connect the base of the wall to the foundation. Therefore, the connection has inadequate strength to seismic lateral forces.
- The horizontal prestressing strands on the wall of the reservoir have inadequate capacity to resist hydrostatic and hydrodynamic hoop forces during an earthquake.
- The main structural deficiency of the reservoir is that the column vertical reinforcing lap splice length is less than the ASCE 41-17 Tier 1 specified minimum length for Immediate Occupancy.

Appendix E. Facilities Cost Estimate Summary

CLASS 5 OPINION OF PROBABLE CONSTRUCTION COST - BASIS OF ESTIMATE (CONFIDENTIAL)

City of Salem

Seismic Resiliency Study

B&V PROJECT NO. 406828

DATE PREPARED

22 MARCH 2023

BLACK & VEATCH
Building a world of difference.®

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Attachment A Summary of Vertical Facilities Costs

1.0 Introduction

1.01 PROJECT DESCRIPTION AND LOCATION

Black & Veatch (BV) developed cost estimates associated with recommended seismic improvements for vertical facilities and replacement of Low to Very High Risk pipelines.

The scope for recommended seismic improvements for vertical facilities is based upon a Draft Technical Memorandum (TM): Pump Station and Reservoir Seismic Vulnerability Assessment, dated September 6th, 2021 (SEFT Project Number: B20028.00).

The scope of replacement work for horizontal facilities (pipelines) was based upon a risk assessment conducted and described in the Seismic Resiliency Analysis Report (main report). Pipelines were first assessed for their consequence and likelihood of failure, then an overall risk score was applied. Pipelines with Low to Very High Risk are proposed to be preventatively replaced over a period of 50 years to improve systemwide resiliency, while Very Low Risk Pipes are proposed to remain and be repaired if needed after a major earthquake occurs. Pipeline replacement costs are based upon Black & Veatch’s cost library information and professional judgement. Assumptions for the pipeline replacement cost estimate are further described in the Seismic Resiliency Analysis Report.

1.02 PURPOSE AND DISCLAIMER

The Opinion of Probable Construction Cost (hereinafter "OPCC") is based on a conceptual level of design detail and information and are generally prepared based on very limited information and subsequently have wide accuracy ranges. The Class 5 OPCC is prepared for any number of strategic business planning purposes, such as market studies, assessment of initial viability, evaluation of alternate schemes, project screening, project location studies, evaluation of resource needs and budgeting, long-range capital planning, etc. The OPCC is based on expected capital construction cost only and does not consider life cycle costs or extended operation, maintenance, design or owner costs unless specifically included in the estimate details. The OPCC does not represent a certainty, and the final project costs may vary from the OPCC cost range presented to clients.

1.03 OPCC ORGANIZATION AND WORK BREAKDOWN STRUCTURE

Improvements were listed in the order presented in the TM with costs applied on a per improvement basis. Where complexity of the improvement required more detail, the improvement was broken down into further line items for clarity of scope and cost.

1.04 OPCC CLASSIFICATION AND ACCURACY

The OPCC can be considered consistent with an Association for the Advancement of Cost Engineering (AACE) Class 5 estimate. Typical accuracy ranges for Class 5 OPCC are -20% to - 50% on the low side, and +30% to +100% on the high side, depending on the technical complexity of the project, and appropriate contingency determinations. Ranges could exceed those shown in unusual circumstances.

The expected accuracy range for the OPCC is based on confidence and assessment of the quality and reliability of information used by the estimator. The range for this project is expected to be -30% to +50% low to high.

2.0 Basis of Estimate

2.01 ESTIMATING METHODOLOGY

A combination of methods, techniques and data sources are used in development of the OPCC.

For areas where quantities are provided with the design criteria, the values are incorporated into the OPCC and compared along with pricing based on both historical unit costs and built-up estimated costs. For estimating scope where quantities are unknown or unclear, the OPCC uses a combination of parametric factoring of known costs for similar systems and analogous projects with comparable corresponding features and sizing.

Where estimating scope was required but specific sizing could not be determined, costs are based on expert judgement and the use of allowances to meet an expected range of accuracy. In some instances, the estimator consults with process or subject matter experts to more clearly define project requirements to meet the confidence level in the allowance made.

2.02 ASSUMPTIONS AND ALLOWANCES

Where assumptions have been made to cover gaps in the scope of work or supply of components, the assumptions have been identified in the OPCC with the leading term "assumed" or "assume" followed by a description of the work. Similarly, where allowances have been made for costs that are not quantifiable or lack sufficient detail to price, the allowances have been identified with the leading term "allow" or "allowance".

2.03 DIRECT COSTS

The OPCC includes direct costs for labor, permanent and incidental materials, construction equipment based on unit pricing for similar projects in the West Coast US Region.

2.04 MARKET CONDITION

Where market conditions in the project location are volatile or know to have extremes, we include a Market Adjustment. This adjustment takes into account unusual project circumstances that would otherwise have little basis for inclusion, including labor shortages and market fluctuations.

2.05 MARKUPS, TAXES AND INSURANCE

The OPCC builds up costs from direct construction and adds markups to represent a complete price for the scope of work representing the methodology a prime contractor would use. The OPCC is only a representation of how contractors may apply markups.

From years of tracking projects, we have determined markup ranges that are applied to direct costs as an aggregate. The aggregate factor used has been calculated for this OPCC to be the appropriate amount based on our experience.

2.06 ESCALATION

All costs are in 2022 dollars.

2.07 ENGINEERING AND CONSTRUCTION MANAGEMENT

Engineering and construction management costs were calculated for each of the identified scope items in the SEFT Seismic Resiliency TM. A minimum 30% multiplier was applied to base construction costs for engineering and construction management for vertical facilities, and a multiplier of 20 to 30% was applied to base costs for pipelines. For many of the scope items, engineering and construction management cost factors higher than 30% were used, due to the high proportion of engineering that is likely required to provide design and engineering relative to the cost of construction for these items.

2.08 CONTINGENCIES

Contingency is included in the OPCC and evaluated based on how complete the scope of work and OPCC are. Contingency at Class 5 level is often assessed at 30%; this contingency was applied to vertical facility cost estimates. While there are norms for contingency, the OPCC considers several factors in the assessment including range of accuracy, completion of scope, quality of cost data and systemic or perceived risk to the contractor. For pipelines, a contingency of 40% was applied, since less is known about the specific conditions for each pipeline.

In addition to this contingency applied at the end of the estimate, some scope items were identified as “contingency” scope additions; these items of work may be required after further study or assessment of the vertical facilities. These additional scope items may not be comprehensive to all required improvements at vertical facilities.

Through the process of creating the OPCC, any clarifications that would have a significant impact on the project were noted in the cost items comments.

2.09 EXCLUSIONS

Based on the discipline estimator’s understanding of the project some scope may be specifically excluded from the OPCC value. Where costs have been excluded, they are identified in the OPCC with the leading term “exclude”, “excluded”, “not in cost” or “NIC”. Exclusions that have not been explicitly identified in the OPCC are listed in this section by the estimator. The following exclusions are not expected to be required in the improvements scope of work.

- Electrical if required
- Instrumentation if required
- Communications if required
- Right of Way Acquisition

2.10 OPCC CALCULATION ASSUMPTIONS

Assumptions and markups for the pipelines and vertical facilities cost estimates are further described in the Seismic Resiliency Analysis Report (main report), in Section 6.2, “Basis for Establishing Opinion of Probable Construction Costs”.

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ATTACHMENT A

Facility	**Known issues**	**Additional Studies**	**Total Base Costs**	**Potential Work Resulting from Studies**	**Total Potential Costs**
ASR 1&2	$180,000	$49,000	$229,000	$100,000	$329,000
ASR 4	$100,000	$36,000	$136,000	$0	$136,000
ASR 5	$60,000	$65,000	$125,000	$170,000	$295,000
Creekside PS	$120,000	$94,000	$214,000	$80,000	$294,000
Deer Park PS	$130,000	$62,000	$192,000	$190,000	$382,000
Mountain View PS	$230,000	$11,000	$241,000	$30,000	$271,000
Salem Keiser Intertie #1	$140,000	$21,000	$161,000	$10,000	$171,000
Turner Control Facility	$70,000	$29,000	$99,000	$100,000	$199,000
Candalaria Reservoir	$10,000	$101,000	$111,000	$240,000	$351,000
Champion Hill Reservoir	$100,000	$8,000	$108,000	$0	$108,000
Champion Hill Reservoir Control Bldg	$180,000	$6,000	$186,000	$10,000	$196,000
Edwards PS	$190,000	$11,000	$201,000	$810,000	$1,011,000
Fairmount Reservoir	$2,650,000	$29,000	$2,479,000	$390,000	$2,869,000
Fairmount Res. Control Bldg	$140,000	$18,000	$158,000	$30,000	$188,000
Grice Hill Res Control Bldg	$150,000	$0	$150,000	$0	$150,000
Lone Oak Res. Cntrl Bldg	$30,000	$44,000	$74,000	$10,000	$84,000
Mill Creek Reservoir	$40,000	$8,000	$48,000	$940,000	$988,000
Mill Creek#1 Res. Cntrl. Bldg	$60,000	$44,000	$104,000	$150,000	$254,000
Mountain View Reservoir	$3,790,000	$0	$3,590,000	$70,000	$3,660,000
Eolia 1B Seismic Valve	$200,000				$200,000
Subtotal - High Priority	**$8,570,000**	**$636,000**	**$8,606,000**	**$3,330,000**	**$12,136,000**
Boone Road PS	$110,000	$25,000	$135,000	$140,000	$275,000
Limelight PS	$100,000	$67,000	$167,000	$310,000	$477,000
Eola #1B Reservoir	$80,000	$8,000	$88,000	$20,000	$108,000
Grice Hill Reservoir	$20,000	$0	$20,000	$20,000	$40,000
Lone Oak Reservoir	$0	$0	$0	$0	$0
Subtotal - Medium Priority	**$310,000**	**$100,000**	**$410,000**	**$490,000**	**$900,000**
Total Program Costs (rounded)	**$8,880,000**	**$740,000**	**$9,020,000**	**$3,820,000**	**$13,040,000**

ATTACHMENT A

ASR 1 & 2

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Construction Cost	Engineering Cost	Total Construction Cost
Correct Wall Connection	Install vertical steel angles where the east-west oriented CMU walls of the ASR #2 addition interface with the west wall of the original ASR #1 structure.		24	LF	$2,756	$1,575	$4,331
Further Assessment Necessary	Remove existing gypsum board interior finish to investigate the load path to transfer seismic roof diaphragm forces at the roof step between the ASR #1 and ASR #2 portions of the structure to the masonry wall below. Likely add a combination of plywood sheathing, blocking, and metal connector hardware to provide an adequate load path.	Study			$12,185	$8,400	$20,585
Remediate Deficiency in Roof Truss to Sheathing Nailing,		Contingency	80	hrs	$20,717	$10,500	$31,217
New Shaped Blocking	Install new shaped blocking between the roof sheathing and masonry wall top plate. Provided boundary nailing between the roof sheathing and blocking, and Simpson A35 clip angles between the blocking and top plate and blocking and trusses.		648	SF Footprint	$20,108	$7,875	$27,983
Correct CMU Wall Support	Install a combination of plywood sheathing, blocking, steel straps, and metal connector hardware to provide adequate out-of-plane support for CMU walls		1	LS	$14,816	$6,300	$21,116
Further Assessment Necessary	Conduct nondestructive scanning to verify the size and location of masonry wall vertical reinforcing. If vertical trim reinforcing is not provided adjacent to door openings, install face mounted steel plates through-bolted to masonry wall and anchored to foundation.	Study	1	0	$7,488	$2,771	$10,259
Repair Door Opening		Contingency	3	EA - assumed openings	$4,223	$6,300	$10,523
Pipe Bracing	Provide bracing for the piping		8	EA - assumed Locations	$14,756	$5,460	$20,216
Valve Bracing	Provide independent bracing for the valves.		4	EA	$10,014	$3,705	$13,719
Valve Bracing	Provide independent support and bracing for the air relief valve		1	EA	$2,503	$2,100	$4,603
Pump Bracing	Vertical pump motors are not braced above the center of gravity of the motor	Contingency	2	EA	$8,446	$3,150	$11,596
Pump Bracing	Brace Pump(s) motor(s).	Contingency	3	EA	$12,670	$4,688	$17,357
Further Assesment Necessary	Perform additional analysis to evaluate the adequacy of the existing connection of the motor to the top of the steel motor support, the motor support, the connection of the motor support to the concrete pedestal, and the concrete pedestal.	Study			$0	$4,200	$4,200
Wall Bracing Improvements	Provide anchorage/bracing between the top of the electrical cabinets and wall.		20	LF	$1,393	$1,575	$2,968
Lens Cover	Provide lens covers for light fixtures with safety devices that prevent the lens cover and light bulbs from falling.		8	EA	$2,456	$909	$3,365
Electrical Flexible Conduits	Provide flexible couplings where conduits connect to the top of wall-mounted electrical panels and cabinets.		10	EA	$7,538	$2,789	$10,327
Further Assessment Necessary	Verify the adequacy of the connection between the horizontal antenna and the supporting pole.	Study	1	EA	$4,133	$1,529	$5,663
Contingency Item to seismically strengthen antenna.		Contingency	1	LS	$5,591	$2,100	$7,691
Correct Wall Connection	Add helical wall ties between the masonry veneer and the ASR #1 masonry walls.				$16,224	$6,003	$22,227
Further Assessment Necessary	Conduct nondestructive scanning to verify the size and spacing of reinforcing in the architectural concrete pillars and perform calculations to verify the adequacy of the existing reinforcing. If reinforcing is found to be inadequate, remove existing architectural concrete pillars	Study			$0	$8,400	$8,400
C0ntingency Item - Remediate structural Deficiencies in architectural concrete pillars	If assessment finds need to reinforce the columns, assume that steel bracing will be provided on outside of the columns.	Contingency			$7,288	$6,300	$13,588
Anchor Electrical Transformer	Coordinate with electrical utility to anchor the utility owned electrical transformer to the concrete pad, as required.		1	LS	$2,037	$3,150	$5,187
Pump Flexible Couplings	Add flexible couplings between the pumps and the connected piping		2	EA	$26,537	$9,819	$36,356
	Total Cost				**$203,881**	**$109,597**	**$313,478**

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ASR #4

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Construction Cost	Engineering Cost	Total Construction Cost
Roof Truss Inspection	Remove a small section of existing roofing to verify the adequacy of the existing roof sheathing to truss nailing.	Study			$6,470	$6,300	$12,770
New Shaped Blocking	Install new shaped blocking between the roof sheathing and masonry wall top plate. Provided boundary nailing between the roof sheathing and blocking, and Simpson A35 clip angles between the blocking and top plate and blocking and trusses.	Contingency	360	SF Approx Footprint	$7,548	$6,300	$13,848
Further Assessment Necessary	Perform additional analysis to investigate if the diaphragm has adequate capacity to transfer seismic forces from the roof diaphragm to the east masonry shear wall, considering the impact of the hatch opening adjacent to the wall. Note that this analysis will require information about the existing roof sheathing to truss nailing (i.e., size, and spacing).	Study		ENG ONLY	$0	$8,400	$8,400
Install Simpson A35 Clips	Install Simpson A35 clips between truss bottom chord and masonry wall top plate in conjunction with cross tie retrofit described in next bullet item	Contingency			$2,964	$4,200	$7,164
Correct CMU Wall Support	Install a combination of plywood sheathing, blocking, steel straps, and metal connector hardware to provide adequate out-of-plane support for CMU walls	Contingency	1	LS	$10,643	$4,200	$14,843
Further Assessment Necessary	Conduct nondestructive scanning to verify the size and location of masonry wall vertical reinforcing. If vertical trim reinforcing is not provided adjacent to door openings, install face mounted steel plates through-bolted to masonry wall and anchored to foundation.	Study	1	0	$7,488	$2,771	$10,259
Repair Door Opening		Contingency		EA - assumed openings	$4,223	$6,300	$10,523
Pipe Bracing	Provide bracing for the piping.		8	EA - assumed Locations	$14,756	$5,460	$20,216
Valve Bracing	Provide independent bracing for the valves.		4	EA	$7,378	$2,730	$10,108
Valve Bracing	Provide independent support and bracing for the air relief valve.		4	EA	$7,378	$2,730	$10,108
Pump Bracing	Brace Pump(s) motor(s).	Contingency	3	EA	$12,670	$4,688	$17,357
Further Assesment Necessary	Perform additional analysis to evaluate the adequacy of the existing connection of the motor to the top of the steel motor support, the motor support, the connection of the motor support to the concrete pedestal, and the concrete pedestal.	Study			$0	$4,200	$4,200
Anchor Control Cabinet to Wall	Anchorage of pump station control cabinet to floor or wall	Yes	6	LF	$517	$1,575	$2,092
Lens Cover	Provide lens covers for light fixtures with safety devices that prevent the lens cover and light bulbs from falling.		4	EA Assumed	$1,228	$454	$1,682
Flexible Coupling	Add flexible couplings between the pump and the connected piping.		1	EA	$13,269	$4,909	$18,178
Anchor Electrical Transformer	Coordinate with electrical utility to anchor the utility owned electrical transformer to the concrete pad, as required.		1	LS	$2,037	$3,150	$5,187
	Total Cost				**$98,568**	**$68,367**	**$166,935**

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ASR #5

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Construction Cost	Engineering Cost	Total Construction Cost
Correct Wall Connection	Add shaped blocking or framing/sheathing with metal connector hardware to provide a load path between the roof of the steel framed pavilion and masonry shear walls of the pump station structure.		480	SF Approx footprint	$10,064	$3,724	$13,788
Further Assessment Necessary	Perform an investigation to determine if the ceiling diaphragm is adequately connected to the interior masonry walls to engage the walls as part of the seismic force resisting system. If not, add blocking/framing to provide a load path between the ceiling diaphragm and interior masonry shear walls.	Study	1	LS - Remove & Replace existing finish for inspection	$17,290	$8,400	$25,690
Contingency - add ceiling to wall bracing if found necessary in investigation		Contingency	480	SF Approx footprint	$6,500	$8,400	$14,900
Further Assessment Necessary	Perform an investigation of the existing ceiling nail size and spacing to verify the adequacy of the existing ceiling sheathing to joist nailing.	Study		ENG ONLY	$0	$2,100	$2,100
Contingency - Add additional nailing	Repairs needed, as a result of item above	Contingency	480	SF Approx footprint	$3,120	$3,150	$6,270
New Shaped Blocking	Perform an investigation to determine if adequate blocking and connections are provided between the ceiling sheathing and masonry wall top plate. Install new blocking with boundary nailing between the ceiling sheathing and blocking, and Simpson A35 clip angles between the blocking and top plate and blocking and ceiling joists, as appropriate.	Study		ENG ONLY	$0	$2,100	$2,100
Install Simpson A35 Clips	Install Simpson A35 clips between ceiling joists and masonry wall top plate in conjunction with cross tie retrofit described in next bullet item.	Contingency			$0	$0	$0
Correct CMU Wall Support	Install a combination of blocking, steel straps, and metal connector hardware to provide adequate out-of-plane support for CMU wall	Contingency	1	LS	$24,180	$8,947	$33,127
Further Assessment Necessary	Conduct nondestructive scanning to verify the size and location of masonry wall vertical reinforcing. If vertical trim reinforcing is not provided adjacent to door openings, install face mounted steel plates through-bolted to masonry wall and anchored to foundation.	Study	1	0	$7,540	$2,790	$10,330
Repair Door Opening		Contingency		EA - assumed openings	$4,290	$6,300	$10,590
Corrosion Assesment	Investigate extent and severity of the corrosion damage to the steel column, repair damage (as appropriate), and mitigate cause of moisture to prevent similar future damage.	Study			$0	$3,150	$3,150
Remediate steel Corrosion		Contingency	6	ea	$51,220	$18,951	$70,171
Pipe Bracing	Provide bracing for the piping.		8	EA - assumed Locations	$14,820	$5,483	$20,303
Valve Bracing	Provide independent bracing for the valves.		4	EA	$7,410	$2,742	$10,152
Valve Bracing	Provide independent support and bracing for the air relief valve		4	EA	$1,950	$722	$2,672
Pump Bracing	Brace Pump(s) motor(s).	Contingency	3	EA	$12,740	$4,714	$17,454
Further Assesment Necessary	Perform additional analysis to evaluate the adequacy of the existing connection of the motor to the top of the steel motor support, the motor support, the connection of the motor support to the concrete pedestal, and the concrete pedestal.	Study			$0	$4,200	$4,200
Flexible Coupling	Add flexible couplings between the pumps and the connected piping.	Contingency	1	EA	$13,390	$4,954	$18,344
Electrical Flexible Coupling	Provide flexible couplings where conduits connect to the top of floor- and wall-mounted electrical panels and cabinets		10	EA	$7,540	$2,790	$10,330
Anchor Pump Station Control Cabinet	Provide anchorage/bracing of pump station control cabinet to floor and wall.		6	LF	$520	$1,575	$2,095
Lens Cover	Provide lens covers for light fixtures with safety devices that prevent the lens cover and light bulbs from falling.		4	EA Assumed	$1,300	$481	$1,781
Anchor Electrical Transformer	Coordinate with electrical utility to anchor the utility owned electrical transformer to the concrete pad, as required.		1	LS	$2,080	$3,150	$5,230
	Total Cost				**$185,954**	**$98,822**	**$284,776**

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Boone Road Pump Station

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Construction Cost	Engineering Cost	Total Construction Cost
Further Assesment Necessary	Perform an investigation to determine if the gable end framing, sheathing nailing, and connection details are adequate to deliver seismic forces from the upper roof to the lower roof. If not, add supplemental nailing, framing, blocking, and/or metal connector hardware, as appropriate.	Study			$6,931	$8,400	$15,331
Contingency Item - Remediate Gable	This is the cost to remediate the above repairs if needed.	Contingency			$11,700	$8,400	$20,100
Wood Structural Overlay	Install a wood structural panel overlay on top of the existing straight sheathing. The joints of the wood structural panels should be placed so that they are near the center of the existing sheathing boards or at a 45-degree angle to the joints between existing sheathing boards.	Contingency			$58,760	$21,741	$80,501
Correct CMU Wall Support	Install a combination of sub-diaphragm framing and connection hardware at the roof level to provide adequate out-of-plane support for CMU walls	Contingency	140	LF	$3,640	$1,347	$4,987
Pipe Bracing	Provide bracing for the piping.		3	EA	$11,180	$4,137	$15,317
Valve Bracing	Provide independent bracing for the valves		3	EA	$11,180	$4,137	$15,317
Pump Bracing	Brace Pump(s) motor(s).	Contingency	3	EA	$12,740	$4,714	$17,454
Further Assesment Necessary	Perform additional analysis to evaluate the adequacy of the existing connection of the motor to the top of the steel motor support, the motor support, the connection of the motor support to the concrete pedestal, and the concrete pedestal.	Study			$0	$4,200	$4,200
Flexible Coupling	Add flexible couplings between the pumps and the connected piping.		3	EA	$39,910	$14,767	$54,677
Cable Tray Bracing	Provide longitudinal bracing for the cable tray. Reconfigure the anchorage of the transverse bracing strut to avoid anchorage into the masonry head joints.		1	LS	$9,360	$3,463	$12,823
Lens Cover	Provide lens covers for light fixtures with safety devices that prevent the lens cover and light bulbs from falling		4	EA Assumed	$1,300	$481	$1,781
Support Framing Bracing	Provide grating clip connections between the grating and steel support framing		16	LF	$1,950	$722	$2,672
Further Assessment Necessary	Verify the adequacy of the connection between the horizontal antenna and the supporting pole.	Study	1	EA	$4,160	$1,539	$5,699
Contingency Item to seismically strengthen Antenna.		Contingency	1	LS	$5,720	$2,116	$7,836
	Total Cost				**$178,531**	**$80,163**	**$258,694**

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Candalaria Reservoir and Valve Vault

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Construction Cost	Engineering Cost	Total Construction Cost
ASCE Tier 2 Assessment	Perform a future ASCE 41 Tier 2 assessment to confirm the adequacy of the provided column reinforcing lap splice length. SEFT indicates that they don't anticipate this being an issue.	Study			$0	$8,400	$8,400
Further Assessment Necessary	Verify that piping, fittings, and valve bodies are constructed of steel or ductile iron. Replace any components that are suspected to be cast iron.	Study	3	Days	$53,950	$19,962	$73,912
Contingency Item	Replace significant portion of reservoir Piping internals.	Contingency			$168,740	$62,434	$231,174
Further Assessment Necessary	Perform additional analysis to evaluate the adequacy of the overflow pipe and valve operator riser shafts to resist seismic forces. Provide lateral bracing of the overflow pipe and valve operator riser shafts, as required	Study	3	EA	$13,390	$4,954	$18,344
Backup Battery Restraints	Provide restraint of backup batteries, as required		1	EA	$520	$192	$712
	Total Cost				**$236,600**	**$95,942**	**$332,542**

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Champion Hill Reservoir Control Building

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Construction Cost	Engineering Cost	Total Construction Cost
Blocking Support	Add blocking to support the edges of the roof sheathing panels and provided boundary nailing between the roof sheathing and blocking.		1,748	SF Footprint	$36,660	$13,564	$50,224
Install Simpson A35 Clips	Install Simpson A35 clip angles, at approximately 2-feet on center, between gable end truss bottom chord and masonry wall top plate.		76	LF	$5,850	$2,165	$8,015
Correct Wall Connection	Install appropriate metal connector hardware to provide a vertical connection between the blocking that the kicker brace frames into and the adjacent roof trusses. Provide local strengthening of trusses, as appropriate.		92	LF	$7,540	$2,790	$10,330
Flexible Pipe Joints	Provide appropriate flexible joints where water system piping penetrates through the control building wall to accommodate potential differential movement between the structure and the surrounding soil at or near the pipe penetration.		2	ea	$31,590	$11,688	$43,278
Pipe Bracing	Provide bracing for the piping.		8	EA	$17,420	$6,445	$23,865
Valve Bracing	Provide independent bracing for the valves.		8	EA	$9,490	$3,511	$13,001
Pump Bracing	Provide independent bracing for the recirculation pump and associated piping.		1	LS	$3,380	$1,251	$4,631
Pressure Tank Bracing	Provide an additional anchor at the base of the pressure tank.		1	LS	$520	$192	$712
Backup Battery restraints	Provide restraint for backup batteries inside the battery cabinet.		1	LS	$910	$337	$1,247
Further Assessment Necessary	Verify the adequacy of the antenna connection to the supporting pole.	Study	1	EA	$4,160	$1,539	$5,699
	Jim indicates that enhancement of attachement between antenna and pole likely, not bracing. Cost includes cost for a bucket truck for installation.	Contingency	1	LS	$5,720	$2,116	$7,836
Correct Wall Connection	Install a combination of blocking and metal connector hardware to provide adequate connections to transfer seismic forces from the ceiling to the masonry walls and eliminate the potential for cross-grain bending of wood ledgers.		1	LS	$10,400	$3,848	$14,248
Electrical Cabinet Restraint	Provide restraint for temporarily stored electrical cabinets and other components (using straps to the wall, etc.).		1	LS	$1,430	$529	$1,959
Electrical Transformer Bracing	Coordinate with electrical utility to conduct further evaluation to validate the seismic performance of pole-mounted transformer and utility pole.			ENG	$0	$175	$175
	Total Cost				**$135,070**	**$50,151**	**$185,221**

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Champion Hill Reservoir

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Construction Cost	Engineering Cost	Total Construction Cost
ASCE Tier 2 Assessment	Perform a future ASCE 41 Tier 2 assessment to evaluate the adequacy of the provided column reinforcing lap splice length	Study		ENG	$0	$8,400	$8,400
Connection Brackets	Install connection brackets at the corners of the pipe support pedestals that are anchored to both the concrete pedestal and concrete floor slab.				$71,890	$26,599	$98,489
	Total Cost				**$71,890**	**$34,999**	**$106,889**

ATTACHMENT A

Creekside Pump Station

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Construction Cost	Engineering Cost	Total Construction Cost
Further Investigation - Roof Truss Inspection	Remove a small section of existing roofing to verify the adequacy of the existing roof sheathing to truss nailing.	Study	4	days	$57,857	$21,407	$79,264
Roof Blocking	Add blocking to support the edges of the roof sheathing panels and provided boundary nailing between the roof sheathing and blocking.	Contingency	940	SF Footprint	$19,760	$7,311	$27,071
Further Assessment Necessary	Perform an investigation to verify the adequacy of the connection between the roof sheathing and gable end masonry wall top plate.	Study		ENG	$0	$10,500	$10,500
Correct CMU Wall Support	Install a combination of plywood, blocking, steel straps, and metal connector hardware to provide adequate out-of-plane support for CMU walls	Contingency	124	LF	$24,180	$8,947	$33,127
Pipe Bracing	Provide bracing for the piping.		3	EA	$11,180	$4,137	$15,317
Valve Bracing	Provide independent bracing for the valves.		3	EA	$11,180	$4,137	$15,317
Valve Bracing	Provide independent support and bracing for the air relief valves.		3	EA	$3,640	$1,347	$4,987
Pump Bracing	Brace Pump(s) motor(s).	Contingency	3	EA	$12,740	$4,714	$17,454
Further Assesment Necessary	Perform additional analysis to evaluate the adequacy of the existing connection of the motor to the top of the steel motor support, the motor support, the connection of the motor support to the concrete pedestal, and the concrete pedestal.	Study			$0	$4,200	$4,200
Flexible Coupling	Add flexible couplings between the pumps and the connected piping.		3	EA	$39,910	$14,767	$54,677
Contingency Item - Cabinet Anchorage	This assumes the electrical cabinets in item above require seismic reinforcement.	Contingency	6	LF	$520	$1,575	$2,095
Correct Wall Connections	Provide anchorage/bracing between the top of the electrical cabinets and masonry wall.		6	LF	$520	$192	$712
Emergency Generator Bracing	Provide anchorage/bracing for emergency generator air intake support frame, muffler, and exhaust pipe.		3	EA	$11,050	$4,089	$15,139
Anchor Electrical Transformer	Coordinate with electrical utility to anchor the utility owned electrical transformer to the concrete pad, as required.		1	LS	$2,080	$3,150	$5,230
Total Cost					**$194,617**	**$90,471**	**$285,089**

ATTACHMENT A

Deer Park Pump Station

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Construction Cost	Engineering Cost	Total Construction Cost
Further Assessment Necessary	Conduct nondestructive scanning to verify the size and location of masonry wall reinforcing and evaluate the adequacy of the masonry walls.	Study	4	ea	$40,876	$15,124	$56,000
Contingency Item - Replace Superstr	Entire CMU superstructure may need to be replaced with equipment in place. (44 ft x 2- ft structure)	Contingency	1,280	SF Area	$43,290	$16,017	$59,307
Roof Replacement	Based on the number of potential deficiencies identified that are associated with the wood framed roof, suggest removing existing roof and replacing with new plywood sheathed wood truss roof with appropriate seismic detailing (including consideration of cross ties between diaphragm chords and out of plane bracing for perimeter and interior masonry walls).	Contingency	1,300	SF Approx Roofing Area	$88,660	$32,804	$121,464
Pipe Bracing	Provide bracing for the piping.		3	EA	$11,180	$4,137	$15,317
Valve Bracing	Provide independent bracing for the valves		3	EA	$11,180	$4,137	$15,317
Pump Support/Bracing	Install angles all around the perimeter of the pump support concrete pedestal with anchors into the floor slab. On two opposing sides, add a pair of steel straps that are welded to the angle and anchored up the face of the pedestal.		3	EA	$18,070	$6,686	$24,756
Flexible Couplings	Add flexible couplings between the pumps and the connected piping.		3	EA	$39,910	$14,767	$54,677
Pipe Support Amchoring	Add anchors through the pipe support stanchion base plates into the slab-on-grade at locations with missing anchors.		12	EA	$3,640	$1,347	$4,987
Contingency Item - Cabinet Anchora	This assumes the electrical cabinets in item above require seismic reinforcement.	Contingency	6	LF	$520	$1,575	$2,095
Emergency Generator bracing	Re-install the restrainer bracket for the emergency generator starter batteries, as appropriate.	Contingency	1	LS	$3,900	$1,443	$5,343
Further Assessment Necessary	Verify the adequacy of the antenna connection to the supporting pole.	Study	1	EA	$4,160	$1,539	$5,699
Anchor Electrical Transformer	Coordinate with electrical utility to conduct further evaluation to validate the seismic performance of pole-mounted transformer and utility pole.		1	LS	$2,080	$3,150	$5,230
TOTAL COST					**$267,466**	**$102,725**	**$370,192**

ATTACHMENT A

Edwards Pump Station

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Construction Cost	Engineering Cost	Total Construction Cost
Replace Pump Station	Based on the age of the structure and the number of potential deficiencies identified, it is recommended that the City consider replacing the Edwards Pump Station structure.	Contingency	1200	SF Approx Building Footprint	$569,140	$210,582	$779,722
Flexible Pipe Joints	Provide appropriate flexible joints where water system piping penetrates through the pump station floor to accommodate potential differential movement between the structure and the surrounding soil at or near the pipe penetration.		3	EA	$39,910	$14,767	$54,677
Pipe Bracing	Provide bracing for the piping.		3	EA	$10,790	$3,992	$14,782
Valve Bracing	Provide independent bracing for the valves.		3	EA	$10,790	$3,992	$14,782
Pump Bracing	Brace Pump(s) motor(s).	Contingency	3	EA	$12,740	$4,714	$17,454
Further Assesment Necessary	Perform additional analysis to evaluate the adequacy of the existing connection of the motor to the top of the steel motor support, the motor support, the connection of the motor support to the concrete pedestal, and the concrete pedestal.	Study			$0	$4,200	$4,200
Flexible Couplings	Add flexible couplings between the pumps and the connected piping.		3	EA	$39,910	$14,767	$54,677
Further Asessment Necessary	Perform an investigation to evaluate the adequacy of the anchorage of electrical cabinets to housekeeping pads and housekeeping pads to slab on grade, and supplement anchorage (as required). Also, provide anchorage/bracing between the top of the electrical cabinets and masonry wall.	Study	6	LF	$1,040	$385	$1,425
Flexible Electrical Couplings	Provide flexible couplings where conduits connect to the top of floor- and wall-mounted electrical panels and cabinets.		20	EA	$15,080	$5,580	$20,660
Emergency Generator Bracing	Provide restraint for the emergency generator starter batteries		1	LS	$2,470	$914	$3,384
Lens Cover	Provide lens covers for light fixtures with safety devices that prevent the lens cover and light bulbs from falling.		4	EA Assumed	$1,300	$481	$1,781
Further Assessment Necessary	Verify the adequacy of the antenna connection to the supporting pole.	Study	1	EA	$4,160	$1,539	$5,699
Contingency Item to seismically strengthen Antenna.	Jim indicates that enhancement of attachement between antenna and pole likely, not bracing. Cost includes cost for a bucket truck for installation.	Contingency	1	LS	$5,720	$2,116	$7,836
HVAC Anchoring	Provide anchorage of the HVAC unit to the concrete pad.		1	LS	$2,470	$914	$3,384
Grating Clips	Provide grating clip connections between the grating and steel support framing.		16	LF	$1,950	$722	$2,672
Crane Restraints	Provide restraint for the overhead bridge crane, when not in use.		1	LS	$4,420	$1,635	$6,055

ATTACHMENT A

Edwards Pump Station

Remedial Action	**Description**	**Category of Work (Base, Study, or Contingency)**	**Quantity**	**Unit**	**Construction Cost**	**Engineering Cost**	**Total Construction Cost**
Rolling Lift Restraints	Provide restraint for rolling lifts, when not in use.		1	LS	$1,690	$625	$2,315
Ladder Restraints	Provide restraint for ladders (using straps to the wall, etc.), when not in use.		1	LS	$780	$289	$1,069
Anchor Electrical Transformer	Coordinate with electrical utility to conduct further evaluation to validate the seismic performance of pole-mounted transformer and utility pole.		1	LS	$2,080	$3,150	$5,230
TOTAL COST					**$726,440**	**$275,363**	**$1,001,803**

ATTACHMENT A

Eola #1B Reservoir

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Construction Cost	Engineering Cost	Total Construction Cost
contingency Item - Repair roof Crack	Assume 1/3 of wall to roof connection will require roof dowels. This is a 92 ft dia reservoir that is completely buried, except for roof.	Contingency	110	ea	$14,040	$5,195	$19,235
ASCE Tier 2 Assessment	Perform a future ASCE 41 Tier 2 assessment to confirm the adequacy of the provided column reinforcing lap splice length. SEFT indicates that they don't anticipate this being an issue.	Study		ENG	$0	$8,400	$8,400
Stainless Steel Plates	Install stainless steel plates and/or angles to connect riser, base, and lid precast components at the precast concrete construction joints.		1	LS	$13,650	$5,051	$18,701
Pipe Bracing	Supplement existing bracing for vertical section of inlet pipe.		1	LS	$30,680	$11,352	$42,032
Valve Bracing	Perform additional analysis to evaluate the adequacy of the piping and valves to resist seismic forces (e.g., span between vault walls). Alternatively, provide bracing for the piping and valves inside the valve vaults.		1	ea	$11,570	$4,281	$15,851
	Total Cost				**$69,940**	**$34,278**	**$104,218**

ATTACHMENT A

Fairmont Reservoir Control Building

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Construction Cost	Engineering Cost	Total Construction Cost
Pipe Bracing	Provide bracing for the piping.		1	LS	$26,650	$9,861	$36,511
Valve Bracing	Provide independent bracing for the valves and valve actuators.		1	LS	$17,940	$6,638	$24,578
Pump Bracing	Provide bracing/restraint for vertical pump bells and valve operator riser shafts.		1	LS	$4,290	$1,587	$5,877
Pipe Replacement	Replace any piping and valve components that are suspected to be cast iron.		1	LS	$12,480	$4,618	$17,098
Corrosion Assessment and Replaceme	Replace any corrosion damaged piping and valve components and connection hardware not already replaced by the bullet item above.	Contingency	1	LS	$12,480	$4,618	$17,098
Pump Base Bracing	Provide anchorage between pump bases and concrete slab.		1	LS	$4,420	$1,635	$6,055
Further Assesment Necessary	Perform additional analysis to evaluate the adequacy of the existing connection of the motor to the top of the steel motor support, the motor support, the connection of the motor support to the concrete pedestal, and the concrete pedestal.	Study			$0	$4,200	$4,200
Further Assesment Necessary	Perform additional analysis to evaluate the adequacy of the existing connection of the motor to the top of the steel motor support, the motor support, the connection of the motor support to the concrete pedestal, and the concrete pedestal.	Study			$0	$4,200	$4,200
Flexible Couplings	Add flexible couplings between the pumps and the connected piping		1	LS	$31,590	$11,688	$43,278
Pipe Bracing	Provide bracing for the air vent vertical pipe		1	LS	$4,420	$1,635	$6,055
Further Assessment Necessary	Perform an investigation to evaluate the adequacy of the anchorage of electrical cabinets to concrete floor slab, and supplement anchorage (as required). Also, provide anchorage/bracing between the top of the electrical cabinets and concrete wall.	Study	12	LF	$2,990	$1,106	$4,096
Further Assessment Necessary	Verify the adequacy of the connection between the horizontal antenna and the supporting pole.	Study	1	EA	$4,160	$1,539	$5,699
Contingency Item to seismically strengthen Antenna.		Contingency	1	LS	$5,720	$2,116	$7,836
	Total Cost				**$127,140**	**$55,442**	**$182,582**

ATTACHMENT A

Fairmount Reservoir

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Construction Cost	Engineering Cost (% of Construction)	Engineering Cost	Total Construction Cost
Shotcrete	Add a 6-inch layer of shotcrete at the inside face of the perimeter walls and footings		683	CY	$1,704,768	$630,764	$630,764	$2,335,532
Correct Roof Connections	Provide stainless steel connections along the roof expansion joints to transfer shear forces between roof panels. Also, provide anchors between the roof slab and the walls to transfer roof seismic loads to the perimeter walls.		73,728	SF	$80,340	$29,726	$29,726	$110,066
Further Assessment Necessary	Perform a future structural assessment to evaluate the potential impact of the interaction between the Fairmount Reservoir and the integrally constructed Fairmount Reservoir Control Building/Pump Station.	Study		ENG	$0	$0	$21,000	$21,000
ASCE Tier 2 Assessment	Perform a future ASCE 41 Tier 2 assessment to confirm the adequacy of the provided column reinforcing lap splice length. SEFT indicates that this is unlikely to result in remedial work	Study		ENG	$0	$0	$8,400	$8,400
Contingency Item - Reservoir Mechanical Replacement of interior piping	Complete	Contingency			$280,410	$103,752	$103,752	$384,162
Electrical Transformer Bracing	Coordinate with electrical utility to conduct further evaluation to validate the seismic performance of pole-mounted transformer and utility pole.			ENG	$0	$0	$175	$175
	Total Cost				**$2,065,518**	**$764,242**	**$793,817**	**$2,859,335**

ATTACHMENT A

Grice Hill Reservoir Control Building

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Construction Cost	Engineering Cost	Total Construction Cost
Blocking Support	Add blocking to support the edges of the roof sheathing panels and provided boundary nailing between the roof sheathing and blocking.		1,665	SF Footprint	$34,909	$12,916	$47,825
Roof Truss Bracing	Install appropriate metal connector hardware to provide a vertical connection between the blocking that the kicker brace frames into and the adjacent roof trusses. Provide local strengthening of trusses, as appropriate.		1	LS	$23,400	$8,658	$32,058
Pipe Bracing	Provide bracing for the piping.		6	EA	$11,180	$4,137	$15,317
Valve Bracing	Provide independent bracing for the valves		3	EA	$5,590	$2,068	$7,658
Valve Seismic Improvement	Repair seismic valve so that is operational in the event of an earthquake.		1	LS	$7,540	$2,790	$10,330
Tank Bracing	Provide anchorage of the pressure tank.		1	LS	$1,430	$529	$1,959
Backup Battery Restraints	Provide additional restraint for backup batteries inside the battery cabinet		1	LS	$780	$289	$1,069
Correct Wall Connection	Install a combination of blocking and metal connector hardware to provide adequate connections to transfer seismic forces from the ceiling to the masonry walls and eliminate the potential for cross-grain bending of wood ledgers.		1	LS	$13,130	$4,858	$17,988
Electrical Cabinet Restraints	Provide restraint for temporarily stored electrical cabinets and other components (using straps to the wall, etc.).		1	EA	$780	$289	$1,069
Ladder Restraints	Provide restraint for ladder (using straps to the wall, etc.), when not in use.		1	EA	$390	$144	$534
Anchor Electrical Transformer	Coordinate with electrical utility to anchor the utility owned electrical transformer to the concrete pad, as required.		1	LS	$2,080	$3,150	$5,230
	Total Cost				**$101,209**	**$39,828**	**$141,037**

ATTACHMENT A

Grice Hill Reservoir

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Total Construction Cost	Study Work	Contingency Work
ASCE 24 Tier 2 Assessment	Perform a future ASCE 41 Tier 2 assessment to confirm the adequacy of the provided column reinforcing lap splice length. SEFT indicates that they don't believe this will result in remedial work	Study	1	Assessment	$19,632	$0	$19,632
	Total Cost				**$35,127**	**$0**	**$19,632**

ATTACHMENT A

Limelight Pump Station

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Construction Cost	Engineering Cost	Total Construction Cost
Further Assessment Necessary	Perform an evaluation of the potential impact of the vertical cracks in the masonry shear walls on the seismic performance of the pump station and implement an appropriate repair concept. Implement repairs of localized deterioration of plywood sheathing and framing to restore these components to their original strength	Study	1	LS	$15,575	$5,763	$21,338
Contingency Item - Replace superstructure	Replace pump Station superstructure.	Contingency			$158,600	$58,682	$217,282
New Shaped Blocking	Install new shaped blocking between the roof sheathing and masonry wall top plate. Provided boundary nailing between the roof sheathing and blocking, and Simpson A35 clip angles between the blocking and top plate and blocking and trusses. Also, suggest engaging the shear resistance of the interior masonry wall by adding a combination of plywood sheathing, framing/blocking, and metal connector hardware to provide a load path between the roof diaphragm and the interior masonry wall.		820	SF Footprint	$17,290	$6,397	$23,687
Further Assessment Necessary	Perform an investigation to verify the adequacy of the connection between the roof sheathing, gable end triangular portion wood framed shear walls and masonry wall top plate below.	Study			$23,010	$8,514	$31,524
Correct CMU Wall Support	Install a combination of plywood sheathing, blocking, steel straps, and metal connector hardware to provide adequate out-of-plane support for CMU walls. The concept is similar to that shown in Figure 4.5, for the Creekside Pump Station, except that all three sub-diaphragms should be installed with added plywood at the at the ceiling level, since the masonry portion of the gable end walls		116	LF	$24,960	$9,235	$34,195
Pipe Bracing	Provide bracing for the piping.		3	EA	$11,180	$4,137	$15,317
Valve Bracing	Provide independent bracing for the valves.		3	EA	$11,180	$4,137	$15,317
Valve Bracing	Provide independent support and bracing for the air relief valves.		3	EA	$3,640	$1,347	$4,987
Pump Bracing	Brace Pump(s) motor(s).	Contingency	3	EA	$12,740	$4,714	$17,454
Further Assesment Necessary	Perform additional analysis to evaluate the adequacy of the existing connection of the motor to the top of the steel motor support, the motor support, the connection of the motor support to the concrete pedestal, and the concrete pedestal.	Study			$0	$4,200	$4,200
Flexible Couplings	Add flexible couplings between the pumps and the connected piping.	Contingency	3	EA	$39,910	$14,767	$54,677
Further Assessment Necessary	Perform an investigation to evaluate the adequacy of the anchorage of electrical cabinets to housekeeping pads and housekeeping pads to slab on grade, and supplement anchorage (as required). Also, provide anchorage/bracing between the top of the electrical cabinets and masonry wall.	Study	20	LF	$2,990	$1,106	$4,096
Contingency Item - Cabinet Anchorage	This assumes the electrical cabinets in item above require seismic reinforcement.	Contingency	6	LF	$520	$1,575	$2,095
Further Assesment Necessary	Verify the adequacy of the antenna connection to the supporting pole.	Study	1	EA	$4,160	$1,539	$5,699
Seismically strengthen Antenna	This is a contingency item in case work is needed as determined in the item above.	Contingency	1	LS	$6,240	$2,309	$8,549
Anchor Electrical Transformer	Coordinate with electrical utility to anchor the utility owned electrical transformer to the concrete pad, as required.		1	LS	$2,080	$3,150	$5,230
	Total Cost				**$334,075**	**$131,571**	**$465,646**

ATTACHMENT A

Lone Oak Reservoir Control Building

Remedial Action	Description	Cost Estimating Assumptions	Quantity	Unit	Construction Cost	Engineering Cost	Total Construction Cost
Pipe Bracing	Provide bracing for the piping.		6	EA	$11,180	$4,137	$15,317
Valve Bracing	Provide independent bracing for the valves.	Assume one floor mounted pipe	3	EA	$5,590	$2,068	$7,658
Anchor Pressure Tank	Provide anchorage of the pressure tank.		1	LS	$1,430	$529	$1,959
Further Assessment Necessary	Perform an investigation to evaluate the adequacy of the anchorage of control cabinet to housekeeping pad, and supplement anchorage (as required). Also, provide anchorage/bracing between the top of the control cabinet and masonry wall.		1	LS	$1,300	$481	$1,781
Further Assessment Necessary	Perform an investigation to evaluate the adequacy of the anchorage of the battery cabinet to the control cabinet, and supplement anchorage (as required).			ENG	$0	$0	$0
Backup Battery Restraints	Provide restraint for backup batteries inside the battery cabinet.		1	LS	$780	$289	$1,069
Further Assessment Necessary	Verify the adequacy of the antenna connection to the supporting pole		1	EA	$4,160	$1,539	$5,699
Further Assessment Necessary	Perform an investigation to evaluate the adequacy of the bracing of the suspended HVAC unit, and supplement bracing (as required).			ENG	$0	$525	$525
Contingency Item	Install HVAC Bracing for item above			960	$1,560	$577	$2,137
Further Assessment Necessary	Perform an investigation to evaluate the adequacy of the anchorage of the water heater/safety shower base skid to the concrete slab, and supplement anchorage (as required).			ENG	$0	$525	$525
Contingency Item	Install Bracing for item above			960	$1,560	$577	$2,137
Further Assessment Necessary	Perform an investigation of the original deferred submittal design details for the chlorine room masonry wall reinforcing and top of wall bracing. Provide supplemental bracing of masonry walls, as required	DC - Please add contractor cost for 4 days contractor crew time	4	Days	$26,000	$9,620	$35,620
Ladder Restraints	Provide restraint for ladder (using straps to the wall, etc.), when not in use.		1	LS	$650	$241	$891
Anchor Electrical Transformer	Coordinate with electrical utility to anchor the utility owned electrical transformer to the concrete pad, as required.		1	LS	$780	$289	$1,069
	Total Cost				**$54,990**	**$21,396**	**$76,386**

ATTACHMENT A

Mill Creek #1 Reservoir Control Building

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Construction Cost	Engineering Cost	Total Construction Cost
Blocking Support	Add blocking to support the edges of the roof sheathing panels and provided boundary nailing between the roof sheathing and blocking.	Contingency	1,932	SF Footrpint	$40,507	$14,988	$55,495
Source Drawings for Further Assessment	Coordinate with the City to attempt to locate deferred submittal design drawings and calculations from original construction. Supplement available drawings with a detailed field investigation (including potential localized removal of architectural finishes) to observe and document details of the truss blocking and associated connections. Once additional details of original construction are available, evaluate the adequacy of the load path to transfer seismic forces from the roof diaphragm to the masonry wall top plate and develop mitigation concepts, as appropriate.	Study	4	days	$24,960	$9,235	$34,195
Truss Bracing	Install a combination of blocking and steel straps between truss bottom chord members in four additional truss bays per line of blocking.	Contingency	1	LS	$16,380	$6,061	$22,441
Pipe Bracing	Provide bracing for the piping.		6	EA	$11,180	$4,137	$15,317
Valve Bracing	Provide independent bracing for the valves.		3	EA	$5,590	$2,068	$7,658
Anchor Pressure Tank	Provide anchorage of the pressure tank.		1	LS	$1,430	$2,220	$3,650
Pump Bracing	Brace Pump(s) motor(s).	Contingency	3	EA	$12,740	$4,714	$17,454
Further Assesment Necessary	Perform additional analysis to evaluate the adequacy of the existing connection of the motor to the top of the steel motor support, the motor support, the connection of the motor support to the concrete pedestal, and the concrete pedestal.	Study			$0	$4,200	$4,200
Flexible Couplings	Add flexible couplings between the pumps and the connected piping.		1	LS	$14,820	$5,483	$20,303
Backup Battery Restraints	Provide restraint for backup batteries inside the battery cabinet.		1	EA	$520	$192	$712
Further Assessment Necessary	Verify the adequacy of the antenna connection to the supporting pole.	Study	1	EA	$4,160	$1,539	$5,699
Correct Wall Connection	Install a combination of blocking and metal connector hardware to provide adequate connections to transfer seismic forces from the ceiling to the masonry walls and eliminate the potential for cross-grain bending of wood ledgers.	Contingency	1	LS	$23,400	$8,658	$32,058
Contingency Item - Brace Walls	Assume 20 ft of freestanding masonry wall to be braced	Contingency	280	SF Footrpint	$15,340	$5,676	$21,016
Ladder Restraints	Provide restraint for ladders (using straps to the wall, etc.), when not in use.		1	LS	$650	$241	$891
Anchor Electrical Transformer	Coordinate with electrical utility to anchor the utility owned electrical transformer to the concrete pad, as required.		1	LS	$2,080	$3,150	$5,230
	Total Cost				**$173,757**	**$72,561**	**$246,319**

ATTACHMENT A

Mill Creek #1 Reservoir

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Construction Cost	Engineering Cost	Total Construction Cost
ASCE 41 Tier 2 Assessment	Perform a future ASCE 41 Tier 2 assessment to confirm the adequacy of the provided column reinforcing lap splice length. SEFT does	Study		ENG	$0	$8,400	$8,400
Contingency Item - Reinforce Wall		Contingency	8792	SF	$680,290	$251,707	$931,997
Pipe Support Bracing	Install connection brackets at the corners of the pipe support pedestals that are anchored to both the concrete pedestal and concrete floor slab.		1	LS	$7,670	$2,838	$10,508
Seismic Seperation	Provide additional seismic separation between the steel framed stair landing platform and reservoir concrete roof and/or provide diagonal bracing between stair landing support posts.		1	LS	$19,370	$7,167	$26,537
	Total Cost				**$707,330**	**$270,112**	**$977,442**

ATTACHMENT A

Mountain View Pump Station

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Construction Cost	Engineering Cost	Total Construction Cost
Roof Truss Repair	Reconfigure roof trusses and framing/blocking to provide plywood shear wall between roof diaphragm and top plate of north masonry wall. Also, suggest engaging the shear resistance of the interior north-south oriented masonry wall by adding a combination of plywood sheathing, framing/blocking, and metal connector hardware to provide a load path between the roof diaphragm and the interior masonry wall.		372	SF	$15,128	$5,597	$20,725
Correct CMU Wall Support	Install a combination of plywood sheathing, blocking, steel straps, and metal connector hardware to provide adequate out-of-plane support for CMU walls		1,798	SF Footprint	$37,700	$13,949	$51,649
Pipe Bracing	Provide bracing for the piping.		4	EA	$16,120	$5,964	$22,084
Valve Bracing	Provide independent bracing for the valves.		4	EA	$16,120	$5,964	$22,084
Valve Bracing	Provide independent support and bracing for the air relief valves		4	EA	$7,410	$2,742	$10,152
Pump Bracing	Brace Pump(s) motor(s).	Contingency	3	EA	$12,740	$4,714	$17,454
Further Assesment Necessary	Perform additional analysis to evaluate the adequacy of the existing connection of the motor to the top of the steel motor support, the motor support, the connection of the motor support to the concrete pedestal, and the concrete pedestal.	Study			$0	$4,200	$4,200
Flexible Couplings	Add flexible couplings between the pumps and the connected piping		4	EA	$53,170	$19,673	$72,843
Anchor Pipe Support	Add anchors through the pipe support stanchion base plates into the slab-on-grade at locations with missing anchors.		8	EA	$780	$289	$1,069
Anchor Chlornation Equipment	Provide additional anchors for chlorination equipment and repair/replace damaged curb.		1	LS	$1,820	$673	$2,493
Wall Bracing	Provide adequate anchorage between the strut and the masonry shear wall for seismic demands and provide positive connection between spacers and main strut.		1	LS	$1,820	$673	$2,493
Transformer Bracing	Provide bracing for transformer hung from roof.		2	EA	$1,690	$625	$2,315
Further Assessment Necessary	Perform an investigation to evaluate the adequacy of the anchorage of electrical cabinets to the slab on grade, and supplement anchorage (as required). Also, supplement the existing anchorage between the top of the electrical cabinets and masonry wall.	Study	6	LF	$780	$289	$1,069
Electrical Flexible Coupling	Provide flexible couplings where conduits connect to the top of floor- and wall-mounted electrical panels and cabinets.		4	EA	$3,120	$1,154	$4,274
Emergency Generator Bracing	Provide restraint for the emergency generator starter batteries within the battery bins (e.g., strap) and anchorage of the battery bins.		1	LS	$910	$337	$1,247
Emergency Generator Bracing	Provide bracing for the emergency generator muffler		1	LS	$780	$289	$1,069
Lens Cover	Provide lens covers for light fixtures with safety devices that prevent the lens cover and light bulbs from falling		4	EA	$1,300	$481	$1,781
Further Assessment Necessary	Verify the adequacy of the antenna connection to the supporting pole.	Study	1	EA	$4,160	$1,539	$5,699
Contingency Item to seismically strengthen Antenna.		Contingency	1	EA	$4,160	$1,539	$5,699
Trolley Restraints	Provide restraint for overhead trolley chain hoist, when not in use		1	LS	$4,420	$1,635	$6,055
Ladder Restraints	Provide restraint for ladders (using straps to the wall, etc.), when not in use.		1	LS	$650	$241	$891
Electrical Transformer Bracing	Coordinate with electrical utility to anchor the utility owned electrical transformer to the concrete pad, as required.		1	LS	$2,080	$3,150	$5,230
	Total Cost				**$186,858**	**$75,718**	**$262,575**

ATTACHMENT A

Mountain View Reservoir

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Construction Cost	Engineering Cost	Total Construction Cost
Seismic Restraints	Install seismic restraint between the reservoir walls and foundation. Potential concepts include using brackets and high-strength rods installed from inside the reservoir or installing new seismic cables in a thickened wall section from the exterior of the reservoir. Both options would likely require modifying/enlarging the existing foundation ring.		76	CY	$265,574	$98,263	$363,837
	Excavation of Existing Reservoir for Seismic Improvements		3,985	CY	$1,129,653	$417,972	$1,547,625
CHOICE 2	Re-wrap the core wall with additional circumferential prestressing strands encased with shotcrete to provide additional capacity to resist the combination of hydrostatic and expected hydrodynamic hoop forces during an earthquake.		459	CY	$1,076,924	$398,462	$1,475,386
FRP Wrap	Provide fiber reinforced polymer (FRP) wrapping of columns.		1	LS	$143,290	$53,017	$196,308
Contingency Item - Replace Reservoir Piping	Llump Sum to replace interior piping	Contingency	25	lf	$49,321	$18,249	$67,570
	Total Cost				**$2,664,763**	**$985,962**	**$3,650,725**

ATTACHMENT A

Salem/Keizer Intertie #1 Pump Station

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Construction Cost	Engineering Cost	Total Construction Cost
Futher Assessment Necessary	Perform additional analysis to investigate the adequacy of the gap between the City of Salem pump station and the adjacent City of Keizer building.	Study		ENG	$0	$8,400	$8,400
New Shaped Blocking	Install shaped blocking at the ridge line to bridge over the existing gap in the roof sheathing. Provide boundary nailing between the roof sheathing and new blocking. Coordinate with architect for any necessary modifications to roof venting.		572	SF Footprint	$11,180	$4,137	$15,317
Wall Bracing	Install a combination of blocking and steel straps between truss bottom chord members in the two truss bays where blocking is not currently installed to provide continuous cross ties in the east-west direction.		130	LF	$8,580	$3,175	$11,755
Flexible Pipe Joints	Provide appropriate flexible joints where water system piping penetrates through the pump station floor to accommodate potential differential movement between the structure and the surrounding soil at or near the pipe penetration.		3	EA Assumed	$39,910	$14,767	$54,677
Pipe Bracing	Provide bracing for the piping.		3	EA	$12,090	$4,473	$16,563
Valve Bracing	Provide independent bracing for the valves.		3	EA	$12,090	$4,473	$16,563
Fleixble Couplings	Add flexible couplings between the pump and the connected piping.		1	EA Assumed	$14,300	$5,291	$19,591
Chlorination Bracing	Provide anchorage of the chlorination skid to the concrete slab on grade and anchorage of chlorination system components to the skid.		1	EA	$520	$192	$712
Further Assessment Necessary	Perform an investigation to evaluate the adequacy of the anchorage of electrical cabinets to the concrete housekeeping pads, and supplement anchorage (as required).	Study	1	LS	$5,200	$1,924	$7,124
Electrical Cabinet Bracing	Remove the existing L-shaped strut brackets at the top of the electrical cabinets and replace with a more appropriate steel bracket.	Contingency	12	LF	$520	$1,575	$2,095
Further Acessment Necessary	Verify the adequacy of the antenna connection to the supporting pole.	Study	1	EA	$4,160	$1,539	$5,699
Electrical Transformer Bracing	Coordinate with electrical utility to anchor the utility owned electrical transformer to the concrete pad, as required.	Contingency	1	LS	$2,080	$3,150	$5,230
	Total Cost				**$110,630**	**$53,096**	**$163,726**

ATTACHMENT A

Turner Control Facility

Remedial Action	Description	Category of Work (Base, Study, or Contingency)	Quantity	Unit	Construction Cost	Engineering Cost	Total Construction Cost
Geotechnical and Structural Assessment	Perform a site-specific geotechnical study to further evaluate the potential earthquake-induced liquefaction and lateral spreading hazard at the site. If required, mitigate the potential permanent ground deformation hazard at the site with geotechnical improvement, or other appropriate techniques. This scope also includes for a high-level structural evaluation of the geotechnical investigation results.	Study		ENG	$0	$10,500	$10,500
Blocking Support	Add blocking to support the edges of the roof sheathing panels and provided boundary fasteners between the roof sheathing and blocking.	Contingency	1,872	SF	$39,260	$14,526	$53,786
Further Assessment Necessary	Perform an investigation to determine if adequate fasteners are provided for the roof sheathing to blocking and blocking to masonry wall top plate connections. Provide supplemental fasteners, as required	Study	54	EA	$4,550	$8,400	$12,950
Install Fasteners	Install additional fasteners between the roof sheathing and outriggers in conjunction with cross tie retrofit described in next bullet item	Contingency	60	EA	$5,850	$2,165	$8,015
Correct CMU Wall Support	Install a combination of blocking, steel straps, and metal connector hardware to provide adequate out-of-plane support for CMU walls in the direction perpendicular to the roof trusses	Contingency	200	LF	$12,350	$4,570	$16,920
Flexible Pipe Joints	Provide appropriate flexible joints where water system piping penetrates through the control facility wall to accommodate potential differential movement between the structure and the surrounding soil at or near the pipe penetration		1	EA Assumed	$15,860	$5,868	$21,728
Pipe Bracing	Provide bracing for the piping.				$13,130	$4,858	$17,988
Valve Bracing	Provide independent bracing for the valves.		2	EA	$8,840	$3,271	$12,111
Valve Bracing	Provide independent bracing for the valve actuators.		2	EA	$3,380	$1,251	$4,631
Anchor Control Cabinet	Provide anchorage of the control cabinet to the housekeeping pad		1	EA	$780	$289	$1,069
Electrical Transformer Bracing	Provide supplemental anchorage of the electrical transformer to the concrete slab on grade.		1	EA	$650	$241	$891
Backup Battery Restraints	Provide restraint for backup batteries inside the battery cabinet.		1	EA	$520	$192	$712
Lighting Restraints	Provide restraint for pendant supported lights to prevent excessive swing.		1	LS	$1,690	$625	$2,315
Lens Cover	Provide lens covers for light fixtures with safety devices that prevent the lens cover and light bulbs from falling.		8	EA Assumed	$2,600	$962	$3,562
Further Assessment Necessary	Verify the adequacy of the connection between the horizontal antenna and the supporting pole.	Study	1	EA	$4,160	$1,539	$5,699
Contingency Item to seismically strengthen Antenna.		Contingency	1	LS	$8,450	$3,127	$11,577
HVAC Bracing	Provide seismic bracing for the ceiling hung inline HVAC fan.		1	LS	$780	$289	$1,069
HVAC Bracing	Provide anchors between the HVAC condenser unit and concrete support pad.		1	LS	$650	$241	$891
Shelving Bracing	Provide anchorage or bracing for the storage shelving to the floor and/or the wall.		1	LS	$650	$241	$891
Fire Extinguisher Restraints	Provide appropriate restraint for the fire extinguisher in its cabinet.		1	LS	$390	$144	$534
	Total Cost				**$124,540**	**$63,296**	**$187,836**