BANKS SCHOOL DISTRICT

Master Planning Report

Photo Credit: Banks School District Website

BR|IC

BRIC ARCHITECTURE, INC. FEBRUARY 2025

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Executive Summary

This report summarizes the work of BRIC Architecture (and consultants), the Banks School District (stakeholders), and the Banks community in developing conceptual plans for the 2024 bond scope and reimagining the possibilities of the 20-year Master Plan for the Banks high school, middle school and elementary school campuses. The process included a due diligence study of existing conditions of the high school to understand design constraints and opportunities, meetings with stakeholders committee and listening and learning meetings with educational staff and community members.

PROCESS

The design team included BRIC Architecture, 3J Consulting (Civil), PCS (Structural), Sazan Group (Mechanical, Electrical, Plumbing), AKS Engineering (Survey), Lancaster Mobley (Traffic), P&C Construction (Construction Logistics). The team provided due diligence reports in October 2024, a draft area program in October 2024 and a revised draft area program in November 2024, a Listening & Learning Summary in December 2024, and concept designs in December 2024. Each topic is discussed in further detail later in this report.

Photo Credit: Banks School District Website

2. EXECUTIVE SUMMARY

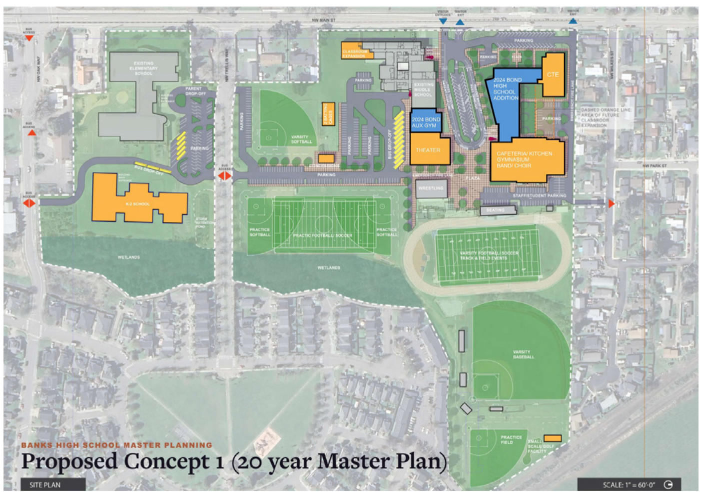
DESIGN CONCEPTS

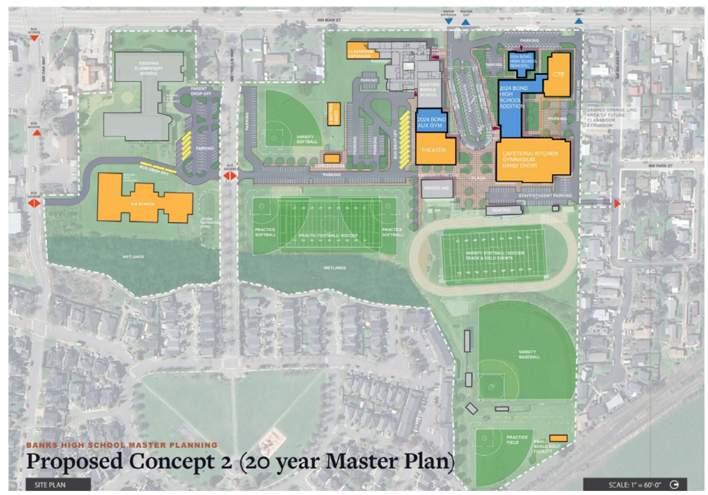
Concept 1

- Provides for a new 43,000 square foot, 2-story high school addition, replacing most of the classrooms.
- The existing gymnasium, cafeteria, CTE and special education classrooms are to remain.
- New Auxiliary gymnasium on the site of the middle school.
- Separation of bus drop off from parent drop-off and pick up area between the middle school and high school.
- Student and staff access from NW Wilkes Street.

Concept 2

- Provides for a new 37,000 square foot, 2-story high school addition and remodel of the existing 10,500 square foot district office for 6,000 square feet of high school program space, replacing most of the classrooms.
- The existing gymnasium, cafeteria, CTE and special education classrooms are to remain.
- New Auxiliary gymnasium on the site of the middle school.
- Separation of bus drop off from parent drop-off and pick up area between the middle school and high school.
- Student and staff access from NW Wilkes Street.





KEY METRICS

- The 20-year Master Plan plans for the following:
    1. Expansion of Banks High School to a minimum capacity of 750 students.
    2. Expansion of Banks Middle School to a capacity of approximately 566 students.
    3. Construction of a new K-2 school for 575 students, while the current Banks Elementary School will continue to serve grades 3-5 in the future.
- Bond Commitments:
    1. Replacement of most of the high school through construction of a new two-story classroom building.
    2. Repair/ replace roofing
    3. Improved heating, cooling, and ventilation
    4. Seismic upgrades
    5. Replacement of some existing buildings with failing safety, mechanical systems, and earthquake vulnerabilities
    6. Physical threat security upgrades at schools, including controlled access, security visibility
    7. General improvements, repairs, and reconfiguration to extend building lifespan and improve traffic flow
    8. Add new auxiliary gym

BUDGET

- The project budget for the bond scope of work for the high school has been developed by Cornerstone Management Group with input from P&C Construction and BRIC Architecture. It is important to note that the document that has been developed is a budget, not a cost estimate. The budget reflects the scope components of the proposed project, but it is too early in the process to conduct a detailed estimate. The first test of the budget will be at the conclusion of the schematic design phase when the first estimate will be developed for the project.
- Budget Assumptions
  - The budget for the project has been based on research regarding current similar projects in the Portland Metro area. Information has been gleaned from various sources including cost estimates and actual construction cost.
  - Budget will continue to be updated over the development of the project.

BANKS HIGH SCHOOL: BUDGET *(As of 11/29/2024)*

		Original	Current
3.1	Building Construction Items	$32,340,000	
3.2	Construction Contingency	$1,750,500	
3.4	GET Construction	$695,000	
3.5	Modular Buildings	$52,500	
4.1	Site Construction Items	$2,455,000	
4.2	Off Site Construction	$114,500	
4.3	Site/Off Site Contingency	$100,000	
4.4	Demolition	$609,350	
5.0	Hazardous Materials	$250,000	
	TOTAL BUDGET	**$38,366,850**	-

BANKS HIGH SCHOOL: SCOPE

	Description	Original	Estimate
	New Addition	$31,450,000	
	District Office	Included	
	HS Admin	Included	
	Library	Included	
	15 Classrooms	Included	
	Title IX Locker Rooms	$1,000,000	
	HVAC Improvements of Existing	$300,000	
	Interior improvements Existing Classrooms / Gym / Commons	$150,000	
	Exterior Improvements	$100,000	
	Site - Parking and Circulation Improvements	$2,570,000	
	Demolition	$609,350	
	Hazardous Materials	$250,000	
	GET Construction	695,000	
	TOTAL	**$37,124,350**	**0**
	Alternates		
A1	CTE Improvements	TBD	
A2	Cafeteria / Performance Improvements	TBD	
A3	Weight Room Improvements	TBD	
A4	BES Parking Lot Repaving	TBD	
Completed or NIC Scope			
1		TBD	
2		TBD	
Notes			
1	Architect / Engineers to coordinate rough-in and locations for Security and Low Voltage Systems		

2. EXECUTIVE SUMMARY

BANKS HIGH SCHOOL: PROJECT SCHEDULE *(As of 11/29/2024)*

	Description	Start	Finish
1	Programming / Pre-Design Phase	12/1/2024	12/31/2024
2	Schematic Design	1/1/2025	3/31/2025
3	Document Review / Approval	4/1/2025	4/15/2025
4	CM/GC Estimate / Budget Reconciliation	4/1/2025	4/15/2025
5	Design Development	4/16/2025	8/31/2025
6	Document Review / Approval	9/1/2025	9/15/2025
7	CM/GC Estimate / Budget Reconciliation	9/1/2025	9/15/2025
8	Start Construction Documents	9/16/2025	3/31/2026
9	50% Construction Documents		12/15/2025
10	50% CD Cost Estimate / Budget Reconciliation		12/15/2025
11	Issue Early Work Package (Demolition, Site Grading)	11/1/2025	
12	Finish Construction Documents		3/31/2026
13	Permit Submittal - Early Work Package (Demo / Grading)	9/1/2025	
14	Permit Submittals - Building / Site	3/1/2026	
15	Bid / Award	TBD	
16	Construction Period	11/1/2025	8/27/2027
17	Substantial Completion		8/27/2027
18	Turn-over		8/30/2027

BANKS AUXILIARY GYM: BUDGET *(As of 11/29/2024)*

		Original	Current
3.1	Building Construction Items	$4,373,562	
3.2	Construction Contingency	$230,188	
3.4	GET Construction	$90,000	
4.1	Site Construction Items	$250,000	
4.2	Off Site Construction	$25,000	
4.3	Site/Off Site Contingency	$226,950	
	TOTAL BUDGET	**$5,195,700**	-

BANKS AUXILIARY GYM: SCOPE

	Description	Original	Estimate
	New Aux Gym	$5,000,000	
	New Facilities Storage Facility	$200,000	
	TOTAL	**$5,200.000**	**0**
	Alternates		
A1	Weight Room	TBD	
Completed or NIC Scope			
1		TBD	
2		TBD	
Notes			
1	Architect / Engineers to coordinate rough-in and locations for Security and Low Voltage Systems		
Notes			

BANKS AUXILIARY GYM: PROJECT SCHEDULE *(As of 11/29/2024)*

	Description	Start	Finish
1	Programming / Pre-Design Phase	12/1/2024	12/31/2024
2	Schematic Design	1/1/2025	2/28/2025
3	Document Review / Approval	3/1/2025	3/15/2025
4	Design Development	3/16/2025	5/15/2025
5	Document Review / Approval	5/16/2025	5/31/2025
6	Start Construction Documents	6/1/2025	3/31/2025
7	50% Construction Documents		6/30/2025
8	Finish Construction Documents		8/31/2025
9	Permit Submittals - Building / Site	9/1/2025	
10	Bid / Award	TBD	
11	Construction Period	12/15/2025	6/15/2026
12	Substantial Completion		6/15/2026
13	Turn-over		6/15/2026

Construction is anticipated to begin for the bond scope in November 2025 for the Auxiliary Gym and January 2026 for the High School and Summer 2026 for upgrades at the Middle School and Elementary School.

Site is big enough for expansion for HS & MS – parking may be an issue, and additional property may need to be purchased in the future.

The following pages include more detailed information on Listening & Learning sessions, Area Program, Design Concepts 1 & 2, and Due Diligence summaries from our consultant team.

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Listening and Learning Summary

In the Fall 2024, Banks School District kicked off the process for planning and designing for the bond promised scope and the 20-year Master Plan for the campuses of the elementary, middle and high schools. The process included eight total meetings with high school staff, community members, the school board and high school students to bring the voices of these groups to the foreground when designing the buildings.

SESSIONS CONDUCTED

- September 18, 2024 – Banks High School Staff Focus Groups #1 (programming)
- September 19, 2024 – Banks High School Staff Focus Groups #2 (programming)
- October 8, 2024 – Banks School District Community Forum #1
- October 15, 2024 – Banks School District Special Education Families
- October 28, 2024 – Banks School District Community Forum #2
- November 12, 2024 – Banks School District Work Session
- November 14, 2024 – Banks School District Multilingual Families
- November 19, 2024 – Banks High School Leadership Class


Photo Credit: Banks School District Facebook Page

LISTENING AND LEARNING: HIGH SCHOOL STAFF FOCUS GROUPS

Key Themes

- Many more facilities needs than can be addressed under the current bond scope. Much of what was discussed will need to be addressed as part of long-term campus master plan.
- Current classrooms are undersized; larger classrooms are desired to accommodate range of class sizes, different teaching/ learning approaches, and flexible room configurations.
- Notable safety and security concerns around the lack of a single main entry or secure vestibule and porous building that is difficult to supervise. Strong need for main office to be placed adjacent to main entry.
- Locker rooms are very outdated with major ventilation issues and significant Title IX concerns. Only one team room is presents (boy's side).
- Need for more purposely designed space to support STEAM and CTE activities.
- Lack of spaces for students to congregate informally.
- Outdated restrooms with solid doors difficult to monitor and not ADA accessible.
- Life skills classroom is in a small general classroom without ADA restroom, sensory room or sufficient space to support different activity zones or the program's student-run coffee business.
- District office needs center mostly around providing sufficient offices/ workstations and a conference room. Dedicated space for School Board Meetings is not necessary (can be held in new BHS Library).

COMMUNITY FORUM MEETINGS

Key Themes

- Safety and Security and a healthy, modern school were the top priorities.
- Safety particularly at the elementary school for fencing and entrance.
- Buildings that hold up over time and are functional.
- Fixing the traffic flow and parking issues at the middle school/ high school.
- Finishing on time and on budget stretching taxpayer dollars as far as possible; budget should be considered over aesthetics.
- Creating a campus and buildings that the students and town can be proud of.
- Ensure there is community and staff involvement in decision making; Keep the community informed and real engagement with the community.
- Facilities that offer the best settings for learning and growth.

Photo Credit: Banks School District Facebook Page

SPECIAL EDUCATION FAMILIES

Key Themes

- A new building that is healthy, safe and updated for all students.
- A school building that represents the needs that students, staff and community members have expressed.
- Parents and specialists like PT, OT, Speech pathology and vision have an opportunity to weigh in on design needs to support all students (community involvement).
- Design that reflects that all students are valued . . . those with special needs, in the arts, etc.
- More single occupant restrooms.

Photo Credit: Banks School District Facebook Page

SCHOOL BOARD WORK SESSION

Key Themes

- Appreciate the development of two master plan options.
- Questions about open areas and the use of space.
- Questions about whether CTE and SPED classrooms could be relocated to the old District Office Building or old Art room.
- Question about ROM pricing for two scenarios.
- Would like square footages and other details to be accurate to spaces.
- Conflicting viewpoints on how to utilize the district office if it is kept (renovate or preserve).

MULTILINGUAL FAMILIES

Key Themes

- The biggest challenges faced are language, their inability to assist their students with their homework, bullying of their children, concern about clearer paths defined for their children beyond high school and the financial ability to support them.
- Magali Benson (Student & Family Engagement Manager) has been an invaluable resource to these families and they are grateful to have her to work with.
- The would like to see improvements to campus fields and sports facilities, spaces for electives, and larger cafeteria.
- They have concerns about the safety of pick up and drop off at the middle school/ high school site.
- Exposure to opportunities for college, CTE, and other options need to be available earlier and it would be great if families could have resources in Spanish to understand these opportunities.

HIGH SCHOOL LEADERSHIP CLASS

Key Themes

- Love the open design with places for students to gather and sit (flex areas, open stairs).
- Excited about separating buses, parent drop off/ pick up, and students.
- Like the back entrance/ exit off Wilkes for students.
- Excited about remodeled locker rooms.
- Like more parking for students and everyone.
- Do not want classes out in the old district office, if the building is kept.

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Phase I: Partial Replacement of Banks High School

The development of the area program for Banks High School's replacement facility commenced in fall 2024. The main focus was development of an area program for the replacement of a large portion of Banks High School's current facility as part of the 2024 bond. Additionally, a rough area program for potential school additions over the next 20 years was completed for reference purposes when developing the long-term campus master plan.

The area program for partial replacement of Banks High School was developed based on input from teachers, staff, and administrators. Programmatic focus group meetings were held with each department on September 18-19, 2024. Participants were provided with a list of questions in advance of their session. Discussions were centered on the high school expansion project; however, some questions were asked to ascertain thoughts on the long-term development of the campus as well (for the 20-year master plan). Focus group areas included:

- General Education Teachers
- Special Education Teachers and Staff
- CTE Teachers
- Science Teachers
- Visual Arts Teachers
- Music / Performing Arts Teachers
- PE / Athletics Teachers and Coaches
- School Administrative Staff
- District Administrative Staff

Major themes from the programmatic focus group sessions are summarized below.

- Facilities needs at Banks High School greatly outweigh what can be addressed under the current bond scope. Many of the facilities needs stated will need to be addressed as part of long-term campus master plan due to funding constraints.
- Current classrooms at Banks High School are undersized; larger classrooms are desired to accommodate range of class sizes, different teaching/learning approaches, and flexible room configurations.
- There are notable safety and security concerns around the lack of a single, secure main entry and a porous building that is difficult to supervise. There is a strong need for the school's main office to be placed adjacent to the main entry (connected via a secure entry vestibule).
- The current locker rooms are very outdated with major ventilation issues and significant Title IX concerns. Only one team room is present (on the boys' side).
- There is a strong need for more purposely designed spaces to support STEAM and CTE activities.
- The current BHS facility lacks spaces for students to congregate informally.
- The current facility has outdated restrooms with solid doors that difficult to monitor and not ADA accessible. The only ADA or gender-inclusive restroom is located within the old district office building.
- The current life skills classroom is in a small general classroom without an ADA restroom, sensory room, or sufficient space to support different activity zones or the program's student-run coffee business.
- District office needs center mostly around providing sufficient offices / workstations and a conference room. A dedicated space for School Board Meetings is not necessary (meetings can be held in BHS Library).

4. AREA PROGRAM

At the onset of the project, it was determined that the scope of the work at BHS would be limited to replacing the two main academic wings with new construction. The capacity of the building would remained unchanged, as a “one-for-one” replacement of teaching stations would occur. Teaching stations and other major areas to be replaced as part of the project include:

- Room 3 - former small classroom currently used as AD office
- Room 4 – Life Skills classroom
- Room 5 – General classroom
- Room 6 – General classroom
- Room 7 – General classroom
- Room 9 – General classroom
- Room 10 – Marketing / Graphic Design classroom
- Room 18 – Visual Arts classroom
- Room 19 – General classroom
- Room 20 – Science lab
- Room 22 – General classroom
- Room 23 – General classroom
- Room 24 – General classroom
- Room 25 – General classroom
- Room 26 – Science lab
- Room 27 – General classroom used for CTE
- Library Media Center
- Main office, administration, and counseling areas

The following instructional areas will remain and will not be replaced as part of the project scope:

- Room 30/28 – CTE shop and instructional area
- Room 17 – former Band classroom currently used for Health instruction and offices
- Room 16 – Music room
- Room 12 (with necessary repairs and abatement)
- Room 31 - SPED resource room
- Gymnasium (PE teaching station)

The standalone auxiliary gymnasium will also be replaced as part of the bond scope. A preliminary area program was developed based on the stated needs of building users; however, it was soon determined that the bond would likely only be able to fund an aux gym of 7,400 GSF. The program listed below represents a larger version that could potentially be provided should additional funding become available.

A draft area program is also provided for the renovation of the existing locker rooms at Banks High School; however, this program requires review and refinement during the schematic design process.

BANKS HIGH SCHOOL - PROPOSED AREA PROGRAM - PHASE 1

	# of Teaching Stations	Quantity	Net SF per Space	Total Net SF	Subtotal by Area
Academics					
General Classrooms *Note: Based on replacing what will be demolished. This would assume that we are keeping: 1) Room 30/28 (CTE); 2) Room 17; 3) Room 16; and 4) Room 12 (with necessary repairs and abatement). Room 31 will also remain, but as it is a SPED resource room, it is not counted as a teaching station currently to allow flexible use for pull-out services and activities.*	11	11	900	9,900	
Science Labs (existing labs would be part of demolition)	2	2	1,300	2,600	
Science Prep Room		1	200	200	
Chemical Storage / Science Storage		1	100	100	
Graphic Design Classroom	1	1	1,400	1,400	
Graphic Design Storage		1	100	100	
Extended Learning Areas		2	300	600	
					14,900
Specialized Programs					
Life Skills Classroom (not counted as teaching station for capacity purposes)		1	950	950	
ADA Accessible Restroom with Changing Table		1	100	100	
					1,050
2-D and 3-D Art					
2-D / 3-D Art Classroom (existing room would be part of demolition)	1	1	1,800	1,800	
Kiln Room		1	150	150	
Art Supply / Storage Room		1	200	200	
Project Storage		1	100	100	
					2,250
Library Media Center					
Library Media Center (existing space would be part of demolition)		1	2,250	2,250	
Office / Workroom		1	150	150	
Enclosed Individual Study Rooms		2	50	100	
					2,500
School Administration					
Entry / Reception / Lobby / Waiting Area		1	400	400	
Open Secretarial Area - Two Workstations (Principal, Secretary & Attendance)		1	200	200	
Principal's Office		1	200	200	
Assistant Principal's Office		1	120	120	
AD Office		1	120	120	
AD Secretary Office		1	80	80	
AD Storage		1	50	50	
Itinerant Staff Offices / Flex Mtg Room / Zoom Room / Testing Room		2	80	160	
Conference Room		1	300	300	
Health Room (one cot)		1	100	100	
ADA Accessible Restroom (single use) Next to Health Room		1	60	60	
Workroom / Copy / Kitchenette		1	300	300	
Supply Storage		1	100	100	
Records Storage (secure, fire-resistant room)		1	150	150	
General Office Storage		1	100	100	
Lactation Room		1	65	65	
					2,505

BANKS HIGH SCHOOL - PROPOSED AREA PROGRAM - PHASE 1 (CONTINUED)

	# of Teaching Stations	Quantity	Net SF per Space	Total Net SF	Subtotal by Area
Counseling					
Counseling Waiting Area (no reception desk)		1	100	100	
Counselor Offices		2	120	240	
Counseling Storage		1	50	50	
					390
Community and Special Use					
Food Pantry / Clothing Closet		1	200	200	
					200
District Office					
Entry / Reception / Lobby / Waiting Area		1	200	200	
Open Secretarial Area - One Workstation		1	100	100	
Office Manager		1	80	80	
Superintendent's Office		1	220	220	
Director of Student Performance Office		1	120	120	
Director of Student Secretary Office		1	80	80	
Sodexo Office (required per contract)		1	80	80	
Itinerant Staff Offices / Flex Meeting Room / Zoom Room		2	80	160	
District Board Room (Note: excluded - assume will be held in library)		0	1,500	-	
IT Office / Network Repair and Storage		1	500	500	
Conference Room		1	300	300	
Copier / Kitchenette (can be an alcove)		0	150	-	
General Office Storage		1	100	100	
					1,940
Building Support - NOTE: ESTIMATES ONLY S - FINAL ALLOCATIONS WILL HEAVILY DEPEND UPON FINAL DESIGN					
Distributed Custodial Closets		3	80	240	
Sodexo Custodial Room (required per contract)		1	100	100	
Distributed Student Restrooms		6	250	1,500	
Distributed Staff Restrooms		3	66	198	
ADA Accessible Single-Use Restrooms (gender-neutral)		2	66	132	
General Building Storage		1	500	500	
Custodial Supplies Center Storage		1	500	500	
Flammable Storage		1	100	100	
Elevator (assume 1) / Equipment		1	200	200	
Receiving Area		1	200	200	
MEP Room		1	2,000	2,000	
MDF Room		1	250	250	
Distributed IDF Room		2	50	100	
					6,020
Total Net SF	**15**			**31,755**	
Grossing Factor (35%)				11,114	
Total Gross SF				**42,869**	
Current # of Teaching Stations (*includes main gym as PE teaching station*)	20				
Teaching Stations to Remain Post-Demolition (*4 classrooms + gym*)	5				
Teaching Stations to be Part of New Construction (General Classrooms, Science Labs, Art Room, Life Skills)	15				
Final Teaching Station Count (includes main gym as PE teaching station)	**20**				
Student Capacity (pre and post-construction)	**450**				

BANKS HIGH SCHOOL - PROPOSED AREA PROGRAM - AUX GYM

	Quantity	Net SF per Space	Total Net SF	Subtotal by Area
Aux Gym				
Aux Gym: 1 court / 3 rows bleachers on one side	1	6,700	6,700	
Men's / Women's Restrooms	2	350	700	
Single-use Gender-Neutral / Family Restroom	1	80	80	
Team Rooms (non-gendered)	2	800	1,600	
PE / Athletics / Dance Storage	1	400	400	
Robotics Storage	1	400	400	
Other Storage (for clubs, etc.)	1	200	200	
				10,080
Total Net SF			**10,080**	
Grossing Factor (35%)			3,528	
Total Gross SF			**13,608**	

BANKS HIGH SCHOOL - PROPOSED AREA PROGRAM - LOCKER ROOM RENOVATIONS (5,100SF)

	Quantity	Net SF per Space	Total Net SF	Subtotal by Area
Girls' Locker Room				
Lockers / Benches	1	750	750	
Private Changing Stalls / Showers / Toilets	1	250	250	
PE Office	1	100	100	
Storage	1	50	50	
Boys' Locker Room				
Lockers / Benches	1	750	750	
Private Changing Stalls / Showers / Toilets	1	250	250	
PE Office	1	100	100	
Storage	1	50	50	
Gender-Neutral Locker Room				
Private Changing Stalls / Showers / Toilets	1	80	80	
Team Rooms				
Small Team Rooms (non-gendered)	2	500	1,000	
Restrooms				
Men's / Women's Restrooms	2	250	500	
Other				
Laundry	1	100	100	
Coaches / Officials Locker Room	1	80	80	
				4,060
Total Net SF			**4,060**	
Grossing Factor (25%)			1,015	
Total Gross SF			**5,075**	

Long-term Campus Master Plan

A rough set of area programs was developed to inform a long-term campus master plan showing potential building expansions and additions over the next 20 years to accommodate projected enrollment growth.

The plan shows facilities required to serve approximately 2,500 students campus wide including:

- Expansion of Banks High School to a minimum capacity of 750 students, including new CTE and performing arts facilities.
- Expansion of Banks Middle School to a capacity of approximately 566 students.
- Construction of a new K-2 school for 575 students, with the assumption that the current Banks Elementary School would serve grades 3-5 in the future. This would nearly double the current elementary school capacity.

It is important to note that the area programs for the campus master plan were developed internally for general planning purposes only.

BANKS HIGH SCHOOL - PROPOSED AREA PROGRAM FOR FUTURE ADDITION (Final Capacity = 750 Students)

	# of Teaching Stations	Quantity	Net SF per Space	Total Net SF	Subtotal by Area
Academics					
General Classrooms	7	7	950	6,650	
Extended Learning Areas / Flex Space		1	500	500	
Science Labs	2	2	1,400	2,800	
Science Prep Room		1	300	300	
General Science Storage		1	150	150	
CTE Shops + Associated Storage (Type TBD)	2	2	3,000	6,000	
STEAM / Robotics Lab	1	1	2,000	2,000	
					18,400
Specialized Programs (Special Education)					
Sensory Room		1	300	300	
Apartment / Kitchen Learning Area for Life Skills		1	700	700	
					1,000
Music (Band and Choir)					
Band Room	1	1	2,000	2,000	
Instrument Storage		1	300	300	
Ensemble Room		1	400	400	
Practice Rooms		2	75	150	
Band Office		1	120	120	
					2,970
Theater / Performing Arts					
Auditorium / Theater (approx. 550 seating capacity)		1	6,500	6,500	
Stage		1	2,750	2,750	
Control Booth		1	200	200	
Theater Storage		1	200	200	
Lighting Storage		1	100	100	
Costume Storage		1	200	200	
Make-up / Dressing Rooms		1	200	200	
Green Room		1	200	200	
Drama Instructor's Office		1	100	100	
					10,450
Athletics / P.E.					
Aux Gym	1	1	6,700	6,700	
Wrestling Room		1	3,000	3,000	
Training Room		1	500	500	
Equipment Storage		1	500	500	
Large Team Rooms (gender neutral)		1	700	700	
Small Team Rooms (gender neutral)		2	300	600	
					12,000
Cafeteria / Commons					
Cafeteria (1 lunch period w/ 25% leaving for open campus - ref. of 750 cap.)		1	8,500	8,500	
					8,500

BANKS HIGH SCHOOL - PROPOSED AREA PROGRAM FOR FUTURE ADDITION (CONTINUED)

	# of Teaching Stations	Quantity	Net SF per Space	Total Net SF	Subtotal by Area
Kitchen / Nutrition Services					
Main Servery		1	1,000	1,000	
Prep and Cooking Areas (Kitchen)		1	2,000	2,000	
Dry Storage		1	400	400	
Walk-in Cooler and Freezer		1	800	800	
Ware-washing		1	300	300	
Allocation for Kitchen Office / Staff Lockers		1	200	200	
Receiving Area		1	200	200	
					4,900
Administration					
Additional Assistant Principal's Office		1	120	120	
Additional Counselor's Office		1	120	120	
Additional Staff Offices or Workstations		1	100	100	
					340
Custodial and Maintenance - NOTE: THESE WILL BE ADJUSTED BASED ON FINAL SIZE AND LAYOUT OF BUILDING					
Distributed Custodial Closets		2	100	200	
Distributed Student Restrooms		6	275	1,650	
Distributed Staff Restrooms		2	66	132	
ADA Accessible Restrooms (single use)		2	60	120	
MDF Room		1	250	250	
Distributed IDF Rooms		4	50	200	
					2,552
Miscellaneous - NOTE: THESE WILL BE ADJUSTED BASED ON FINAL SIZE AND LAYOUT OF BUILDING					
Sub Electrical Room		2	75	150	
Elevator		1	120	120	
Elevator Equipment		1	80	80	
					350
Total Net SF				**61,462**	
Grossing Factor (35%)				21,512	
Total Gross SF				**82,974**	

BANKS HIGH SCHOOL - PROPOSED AREA PROGRAM FOR FUTURE ADDITION (Final Capacity = 566 Students)

	# of Teaching Stations	Quantity	Net SF per Space	Total Net SF	Subtotal by Area
Academics					
General Classrooms	6	6	950	5,700	
Extended Learning Areas / Flex Space		1	500	500	
Science Labs	1	1	1,400	1,400	
Science Prep / Storage Room		1	300	300	
STEAM / Robotics Lab	1	1	2,000	2,000	
					9,900
Building Operations / Maintenance / Restrooms					
General Allowance		1	2,000	2,000	
					2,000
Total Net SF				**11,900**	**11,900**
Grossing Factor (35%)				4,165	
Total Gross SF				**16,065**	

Final Capacity of 566 Students.

Assume they will use existing cafeteria, but will move to 2 lunch periods.

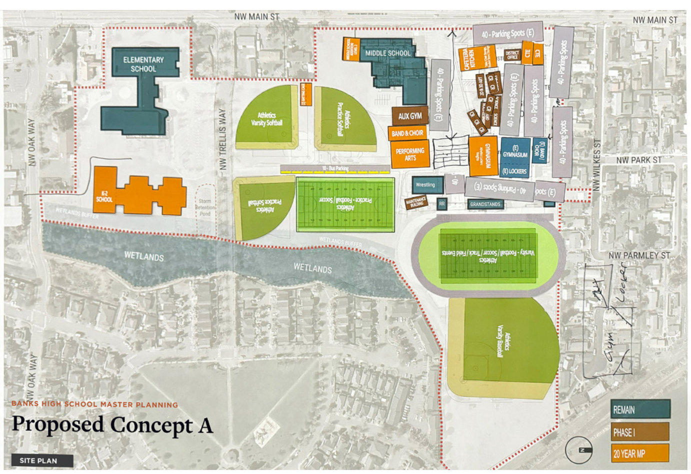
BANKS K-2 SCHOOL - PROPOSED AREA PROGRAM (Capacity = 575 Students)

	# of Teaching Stations	Quantity	Net SF per Space	Total Net SF
Academics				
General K-2 Classrooms	23	23	950	21,850
Shared Learning (one per grade)		3	700	2,100
Music Room		1	1,200	1,200
Special Education Classroom		1	1,200	1,200
SPED ADA Restroom w/ changing table		1	120	120
Sensory Room		1	300	300
P.E.				
Multipurpose Fitness Room		1	3,000	3,000
Office		1	100	100
Equipment Storage		1	300	300
Library Media Center				
Library / Circulation Desk		1	1,500	1,500
Workroom		1	100	100
Technology and Storage		1	100	100
Cafeteria				
Cafeteria (assume 3 lunch periods - 1 per grade)		1	2,500	2,500
Warming Kitchen		1	500	500
Walk-in Freezer / Cooler		1	100	100
Ware-washing		1	100	100
Kitchen Staff Toilet		1	60	60
Kitchen Office		1	100	100
Dry Storage		1	150	150
Administration				
Lobby / Reception (includes 2 workstations)		1	500	500
Secure Vestibule		1	400	400
Principal Office		1	150	150
Counselor's Office		1	120	120
Itinerant Offices		3	100	300
Conference Room		1	220	220
Workroom		1	350	350
Health Room		1	200	200
File Storage		1	80	80
Building Operations / Maintenance / Restrooms				
Restrooms		Dist.	1,500	1,500
General Storage		2	200	400
Custodial Closets		2	50	100
Mechanical Room		2	200	400
Electrical Room		1	200	200
MDF		1	100	100
IDF		2	60	120
Total Net SF				**40,520**
Student Capacity	575			
Net to Gross Ratio				**1.35**
Total Gross Building Square Footage				**54,702**
Covered Play Area		1	3,000	**3,000**

Design Concepts

The reimaging of Banks High School requires an awareness of history, context, and a vision for the future. The thread that will tie all this together is a design process with both a window to the past and a view to the future.

The proposed high school classroom addition will be a transformative project designed to enhance educational experiences for students in the Banks School District. The proposed new two-story addition aims to modernize the learning environment while integrating with essential existing facilities. Situated adjacent to the existing school infrastructure, the site not only accommodates the new addition but also respects and integrates with the surrounding constraints. The existing gymnasium, cafeteria, Career and Technical Education (CTE) facilities, and special education classrooms will remain, providing continuity in the school's educational offerings. The design carefully considers the movement patterns of students and staff to enhance safety, security, and accessibility. The architectural vision for this addition emphasizes both functionality and community integration. By replacing most of the outdated classrooms, the new addition will foster collaborative learning. The strategic decision to include an auxiliary gymnasium on the site of the middle school promotes a seamless connection between physical education and recreational activities for both middle and high school students. The addition incorporates a thoughtful layout that promotes interaction and accessibility. The separation of bus drop-off areas from parent drop-off zones between the middle school and high school minimizes congestion and enhances safety. Student and staff access from NW Wilkes Street ensures a clear and defined entry point, facilitating a smooth transition from arrival to the classroom. The new high school addition represents a significant investment in the future of education in the Banks School District. By enhancing the learning environment and integrating safety features through thoughtful design, this project aspires to foster academic success and support all students and staff. The development will build a foundation for future educational excellence.



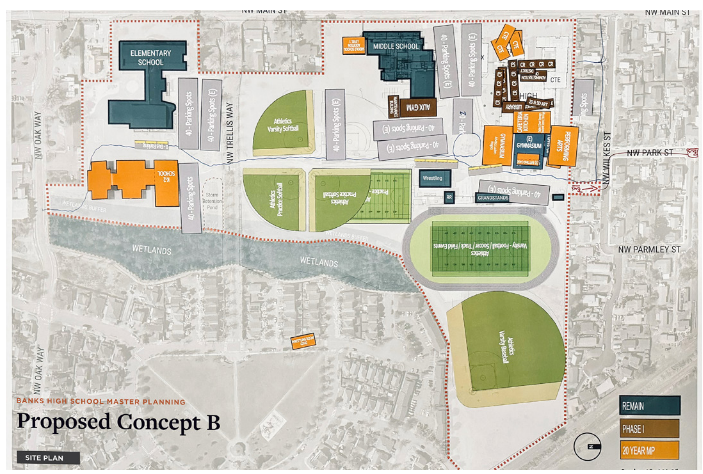
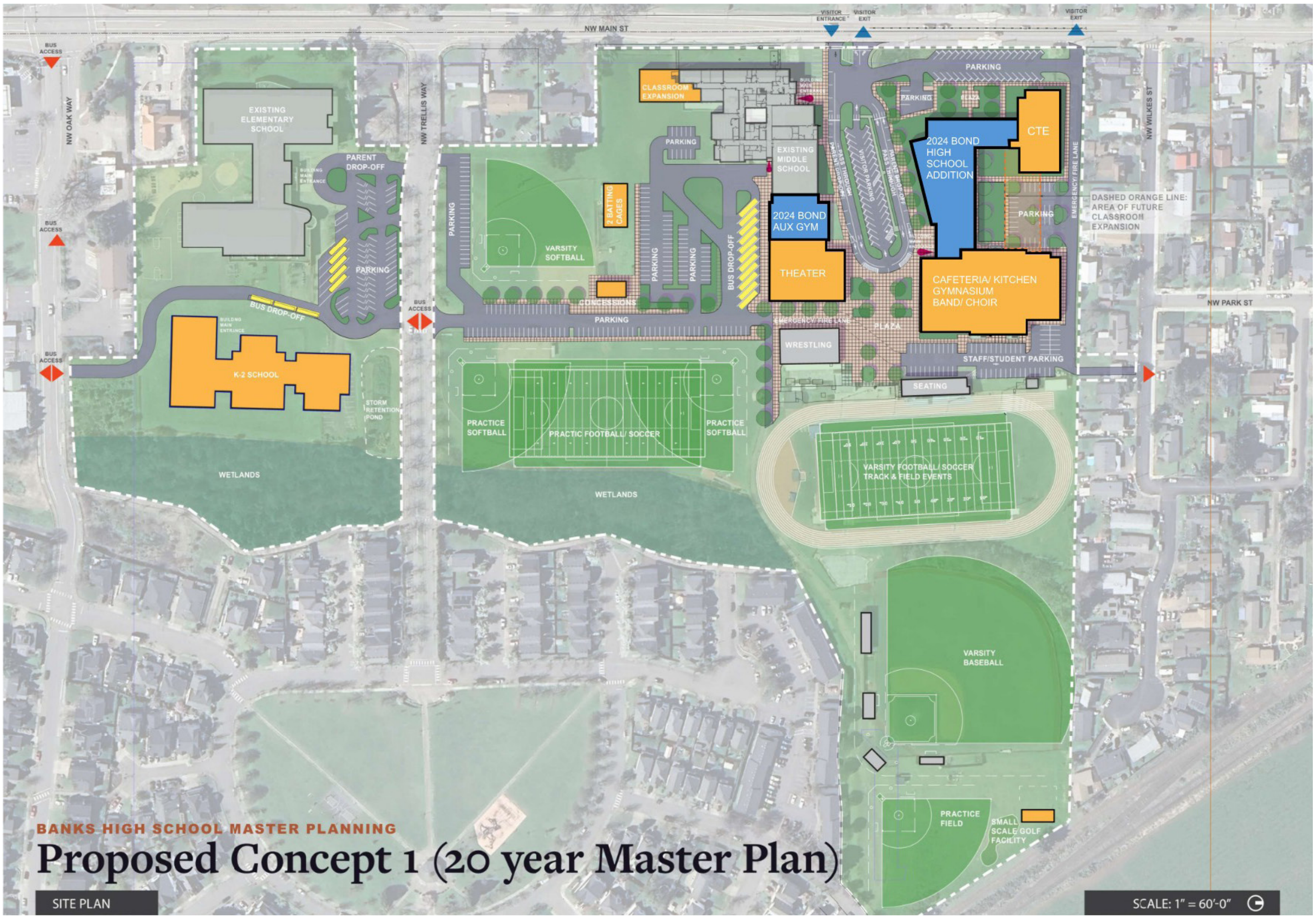



Table A


Table B

Concept 1: 20-Year Master Plan



Concept 1: 20-Year Master Plan Design Considerations

SITE DEVELOPMENT

→ **Enhance Campus-Wide Circulation and Traffic Flow:**

- Create separate bus access and designated parking for a total of 11 buses.
- Establish distinct bus drop-off and pick-up zones for improved safety and efficiency.
- Designate visitor parking areas to facilitate access for guests.
- Maximize student parking to accommodate current and future needs.
- Optimize staff parking to ensure convenient access for teachers and staff.
- Ensure clear pathways for fire, emergency, and service access.
- Utilize Wilkes, Oak Way, NW Trellis Way, and NW Main Street as entry points to the campus.
- Identify site entry access points to improve traffic management.
- Design site circulation that separates student pathways from vehicle traffic to enhance safety.

→ **Optimize Athletic Fields:**

- Develop practice softball fields for all use.
- Construct a varsity softball field for competitive events.
- Establish multi-purpose fields that can be used for football and soccer practices.
- Install batting cages to support student athletes.
- Incorporate concessions to serve spectators during events.

BUILDING IMPROVEMENTS

→ **Replace the Auxiliary Gym** and identify an appropriate new location for enhanced facilities.

→ **Assess the Current Administration Building** to determine potential new design.

→ **Develop a new 43,000 sf 2-Story Building** to replace majority of existing learning spaces, optimizing learning spaces, layout, and access.

→ **Modernize Locker Rooms** to meet Title IX requirements, ensuring equitable facilities for all students.

→ **Enhance Safety and Security** through access control measures and perimeter hardening.

→ **Ensure ADA Compliance** across all buildings and throughout the campus for inclusive access.

→ **Plan for a Future Theater** that supports performing arts and events.

→ **Design a Future Cafeteria** to meet the growing needs of the student population.

→ **Incorporate a Future Career and Technology Education (CTE) Facility** to support vocational training.

→ **Plan for Future Classroom Additions** to accommodate enrollment growth.

→ **Minimize Construction Disruption,** aiming to limit the use of temporary portables with the goal of utilizing no portables.

ADDITIONAL STRATEGIC GOALS

→ **Maximize Site Capacity** to effectively accommodate projected increases in student population.

→ **Ensure Fiscal Responsibility** in planning to maximize the value of taxpayer dollars, focusing on creating safe, functional, and durable facilities.

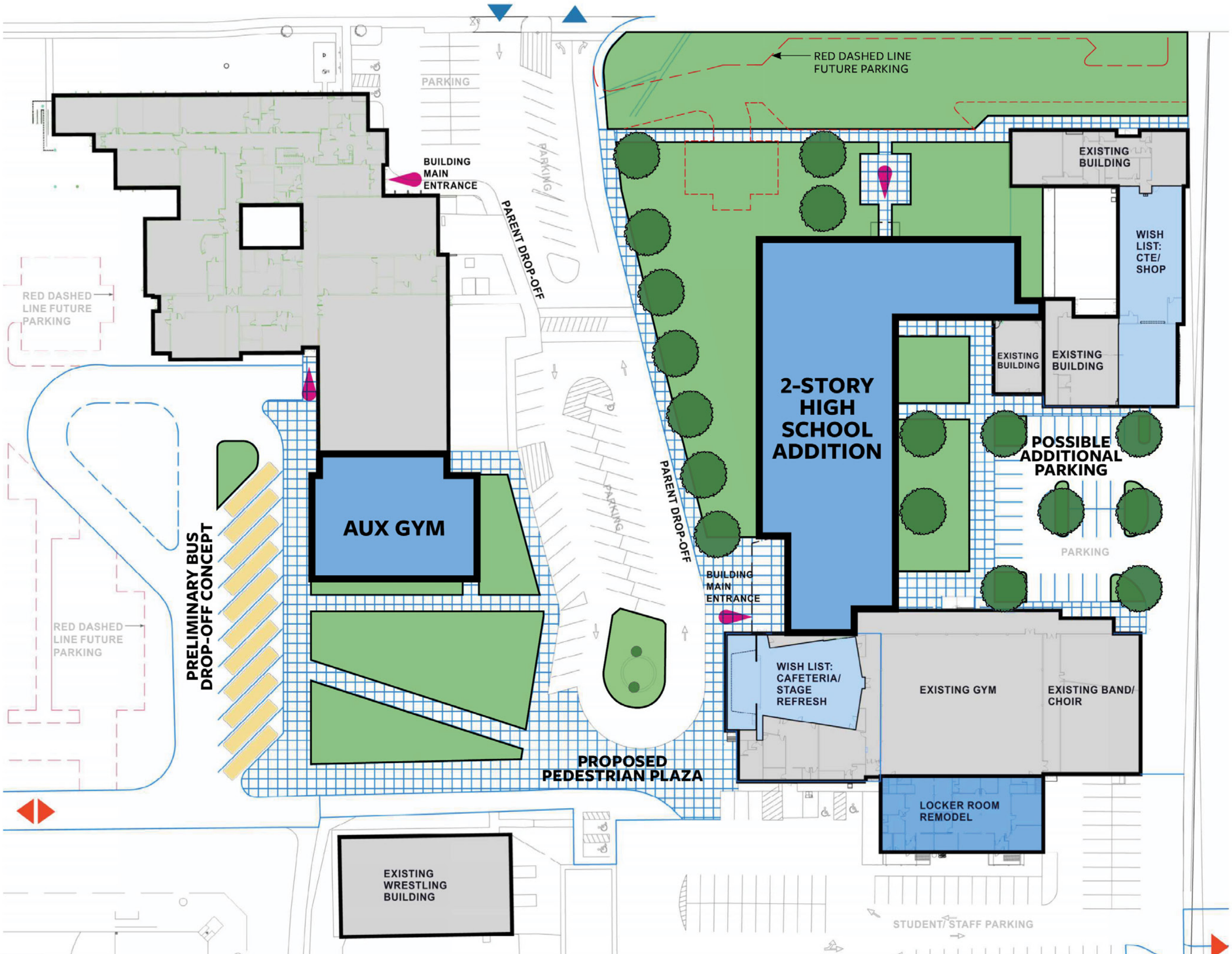
- Strive to complete projects on time, within budget, and in alignment with bond commitments.

→ **Prioritize Campus Safety and Security** by implementing Crime Prevention Through Environmental Design (CPTED) principles.

→ **Prepare for a K-2 Building** on-site to respond to future increases in elementary enrollment.

→ **Plan for Future Classroom Expansion** at the Middle School level to meet evolving educational demands.

Concept 1: 2024 Bond Program



SITE PLAN

Concept 1: 2024 Bond Program

SITE DESIGN REQUIREMENTS

→ **Enhance Campus-Wide Circulation and Traffic Flow:**

- Create separate bus access and designated parking for a total of 11 buses.
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- Design circulation that separates student pathways from vehicle traffic to enhance safety.

BUILDING DESIGN REQUIREMENTS

→ **Construct a New 43,000 sf Two-Story Building:** This facility will replace the majority of existing learning spaces, optimizing layouts and enhancing accessibility for students.

→ **Modernize Locker Rooms:** Upgrade facilities to meet Title IX requirements, ensuring equitable access for all students.

→ **Replace the Auxiliary Gym:** Identify an appropriate new location for enhanced athletic facilities.

→ **Focus on Safety and Security:**

- Ensure ADA compliance across all buildings and throughout the campus for inclusive access.
- Plan for future classroom additions to accommodate enrollment growth.
- Minimize construction disruption, aiming to limit the use of temporary portables, ideally utilizing none.

WISH LIST CONSIDERATIONS

→ **CTE Shop Upgrades:** Update vocational training spaces to better serve students.

→ **High School Cafeteria/Stage Refresh:** Modernize dining and performance areas.

→ **High School Weight Room Upgrades:** Enhance fitness facilities for student use.

→ **ES Parking Lot Paving:** Improve parking facilities for accessibility and safety.

→ **ES Boiler Replacement:** Upgrade heating systems for efficiency.

→ **Updated Security Cameras:** Install modern surveillance across all schools.

→ **Weapon Detection Systems:** Enhance school safety protocols.

→ **Portable Panic Buttons:** Equip staff with safety tools for emergencies.

→ **Modern Safety Software:** Implement up-to-date systems for managing campus safety.

→ **Maintenance Fund:** Create a fund for ongoing maintenance of future projects.

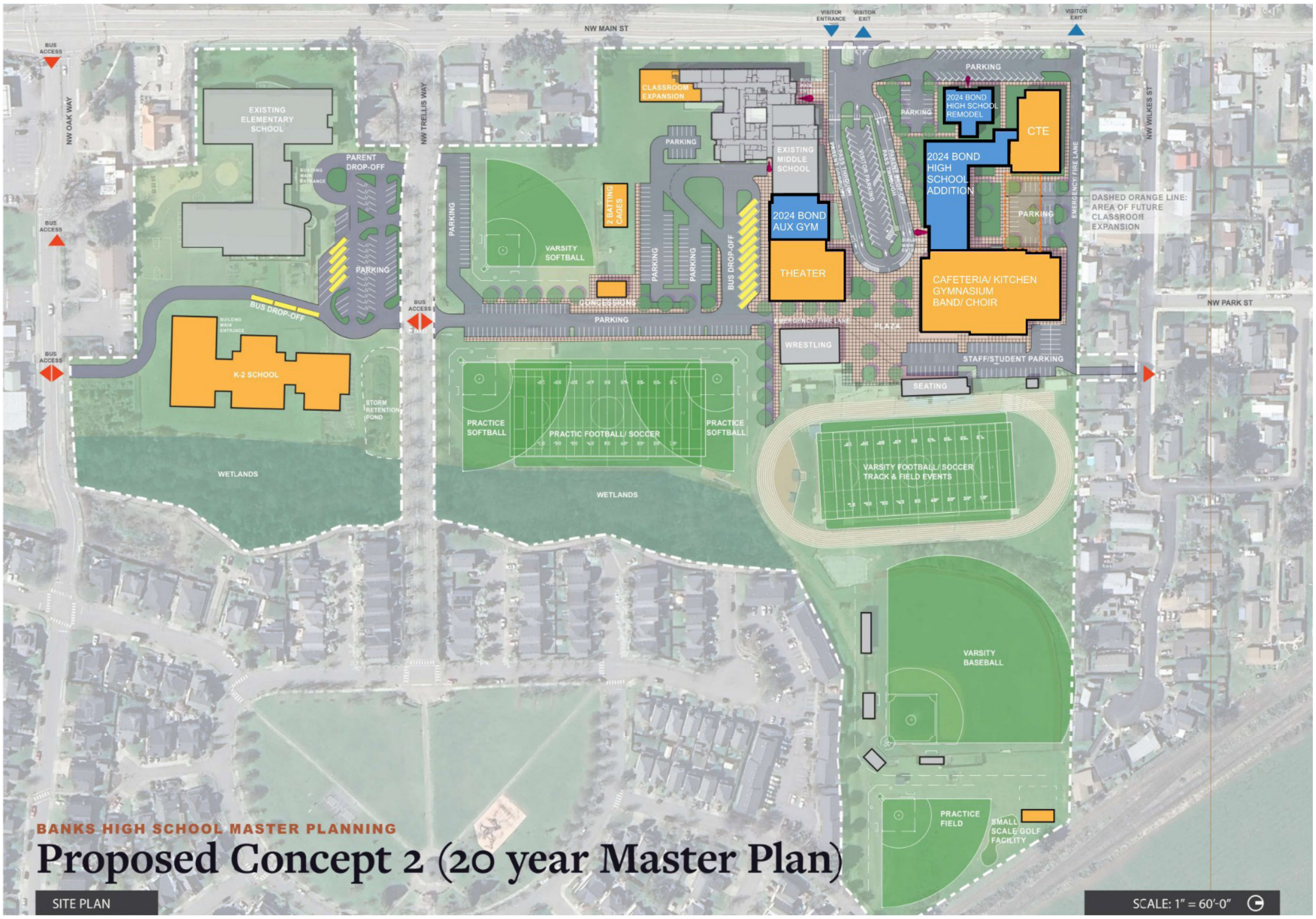
NEW ADDITION

43,000 SF OF PROGRAM IN NEW ADDITION

43,000 SF of Program x $740/SF = $31,820,000

TOTAL CONSTRUCTION BUDGET FOR CONCEPT 1 = $31,820,000

Concept 2: 20-Year Master Plan



Concept 2: 20-Year Master Plan Design Considerations

SITE DEVELOPMENT

→ **Enhance Campus-Wide Circulation and Traffic Flow:**

- Create separate bus access and designated parking for a total of 11 buses.
- Establish distinct bus drop-off and pick-up zones for improved safety and efficiency.
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→ **Optimize Athletic Fields:**

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- Establish multi-purpose fields that can be used for football and soccer practices.
- Install batting cages to support student athletes.
- Incorporate concessions to serve spectators during events.

BUILDING IMPROVEMENTS

→ **Replace the Auxiliary Gym** and identify an appropriate new location for enhanced facilities.

→ **Assess the Current Administration Building** to determine potential new design.

→ **Develop a new 37,000 sf 2-Story Building** to replace majority of existing learning spaces, optimizing learning spaces, layout and access.

→ **Reimagine the existing 10,500 sf District Administration Building:** Alongside the new building, the redesigned building will prioritize student services and access, creating a more welcoming environment for all students.

→ **Modernize Locker Rooms** to meet Title IX requirements, ensuring equitable facilities for all students.

→ **Enhance Safety and Security** through access control measures and perimeter hardening.

→ **Ensure ADA Compliance** across all buildings and throughout the campus for inclusive access.

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ADDITIONAL STRATEGIC GOALS

→ **Maximize Site Capacity** to effectively accommodate projected increases in student population.

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→ **Prepare for a K-2 Building** on-site to respond to future increases in elementary enrollment.

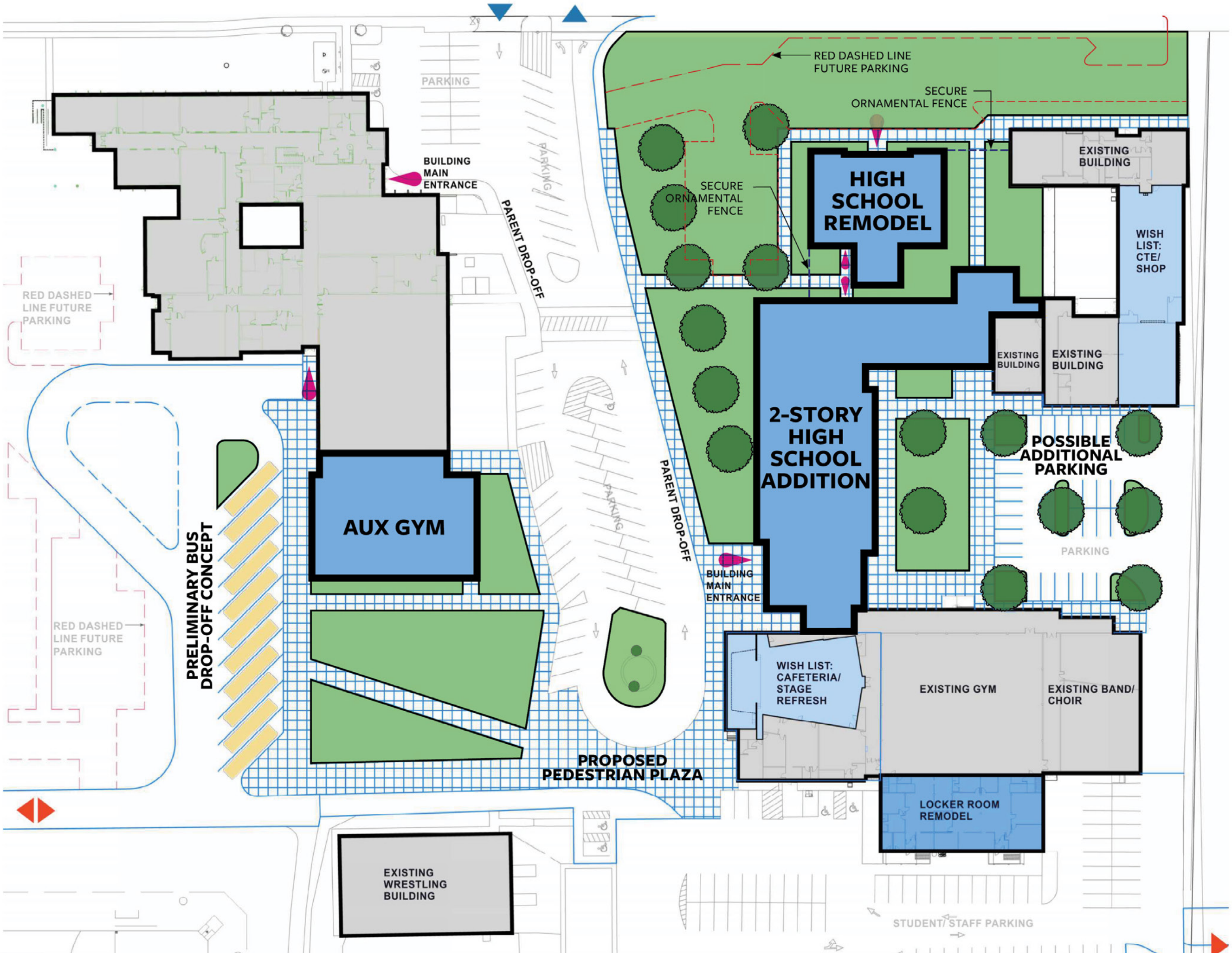
→ **Plan for Future Classroom Expansion** at the Middle School level to meet evolving educational demands.

DISTRICT OFFICE RESTORATION

→ **Develop Scenarios** for restoring the existing 10,500 sf district office into state-of-the-art learning environment.

- Potential renovations to improve functionality and aesthetics.
- Evaluating budgetary impacts
- Maintain historically significant building while integrating into the new addition.

Concept 2: 2024 Bond Program



SITE PLAN

Concept 2: 2024 Bond Program

SITE DESIGN REQUIREMENTS

→ **Enhance Campus-Wide Circulation and Traffic Flow:**

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→ **Modern Safety Software:** Implement up-to-date systems for managing campus safety.

→ **Maintenance Fund:** Create a fund for ongoing maintenance of future projects.

REMODEL OF DISTRICT OFFICE

6,000 SF (APPROX.) OF PROGRAM BUT REQUIRES FULL REMODEL OF ENTIRE 10,500 SF BUILDING

10,500 SF of Program x $800/SF = $8,400,000

NEW ADDITION

37,000 SF OF PROGRAM IN NEW ADDITION

37,000 SF of program x $740/SF = $27,380,000

TOTAL CONSTRUCTION BUDGET FOR CONCEPT 2 = $35,780,000

* See Appendix for Concept 2 Budget Neutral Option for reference.

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Civil

FRONTAGE IMPROVEMENTS

- Tax lots 6900 (HS) and 2300 (MS) have a combined 3 access locations on OR 47. Coordination with ODOT required to determine if proposed access locations will be allowed. Tax lot 2400 (south of middle school) currently has no driveway approach on OR 47. ODOT may consider access in this location if the project proposed to change current access locations in general.
- Center turn lane within OR 47 ends adjacent to tax lot 2300 (near middle school). Extending a center turn lane north to the high school may be a consideration from ODOT to improve vehicle movements on OR 47 depending on traffic input.

UTILITIES

(Public sanitary sewer and stormwater systems maintained by Clean Water Services, and public water system maintained by City of Banks, pass through the school campus, across multiple tax lots.)

Sanitary sewer

- Existing 15-inch diameter sanitary sewer should provide adequate pipe service. Pipe condition unknown.

Stormwater

- Existing 18inch diameter stormwater sewer should provide adequate pipe capacity. Pipe condition unknown.
- Changes to impervious area will likely result in needing additional onsite stormwater detention.
- No known onsite underground injection control facilities (UICs) identified by Oregon DEQ.

Water & Fire Protection

- Existing 8 inch (as identified by 2013 MS as-builts) should provide adequate service. City of Banks 2011 Water System Master Plan does not identify any deficiencies in the City water system on the project site. Further investigation and coordination with City required to determine available fire flows during design.

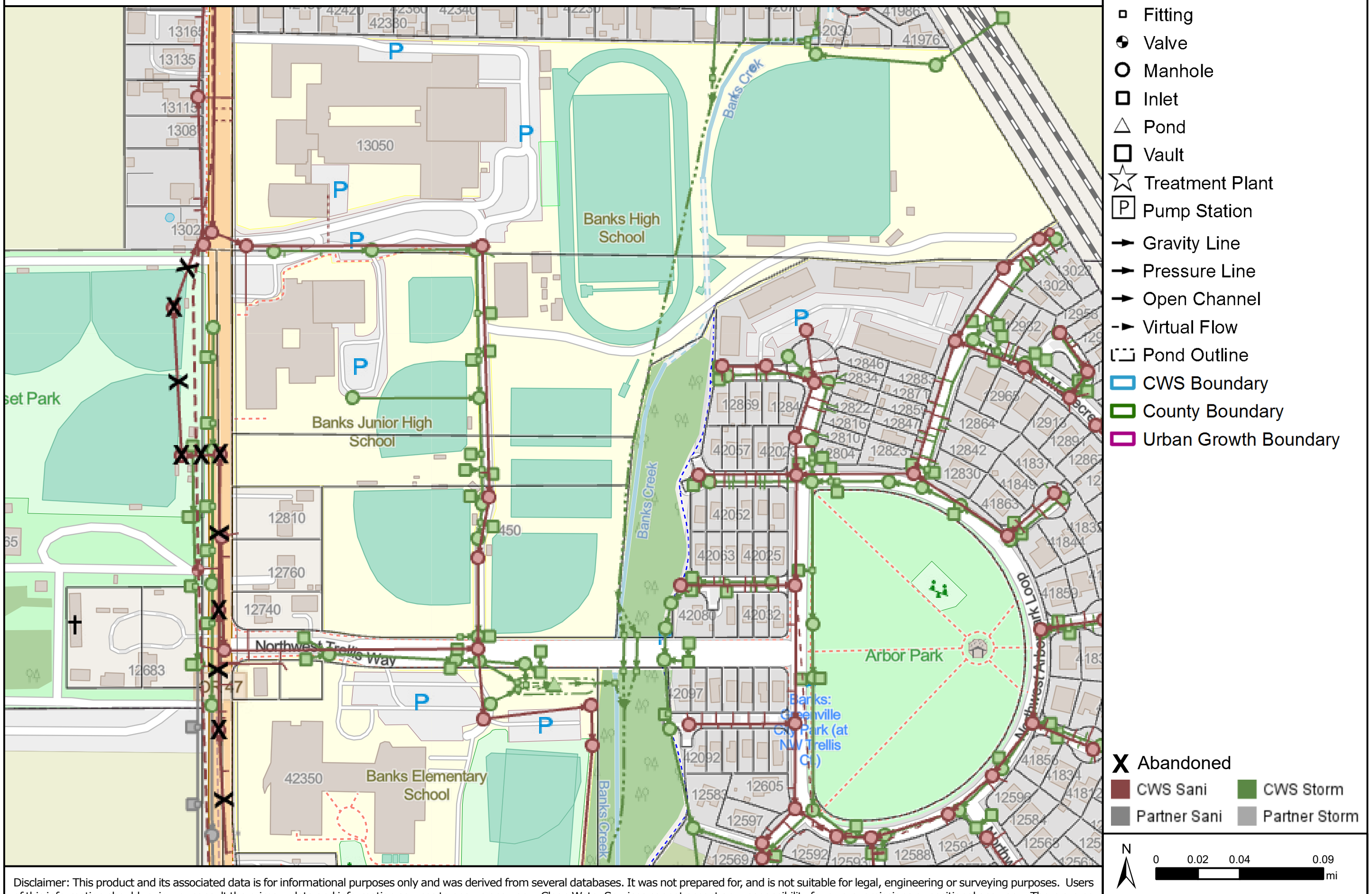
FIRE DEPARTMENT ACCESS

- New circulation patterns and building design will need to consider fire apparatus access routes.

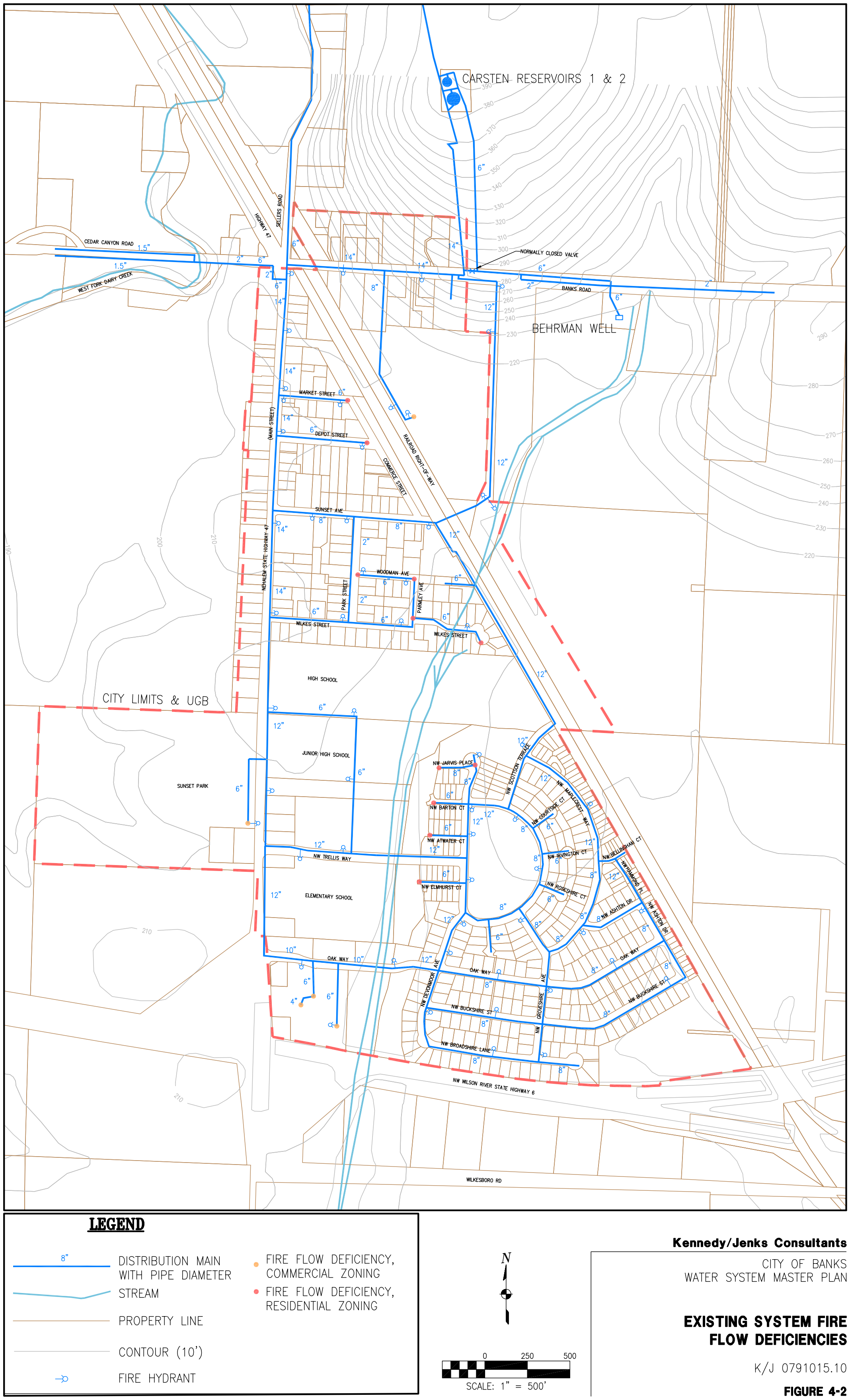
SENSITIVE AREAS

- DEQ Leaking Underground Storage Tanks (LUST) database lists a leaking underground tank as of 2014 at 13050 NW Main St with status “decommissioning”. Further investigation into location/incident/status needed.
- FEMA flood hazard: Zone X (area of minimal flood hazard) for all school tax lots
- It appears Banks Creek has been undergrounded through ~250 ft long culvert(s) near the shotput location. Modifications to this area may require lengthy permitting requirements.

PrintServiceTemplate



Disclaimer: This product and its associated data is for informational purposes only and was derived from several databases. It was not prepared for, and is not suitable for legal, engineering or surveying purposes. Users of this information should review or consult the primary data and information sources to ensure accuracy. Clean Water Services cannot accept any responsibility for errors, omissions or positional accuracy. There are no warranties for this product. Mainline and service lateral locations are depicted using best available information but must be field verified and located before digging. Service laterals are marked in the field as "Unlocatable underground facilities" as defined in OAR 952-001-0010 (20). Easement data is not currently completed District-wide and should be used for general reference only. All sanitary or storm sewer data, with the exception of sanitary lines 24" and larger located within the city limits of Beaverton, Cornelius, Forest Grove, Hillsboro, Lake Oswego, Portland, Sherwood, Tigard or Tualatin, need to be verified by contacting the individual city. Notification of any errors would be appreciated. Clean Water Services, Development Services, 2550 SW Hillsboro Highway, Hillsboro OR 97123, (503) 681-5100.



CARSTEN RESERVOIRS 1 & 2
BEHRMAN WELL
CITY LIMITS & UGB
CEDAR CANYON ROAD
WEST FORK DAIRY CREEK
NORMALLY CLOSED VALVE
BANKS ROAD
MARKET STREET
DEPOT STREET
SUNSET AVE
WOODMAN AVE
WILKES STREET
HIGH SCHOOL
JUNIOR HIGH SCHOOL
SUNSET PARK
ELEMENTARY SCHOOL
NW TRELLIS WAY
OAK WAY
NW WILSON RIVER STATE HIGHWAY 6
WILKESBORO RD
LEGEND
DISTRIBUTION MAIN WITH PIPE DIAMETER
STREAM
PROPERTY LINE
CONTOUR (10')
FIRE HYDRANT
FIRE FLOW DEFICIENCY, COMMERCIAL ZONING
FIRE FLOW DEFICIENCY, RESIDENTIAL ZONING
N
SCALE: 1" = 500'
Kennedy/Jenks Consultants
CITY OF BANKS
WATER SYSTEM MASTER PLAN
EXISTING SYSTEM FIRE FLOW DEFICIENCIES
K/J 0791015.10
FIGURE 4-2

Structural

STRUCTURAL – DISTRICT OFFICE BUILDING ONLY

- District Office Building is a two-story building built in the 1920s.
- Building is constructed with unreinforced masonry exterior walls, slab on grade for the first floor and wood framed second floor, roof, interior walls, and columns.
- The building has currently not been seismically upgraded.
- Proposed seismic upgrades that would be needed for the building for use as an educational facility:
  - Installation of furring walls on the inside of the URM walls attached at top and bottom either mechanically or with adhesive foam to the URM wall along its height.
  - Add plywood sheathing at floor and roof structure.
  - Add anchor ties added around the all URM exterior walls to tie to the floor and roof diaphragms.
  - Add new interior shear walls by applying plywood to the face of existing interior walls, where required.
  - New continuous concrete foundation will likely be required at the perimeter of the existing building and at existing walls that are converted to shear walls at the interior walls.
  - Parapets and chimneys will need to be braced back to the roof structure.

Mechanical, Electrical, Plumbing

DISTRICT OFFICE

- Electrical: service is antiquated and insufficient for modern classroom use. All new electrical service, panels and distribution is required.
- Plumbing: existing piping will not be sufficient for modern classroom use. An increase in restrooms, sinks and drinking fountains will be required and the existing plumbing piping will not be able to accommodate this. All new plumbing piping, sanitary piping, domestic hot water heater and distribution will be required.
- Mechanical: existing mechanical system is not functional. All steam unit ventilators are not functional. One air handler in the center room of the building is operational and does not provide adequate heat and ventilation for the building. No mechanical cooling is present. All new mechanical system would be required to meet code for a modern classroom use.

HIGH SCHOOL

- Electrical: electrical service is adequate, but location of electrical room and transformer is difficult to accommodate with expected work. Preemptive relocation of transformer is recommended. Possibility of relocating the electrical room with a phased approach may be required.
- Plumbing: overall, plumbing piping is adequate. Some modifications/additions have been made that could be redone as part of this scope. Domestic hot water heater is not accessible and barely functional. New water heater is required.
- Mechanical: existing system is adequate and meets current code but is inefficient and maintenance heavy. With a separate air handler and condenser for each classroom, maintenance is required on many small pieces of equipment which is time intensive and expensive. It would be recommended to provide the new school with a centralized mechanical heating and cooling system, ideally chilled and hot water system from a heat pump chiller.
    - Existing steam boiler is barely functional and only serving a few spaces (District office, Cafeteria, Gym). Removal of steam system in its entirety is recommended.
    - Decommissioned oil tank is located underground outside of the boiler room. This tank should be removed and properly disposed of.

Safety and Security Assessments

Safety and security assessments were conducted in fall 2024 for each of Banks School District's three (3) school facilities based on the principles of Crime Prevention through Environmental Design (CPTED). The assessment process included onsite observations by BRIC Architecture on October 22, 2024 as well as follow-up interviews with school principals on October 24-25, 2024. The assessments were conducted by a practitioner holding NICP's CPTED Professional Designation (CPD), meeting all associated training, testing, and continued education requirements.

Recommendations for security improvements across all three campuses were developed based on the assessments. The recommendations were organized into three (3) categories:

- **Tier I:** Improvements to existing campuses and buildings to be incorporated as part of the Phase I bond projects. This cultivated list of improvements was limited by the budgets established under the 2024 bond.
- **Tier II:** Improvements to be implemented as part of the long-term master plan (as additional funds become available).
- **Low Cost / Maintenance Projects:** Lower cost security improvements that could potentially be implemented by maintenance staff.

It was additionally noted that the Banks High School replacement facility would be designed to incorporate CPTED principles, addressing many of the school's existing security vulnerabilities.

The identified Tier I security projects to be addressed as part of Phase 1 are summarized below.

In Oregon, security assessments may be considered public records exempt from disclosure under ORS 192.345 as records or information that reveal or identify security measures or weaknesses in security measures in a building or property (unless the public interest requires disclosure in particular instances). As such, the full assessment report was provided to the District, but is not included in this document.

BANKS ELEMENTARY SCHOOL AND BANKS MIDDLE SCHOOL

- Create secure entry vestibules at BES and BMS that funnel visitors through the main office before granting access to the larger facility.
- Install camera / intercom / buzzer (remote entry) system at main entry doors. Institute one (1) point of entry w/ staff workstation positioned to monitor.
- Add electronic key card access at major exterior doors. Remove exterior door hardware at secondary exterior doors to limit their use to exiting only.
- Install a single integrated security platform:
  - Video management
  - Physical access control and intrusion detection system with central monitoring
  - Lockdown button in the main office.
  - Expanded surveillance camera coverage to encompass hard-to-view areas (interior and exterior), including entries to student restrooms.
- Add external PA speakers, as well as speakers serving interior areas where announcements cannot currently be heard.
- Extend fencing / add gate to connect to northeast side of building at BES.
- At BMS, extend exterior fencing south of building facing Highway 47 to abut school, adding a gate.

BANKS HIGH SCHOOL

- As part of the new design, create a highly visible and celebrated main entry. Design a secure entry vestibule that funnels visitors into the main office before being admitted to the larger building. Install camera / intercom / buzzer at main entry doors.
- Add electronic key card access at major exterior doors (new and existing building areas). Remove exterior door hardware at secondary exterior doors to limit their use to exiting only.
- Add School Guard Glass or security film glazing at the new main entry and reception areas.
- Provide a single integrated security platform (covering both new and existing areas of the building) with video management, physical access control, and intrusion detection systems with central monitoring. Lockdown button to be placed in main office. Thoughtfully incorporate interior cameras with thorough coverage of all corridors, restroom entries, and common areas.
- In the new building, install intruder locks (that can be locked from both sides of the door) in all classroom and office doors.
- In the new building, provide permanently installed window coverings for all classrooms, offices, and core areas.
- Add bollards in front of the school to guard against vehicle impacts.
- Use boulders, seating walls, landscaping, paving, and signage to better delineate public, semi-public, and private zones and distinguish between different school sites
- In the new building, add interior glazing to promote views from classrooms.
- In the new building, design student restrooms to provide a maze entry and open handwashing to provide passive supervision.
- Relocate bike racks to an area within view of staff
- In the new building, add murals, creative displays, or other forms of artwork at key areas of the building and site to communicate a sense of ownership and belonging among students and families as well as promote wayfinding.
- If new construction does not result in a single contiguous building, add 6′ (or higher) fencing encompassing walkway to detached building.
- Add external PA speakers, as well as speakers serving interior areas where announcements cannot currently be heard.
- Address roof access vulnerabilities at BHS in areas of the building to remain, including the railing at the door to (cafeteria) stage, at vehicle gate in alley. (Note: Assuming that roof access vulnerability where existing building intersects with district office will be addressed as part of the new construction.)
- Separate parent and bus drop-off lanes.

Landuse

PROPERTY LINE ADJUSTMENT (PLA) APPLICATION

→ Given the complexities the natural resource areas on the east of the school district properties add to any land use application review, a PLA is recommended to be submitted and approved ahead of submitting for land use for any of the school improvements.

- Time to complete application: 3 weeks
- City review time: ±8-12 weeks (Type II process with notice, but will take limited staff review)
- Record record of survey at Washington County

AUX GYM- SITE DESIGN REVIEW

→ Submit a land use application and obtain approval for the auxiliary gym. (Assumed to be a Type II Site Design Review application.) This would be ahead of the broader master planning efforts of the district and the bigger land use application package that would result.

- Time to complete application package: 3-4 weeks after receipt of ALL plans and other materials
- Submit Clean Water Services SPL after PLA recorded at Washington County
- Submit Application to the City after obtaining CWS SPL
- City review time: ±12-15 weeks (Type II process with notice)

CONDITIONAL USE PERMIT MAJOR MODIFICATION

→ After the BSD led master planning is complete, submit a land use application to modify the Conditional Use Permit for the entire BSD site. This is a Type III process with a neighborhood meeting and public hearing requirement. The Planning Commission is the decision-making body. This would include the changes to circulation, the traffic study, natural resource assessments as needed, etc.

- Have a pre-application meeting with the City to discuss plans and phasing of improvements
- Hold neighborhood meeting prior to application submittal
- Time to complete application package: 3-4 weeks after receipt of ALL plans and other materials
- City review time: ±12-15 weeks (Type III process with public hearing before the Planning Commission)

SITE DESIGN REVIEW FOR BUILDING MODIFICATIONS/ADDITIONS

→ Site Design Review for larger building improvements (those that exceed the thresholds in 151.252.B) require a Type III review process.

- We recommend combining any Type III SDR with the CUP mod application above, so they are processed together and only one neighborhood meeting and hearing are required. The same process as described under "3." above applies.
- Smaller improvements that don't meet the thresholds in 151.252.B could be processed separately through a Type II application. Type II improvements include additions up to 20% of an existing building, or other exterior alterations. The Type II SDR could also be combined with the Type III application if preferred.

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MEMORANDUM

To:	Dan Hess *Principal – BRIC Architecture*
From:	Brian O'Rourke, PE *Project Manager – 3J Consulting*
Date:	November 01, 2024
Project Name:	**Banks School District Master Plan**
Project No:	**24957**
RE:	**Due Diligence Narrative - Civil**

This memorandum has been prepared to list anticipated Civil components required for the Banks School District Master Plan. The Banks School District property is comprised of six separate tax lots ("site"). The elementary school, middle school, high school, school district offices, and athletic fields are all located within the site. Existing site conditions and potential improvements have been evaluated and are provided herein. This list is meant as a supplement to the Banks School District Master Plan site layout options.

Civil Narrative

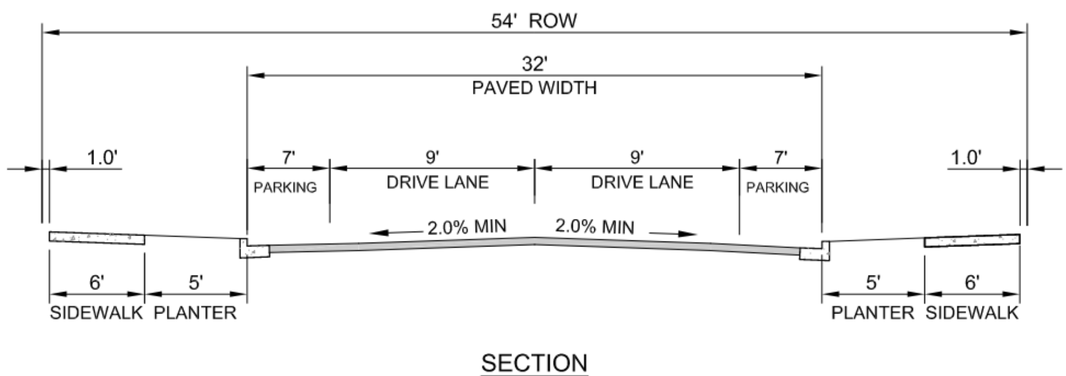
Frontage Improvements

Oregon State Route 47 is a state principal arterial road along the western frontage of four of the six tax lots that make up the site, with a varying number of lanes. OR-47 has two travel lanes and one center lane from NW Oak St until transitioning to two travel lanes with no center lane at approximately the middle school location. Development of the site may require improvements to be made in the school district half of the street, including associated storm water facilities for the added impervious area. Improvements along the frontage should consider the potential for installation of utilities to a new PUE at the back of the new right-of-way. Although not specifically known at this time, Improvements may include continuing a center turn lane within OR-47 along the entirety of the school properties.

NW Wilkes Street is a local road with two travel lanes, parking width, and curbtight sidewalks, and is not currently adjacent to school property. However, the site will have a frontage along NW Wilkes St with plans for the school district to utilize 42270 NW Wilkes St in the site plan. The dimensional standards for a city local road are:

- One half of a local road for approximately 50 feet of frontage including the following:
  - Two 12-foot travel lane with two 7-foot parking widths
  - 5-foot landscape strip (south side only)
  - 6-foot sidewalk
- Associated signage, striping, street lighting and landscaping

Frontage improvements are based on street cross sections from the City of Banks Public Works Design Standards, Appendix A, adopted by City Council on June 10, 2014, and revised February 2022.



NOTES:

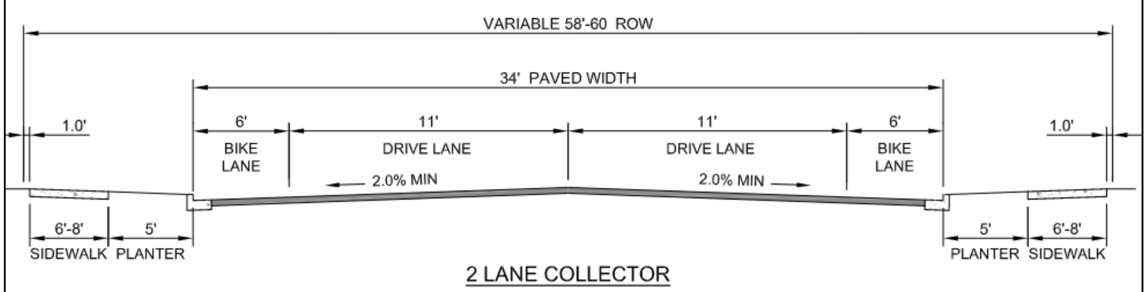
1. PAVED WIDTH AND PLANTER STRIP ARE MEASURED TO FACE OF CURB.
2. PROVIDE 1.0 FEET FROM RIGHT-OF-WAY LINE TO THE BACK OF SIDEWALK FOR MAINTENANCE AND SURVEY MONUMENT PROTECTION.
3. STREET TREES AND STREET LIGHTS ARE REQUIRED.

Local Road - Typical Street Section (City of Bank Public Works Design Standards, Dwg No. 103)

NW Trelles Way is a collector that is in the middle of the site and splits the elementary school to the south and middle/high schools to the north. It currently has two travel lanes with a landscaped median and separated sidewalks with landscaped strips. between the elementary school and middle school. Required improvements are currently unknown, as the existing conditions do not match the City design standard for a collector but have been improved with previous development.

NW Oak Way is a collector that is located on the south end of the site and is adjacent to only one school tax lot. It currently consists of two travel lanes, bike lanes, and separated sidewalks with landscaping strips. The dimensional standards for a two-lane collector are:

- One half of a local road for approximately 175 feet of frontage including the following:
  - Two 11-foot travel lanes with two 6-foot bike lanes
  - 5-foot landscape strip (south side only)
  - 6-foot sidewalk
- Associated signage, striping, street lighting and landscaping



NOTES:

1. PAVED WIDTH AND PLANTER STRIP ARE MEASURED TO FACE OF CURB.
2. PROVIDE 1.0 FOOT FROM RIGHT-OF-WAY LINE TO THE BACK OF SIDEWALK FOR MAINTENANCE AND SURVEY MONUMENT PROTECTION.
3. STREET TREES AND STREET LIGHTS ARE REQUIRED.
4. 8' SIDEWALKS FOR FOR RESIDENTIAL STREETS AND 6' SIDEWALKS FOR COMMERCIAL/INDUSTRIAL AREAS.

Collector Road - Typical Street Section (City of Bank Public Works Design Standards, Dwg No. 102)

Driveway Access

- NW Main St: The site currently has three (3) access on NW Main St, which is an ODOT right-of-way. The improvements currently propose an additional fourth access point. Coordination with ODOT is required to determine implications of proposing an additional access.
- NW Trellis Way: The site currently has an access location from the north and south properties to NW Trellis Way, which is a city right-of-way. The project does not propose changing these accesses.
- NW Oak Way: The site currently has one access to NW Oak Way, which is a city right-of-way. The access is chained off and not currently in use. The project proposes utilizing this existing access for bus circulation.
- NW Wilkes St: The site has no current access to NW Wilkes St, which is a city right-of-way. The School District has purchased 42270 NW Wilkes St and intends to use this property to connect access from the site to NW Wilkes St. Coordination with the City is required to determine implications of proposing a new access.

Parking

- The site has a total of 172 striped parking spaces near the high school and middle school.
- Proposed parking spaces should meet the City of Banks minimum parking requirements.

Fire Department Access

- Fire department access will be required across the site for all facilities. The site layout must consider adequate fire access spacing and turn radius for all drive aisles, including aerial apparatus access. The existing drive aisle on the north side of the high school does not meet current fire access width and spacing. Final coordination with the Fire Marshal will be occurring during the next phase of the project.

Utilities

- Stormwater
    - An existing 18-inch diameter stormwater sewer maintained by Clean Water Services provides service to the site. The pipe flows west to east between the high school and middle school, then south through the site and across NW Trellis Way and outfalls into an existing extended dry pond. The pond currently provides both water quality treatment and detention. Stormwater then outlets directly into Banks Creek to the east.
    - Gravity PVC pipe connections from the new site buildings and impervious areas to the existing main will be required. The existing 18-inch diameter pipe should provide adequate pipe capacity to service the new buildings on site.
    - New water quality and detention facilities will be required to accommodate the proposed new and modified impervious areas per Clean Water Services design standards.
    - There are no onsite underground injection control (UIC) facilities identified by Oregon DEQ.
    - Based on the draft geotechnical report infiltration testing rates, stormwater infiltration on the site is not feasible.

- Sanitary Sewer
    - An existing 15-inch diameter sanitary sewer main pipe maintained by Clean Water Services provides service to the site. The pipe flows north to south down NW Main St, east through the site between the high school and middle school, then south through the site and connecting to the existing system in NW Oak St. Pipe conditions are unknown. A public utility easement for this sanitary line may exist but is not known at this time.
    - Gravity PVC pipe connections from new site buildings to the existing main will be required. The existing 15-inch diameter pipe should provide adequality pipe capacity to service new school buildings on the site.
    - Any new kitchen will require a new grease interceptor be installed.

- Water and Fire Protection
    - The existing water system maintained by the City of Banks provides domestic and fire service to the site. There is a 12-inch main in NW Main St, a 12-inch main in NW Trellis Way, and a 10-inch main in NW Oak Way. An 8-inch main, connecting the lines in NW Main St and NW Trellis Way, loops through the site between the high school and middle school, and along the current bus drive aisle. There has been no public utility easement identified for the 6-inch waterline at this time.
    - There are no identified fire flow deficiencies identified at the site in the 2023 City of Banks Water System Master Plan. Fire flow will need to be confirmed adequate for all proposed buildings on site during design.
    - New fire protection service will require a DCDA in a below grade vault (with power)

and routed directly to the building mechanical room. Additional fire hydrants may be needed onsite to provide the needed fire protection around the buildings.
  - Materials will be ductile iron pipe and precast concrete vaults for water and fire protection. Irrigation pipe materials will be as specified by the project landscape and irrigation design.

Earthwork

The site currently consists of an elementary school, middle school, high school, school district offices, parking facilities, and athletic fields. Grades generally slope gradually from the west to the east towards Banks Creek, which borders the site to the east. The following are anticipated efforts for earthwork activities.

- Stripping of all vegetation surface organics and loose surface soils within all proposed building and hardscaped areas receiving structural fill per geotechnical recommendations.
- Over excavations required to remove brush and trees to be backfilled with structural fill where necessary.
- Existing building demolition may show areas of utility tunnels or other pits and should be considered when backfilling for the new structure or filling to final grades.
- The site will likely require several areas of over excavations required to achieve compaction requirements, even in the dry weather months.
- Earthwork completed during the wet weather months between October 31st and June 1st should consider excavated soil to be unsuitable to be recompacted and any fill to be imported borrow.
- Existing building demolition may reveal areas of utility tunnels or other pits and should be considered when backfilling for new structures or filling to final grades.
- Site accessibility requirements per the Americans with Disabilities Act Accessibility Guidelines will be accommodated as the design of the building and site progresses.

Sensitive Areas

- Banks Creek runs from north to south near the eastern edge of the site. The creek routes through the two northernmost tax lots, which include the high school and middle school. A wetland biologist should determine the associated wetland limits and wetland buffer limits for this creek.
  - It appears that Banks Creek has been undergrounded through approximately 250-ft long culvert(s) just east of the track. Modifications to this area may require lengthy permitting requirements.
  - The site contains a Low Hydromodification Risk Level area along Banks Creek.
- The FEMA flood hazard is Zone X (area of minimal flood hazard) for all school tax lots.
- The DEQ Leaking Underground Storage Tanks (LUST) database lists a leaking underground tank as of 2014 at 13050 NW Main St with status “decommissioning”. Further investigation into this location and incident will be needed.

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BANKS SCHOOL DISTRICT – DISTRICT OFFICE
CAMPUS PLANNING
STRUCTURAL NARRATIVE

DESCRIPTION

The existing Banks School District, District Office Building, located at 12950 NW Main Street, Banks, Oregon is a two-story structure partially occupied and used mostly for administrative staff office space and storage. The building is estimated to have been built in the 1920's; however, existing drawings are not available, and its exact construction date is unknown. From what we can gather visually, the building is constructed with unreinforced masonry exterior walls, slab on grade concrete at the first floor, and wood framed second floor, roof, interior walls, and columns. To our knowledge, the building has never been seismically upgraded.

EXISTING LATERAL SYSTEM

Given the era of construction, the building's lateral force resisting system, or lack thereof, is deficient compared to today's building codes and construction practices. Seismic renovations to the existing structure would include but not be limited to the following:

- **Unreinforced Masonry Exterior Walls**
  - URM exterior walls need to be completely alleviated of their duty of supporting the building both laterally and vertically. This is done by installing a new furring wall constructed of either wood or CFS studs on the inside of the URM over its entirety. The furring wall is mechanically fastened top and bottom to the foundation, floor, or roof. The new wall will also need to be attached to the existing URM along its height. For this, we propose a special adhesive foam that essentially glues the stud wall to the existing URM. This has become more common recently and eliminates some of the costs associated the mechanical fasteners, which has been the traditional approach.

- **Floor and Roof Diaphragms**
  - Plywood sheathing needs to be added to both the floor and roof structure. This can be done from either above or below the existing structure at the floor and potentially the roof, though it is more likely the new sheathing is best installed over the top of the roof framing. This means the roofing material needs to be completely demolished and replaced in order for the roof sheathing to be installed. Miscellaneous steel straps should also be anticipated at discontinuities in the framing, such as around the clerestory opening in the second floor and around the exterior.

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BANKS SCHOOL DISTRICT – DISTRICT OFFICE
CAMPUS PLANNING
STRUCTURAL NARRATIVE

- **Exterior Wall to Diaphragm Ties**
  - As is, the URM exterior walls are not adequately attached to the floor and roof diaphragms. Horizontal anchor ties need to be added around the entire exterior of the building at 4-8 feet on center at the floor and roof. This requires special inspection at all attachment points.

- **New Interior Shear Walls**
  - New shears walls likely need to be created inside of the existing building footprint. This would be done by applying plywood sheathing and panel edge blocking to the face of existing wood stud walls. A continuous concrete foundation would also need to be added or renovated at these locations depending on existing conditions.

- **Foundations**
  - Information on the foundations is unknown at this time, but given the era of construction, they are most certainly undersized and lack the rebar reinforcing steel that would be seen in more modern construction. A new continuous concrete foundation will need to be poured beneath the new furring walls around the perimeter and anchored to the existing foundations beneath the existing URM. It is also likely that any wood framed wall at the interior that is converted to a shear wall will need additional foundation work as well.

- **Parapets and Chimneys**
  - The existing parapets need to brace back to the roof structure to ensure adequate lateral stability in a seismic event. The process of installing the parapet braces will require removing and replacing some roofing material if the roofing is not already removed to apply plywood sheathing.
  - It is recommended that the existing chimney structures be demolished entirely. However, if they are to remain, a furring wall around the interior of chimney can be constructed to support it. It is also possible to build a steel cage like structure internally to support it depending on the exact dimensions and construction of the chimney which is unknown at this time.

EXISTING GRAVITY SYSTEM

The vertical force resisting system consists of URM exterior walls, wood floor joists, wood roof structures, interior wood framed stud walls, and wood posts/columns. The first floor has concrete slab on grade, no information is known about the exterior or interior foundations at this time.

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CAMPUS PLANNING
STRUCTURAL NARRATIVE

The ASCE 41 seismic renovation procedures exclusively focus on the analysis and improvement of an existing buildings lateral system; however, that does not mean the gravity system should be ignored. Elements that do not participate in the lateral force resisting system still need to maintain adequate integrity during a seismic event. It is recommended that the gravity system is assessed during the seismic renovation construction work to ensure a gravity load path is maintained. This could mean potentially upgrading or providing proper connections at the tops and bottoms of walls and columns, upgrading floor/roof ledgers, additional anchorage, and hangers. It would also be recommended to address any serviceability issues such as sagging of horizontal framing members or vibrations reported by the users.

SUMMARY

In summary, the District Office building needs to be completely stripped down to its bare bones in order to be seismically renovated and brought up to modern code standards for existing buildings. All interior wall finishes, flooring and/or ceiling finishes, and roofing needs to be demolished in order to get access to and install the seismic renovation remedies. New foundations would need to be poured to support this new structure, slabs on grade would need to be partially or fully demolished in order to pour these new foundations at the interior in many instances as well. In our experience, there needs to be significant contingency funds set aside for the unknowns that the design team and contractor will certainly come across once the building is opened up. Buildings of this era were built to a different standard and often required the contractor to improvise on site with no oversite and documentation of these decisions.

Though not uncommon, this type of extensive seismic renovation is mostly exclusive to buildings on the historic registry list or other building of major significance. The cost per square foot of these renovations should be carefully compared with that of modern construction.

SÄZÄN GROUP

REPORT DATE: 11/25/2024

PROJECT: Banks SD Site Assessment

ATTENDEES: Kayla Edwards, Steve Watkins, Arnold Luong

Report Purpose

Säzän Group (Säzän) was on-site to conduct a site observation to assess the condition of the mechanical and electrical systems at Banks High School and Banks Middle School.

High School – Mechanical

General Observations

Säzän arrived onsite at 2PM and met with Dale from the Banks School District maintenance department. All areas of the high school and middle school were reviewed, including the roofs, mechanical and electrical spaces.

The current high school is served by various different HVAC systems. The classroom spaces have been converted to residential style furnaces and outdoor heat pumps. The gym and cafeteria are served by large air handling units that have steam heating coils. There are also several rooftop heat pumps serve various spaces throughout the building. Exhaust fans serve as general exhaust and lab exhaust.

There is a domestic hot water heater that serves the gym and cafeteria areas. It has been reported that domestic hot water to the main sections of the school have had to be shut off. Domestic cold water piping is routed in the hallways to reach the chemistry lab.

Figure 1 *To upgrade the chemistry classroom, cold water piping has been run along the ceiling in the hallways. It is at risk of failure and could cause large flooding issues in the hall.*

The building does not have a fire sprinkler system and is served only by a fire alarm system.

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Equipment Condition

While the classroom furnaces and the rooftop air handling units are functional at this time, they are past expected useful life and are not an efficient means for heating and cooling the school. All units are using R410 refrigerant and replacement parts are no longer being manufactured for these units.

Figure 2 *There are four rooftop air handling units that past expected useful life.*

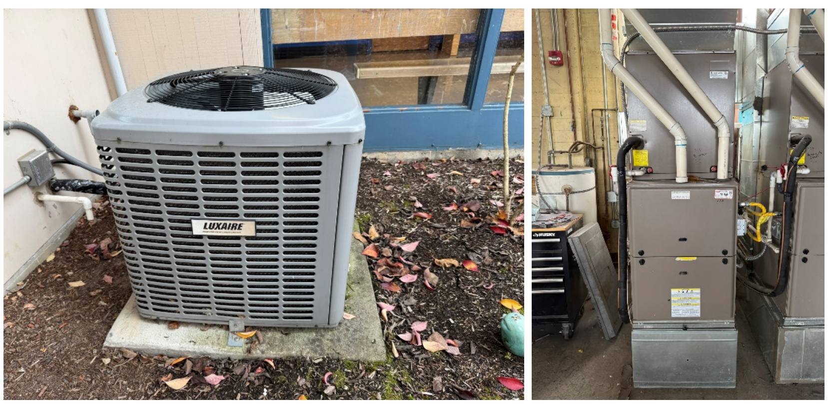

Figure 3 *The classrooms are served by residential style furnaces and outdoor condensing units.*

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Figure 4 *Gas-fired HVUs serve the CTE classroom spaces. These units appear to be in good condition overall and upgrades may not be necessary.*

The boiler system is past end of useful life and failing. Steam lines to certain areas of the building have had to have the steam lines shut off due to leaks below the gym floor and other areas. There is a decommissioned oil tank outside the boiler room located underground.

Figure 5 *The existing steam boiler is aging and minimally functional. Steam distribution throughout the building is failing and sections of piping have had to be valved off to prevent leaks coming up through flooring.*

The gym and cafeteria air handling units are also past end of useful life. The air handling unit serving the locker rooms is not functional. Split-system AC units were installed in 2023 to provide heating and cooling for the locker room spaces.

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Figure 6 *The gym air handling unit has steam heating coils and while currently functional, is past end of useful life.*

A majority of the exhaust fans serving the school have been reported to have failed. The building exhaust was adjusted during COVID to meet minimum exhaust requirements, and all other fans have been left in their failed state.

There is a mix of DDC and pneumatic controls for the building. The system is aging overall and due for an upgrade. The air compressor that serves the pneumatic controls is not functional.

The domestic hot water heater is failing with only one heating element remaining operational. It is also inaccessible for maintenance.

Recommendations

It is recommended that the HVAC system be upgraded completely to a centralized heating and cooling plant that utilizes a heat pump chiller to provide hydronic heating and coolinFj8g to the school. The steam system should be removed entirely as it is functional in only a few spaces and is causing maintenance issues with distribution piping leaking. Along with a mechanical equipment upgrade, a full controls upgrade will be needed.

The building would need to have fire sprinkler system added to meet current code requirements.

<u>Administration Building – Mechanical</u>

General Observations

The HVAC system for the administration building consists of an air handling unit with steam heating that serves the main boardroom. There are unit ventilators located throughout the building to heat other spaces throughout. There are split-system AC units serving the IT spaces.

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***Figure 7** The unit ventilators serving the admin building are not functional and past end of useful life.*

***Figure 8** Split-system AC units serve the IT office and server room to provide cooling.*

There does not appear to be any domestic hot water in the building.

The building does not have a fire sprinkler system.

Equipment Condition

All equipment is past its expected useful life. The air handling unit serving the board room is functional, but all unit ventilators are inoperable. While the air handling unit is functional, it does not provide adequate heating or ventilation for the space.

Recommendations

The administration building would need a complete mechanical upgrade to meet code requirements for modern classrooms. Domestic hot water would also need to be provided to the building to meet code. All upgrades would need to include mechanical equipment and controls.

A fire sprinkler system would also need to be added to meet current code requirements.

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High School – Electrical

General Observations

The electrical system for Banks High School begins with the primary power supply from a utility pole, where conductors are routed underground through conduits to an underground vault that serves as a central connection point. From the vault, power is distributed to two pad-mount transformers. The pad-mount transformers steps down the voltage to a 208V system and feeds the main electrical service switchboards, which distributes power to the school's various electrical distribution equipment.

Figure 9 *Utility pole*

Figure 10 *Underground Vault*

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Figure 11 *Pad-mounted Transformers*

Equipment Condition

Electrical Service #1

Existing service #1 is a 600A, 120/208V, Main Switchboard (MSP) located outside on the high school building wall in the north parking lot and was installed in 2014. We were not able to gain access to this switchboard to verify all loads and should be further investigated.

Figure 12 *Existing Electrical Service #1 (MSP)*

MSP feeds power to (3) branch panels and (1) subpanel located in the high school electrical room across the hallway from Room #12: A, B (Sec 1), B (Sec 2), and C. The main loads fed from these panels are for HVAC/Mechanical equipment throughout the high school. These panels appear to be in adequate condition. It is possible to reuse these panels for future project needs.

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Figure 13 *Branch Panels A, B, and C in Electrical Room*

The MSP switchboard also serves the shop building branch panels, refer to the shop building section further down in this narrative.

<u>Electrical Service #2</u>

Existing service #2 is a 1200A, 120/208V Main Switchboard located in the storage room west of the gymnasium, adjacent to the courtyard.

Figure 14 *Existing Electrical Service #2*

Existing service #2 Main Switchboard feeds (4) branch panels: J, L, H, and K, with panel K having (3) sections total. Panel L is located next to the existing service #2 Main Switchboard in the same room, and panel K is located in the hallway next to classroom #25. Panel J and H locations are not identified and further investigation is needed.

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Figure 15 *Panel L*

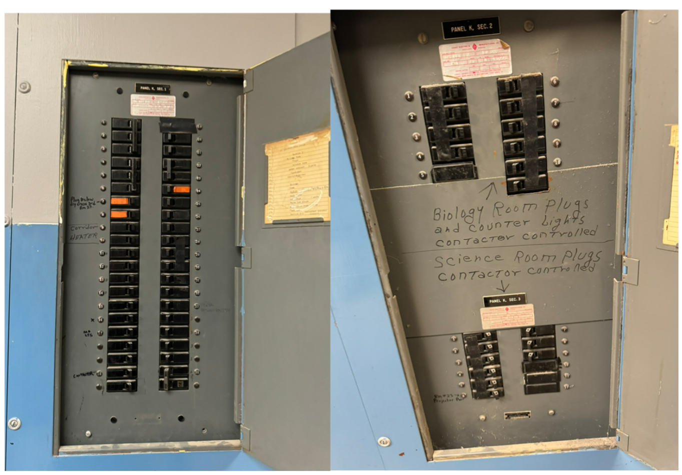

Figure 16 *Panel K*

The existing electrical service #2 distribution and panels appear to be antiquated and lacks sufficient capacity for the future project needs.

<u>Electrical Service #3</u>

Existing service #3 is a 1200A, 120/208V, Main Switchboard located in the high school electrical room across the hallway from Room #12. It is located in the same room with the branch panels (Panels A, B, and C) fed from existing service #1.

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Figure 17 *Existing Electrical Service #3*

Existing service #3 Main Switchboard has existing service disconnects that feeds various loads throughout the high school, however many of them are not labeled. It is uncertain what panels or equipment is currently fed by this switchboard except for a few HVAC/Mechanical equipment.

The existing electrical service #2 distribution and panels appear to be antiquated and lacks sufficient capacity for the future project needs.

Recommendations

The existing electrical service #1 distribution and panels appear to be in adequate condition, it may be possible to reuse the branch panels. However, the distribution equipment is unlikely to have adequate capacity to serve additional loads for the future Banks HS Improvement project. This will be further explored once the scope of work and details for the future project is more established.

The existing electrical service #2 distribution and panels appear to be antiquated and lacks sufficient capacity for the future project needs. It is recommended to provide all new distribution and branch panel equipment.

The existing electrical service #3 distribution and panels appear to be antiquated and lacks sufficient capacity for the future project needs. It is recommended to provide all new distribution and branch panel equipment.

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<u>Administration Building – Electrical</u>

General Observations

Electrical service source to the administration building distribution and panels is not identified, further investigation is needed. There are existing branch panels and disconnects located in the administration building which currently serve power to loads throughout the building.

Equipment Condition

The existing electrical panels and subsequence disconnects located in the administration building are antiquated and insufficient for continued use and longevity for the future project.

Figure 18 *District Office Panels and Disconnects*

Recommendations

The administration building distribution and panels appear to be antiquated and insufficient for administration building use. It is recommended to provide all new distribution and branch panels.

<u>Shop Building – Electrical</u>

General Observations

The existing shop building is currently proposed to remain. There are panels and load centers located inside the building which have been recently installed, but some require additional upgrades.

Equipment Condition

The main electrical service #1 Main Switchboard feeds power to (2) branch panels located in the existing shop building: W and M (See section Electrical Service #1 for additional information for the main switchboard). These branch panels feed power to various loads throughout the existing shop building. There are numerous available circuit breaker spaces inside these panels to potentially feed

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future loads. These panels appear to be in adequate condition. It is possible to reuse these panels for future project needs.

There are also load centers located in the shop that feed power to various loads throughout the existing shop building. Electrical service source to these load centers is not identified. Further investigation is needed. The load centers appear to be antiquated and lacks sufficient capacity for the future project needs.

Figure 19 *Branch Panels W and M fed from MSP in existing Shop Building*

Figure 20 *Load centers in existing Shop Building*

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Recommendation

The existing shop panels W and M appear to be in adequate condition and may be utilized for continued use. This will be further explored as the future project design develops.
The load centers are antiquated and insufficient for shop building use. It is recommended to provide all new load centers.

Cafeteria/Gymnasium Building – Electrical

General Observations

There is a distribution panel located inside the cafeteria/gymnasium area that require additional upgrades.

Equipment Condition

There is an existing 1000A, 120/208V distribution panel located in the cafeteria/gymnasium area. The distribution panel serves various panels located throughout the high school. The locations of these panels and the electrical service source to this distribution panel is not identified. Further investigation is needed. The distribution panel appears to be antiquated and lacks sufficient capacity for future project needs.

Figure 21 *Distribution Panel located in Cafeteria/Gymnasium Area*

Recommendations

The distribution panel appears to be antiquated and insufficient for cafeteria/gymnasium building use. It is recommended to provide new distribution.

End of Report.

Geotechnical Investigation and Site-Specific Seismic Hazard Study

Banks Middle and High School Improvements

Banks, Oregon

November 8, 2024

Prepared for

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1 INTRODUCTION

As requested, GRI completed a geotechnical investigation and site-specific seismic hazard study for the proposed improvements to Banks Middle School and High School in Banks, Oregon. The Vicinity Map, Figure 1, shows the general location of the site. The purpose of the investigation was to evaluate the subsurface conditions at the site and develop geotechnical recommendations for design and construction of the proposed improvements. The investigation included a review of existing geotechnical information for the site and surrounding area, subsurface explorations, laboratory testing, engineering analyses, and seismic studies. As part of our investigation, GRI completed a site-specific seismic hazard study in accordance with the amended Section 1803 of the 2022 *Oregon Structural Specialty Code* (OSSC) for special-occupancy structures, which references the 2016 American Society of Civil Engineers (ASCE) 7-16 document, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7-16). This report describes the work accomplished and provides conclusions and recommendations for use in the design and construction of the proposed improvements.

2 PROJECT DESCRIPTION

We understand the improvements to the middle school will consist of a new auxiliary gymnasium addition, and improvements to the high school will include a new addition. The new auxiliary gymnasium will be constructed adjacent to the northeast corner of the existing middle school in an area that is currently occupied by a sports field and paved areas. The new high school addition will be constructed in the southeast corner of the existing high school footprint, which is currently occupied by a two-story masonry structure that is part of the high school. Structural loads are unknown at this time; however, we anticipate the new additions will have maximum wall loads on the order of 3 kips per foot (kips/ft) to 5 kips/ft. We understand the improvements will be designed as a Risk Category III structure in accordance with ASCE 7-16. Structural performance criteria for the additions under static and seismic conditions are tabulated below, in Table 2-1.

Table 2-1: STRUCTURAL PERFORMANCE CRITERIA

Condition	Total Displacement	Differential Displacement
Static Settlement	< 1 inch	< ½ inch over 50 feet
Seismic Vertical Displacement	N/A	< 1.8 inches over 50 feet
Seismic Lateral Displacement	< 12 inches	N/A

Abbreviation: N/A = not applicable

The structural performance criteria provided in Table 2-1 are based on our experience with similar projects and our review of Section 12.13 of ASCE 7-16. We anticipate the finished floor elevation for the new additions will be consistent with that of the existing schools, and the maximum height of cuts and fills to establish final site grades will be minimal and on the order of 1 foot or less. However, designs for the additions have not been finalized, and permanent retaining walls on the order of 5 feet in height or less may be required to maintain the final site and building grades. In addition, we understand on-site disposal of stormwater is also being considered for this project.

3 SITE DESCRIPTION

3.1 General

Banks Middle School and High School are located at 12850 NW Main Street and 13050 NW Main Street, respectively, in Banks, Oregon. The school campuses are adjacent to each other on one property that is bordered by NW Main Street to the west, residential developments to the north and southeast, agricultural fields to the east, and Banks Elementary School to the south. Review of satellite imagery indicates the ground surface at each school is relatively flat, with elevation changes of less than about 2 feet to 3 feet across the site. We anticipate any existing improvements, such as pavements, foundations, and floor slabs, within the area of the proposed improvements will be demolished as part of this project.

3.2 Geology

Published geologic mapping indicates the site is mantled with Missoula flood deposits, locally referred to in the project area as the Willamette Silt Formation (Wells et al., 2020). In general, Willamette Silt is composed of beds and lenses of silt and sand. Stratification within this formation commonly consists of 4- to 6-inch-thick beds; although in some areas, the silt and sand are massive, and the bedding is indistinct or nonexistent. The Hillsboro Formation, which typically consists of stratified clay, silt, sand, and gravel, commonly underlies the Willamette Silt at depths between about 35 feet and 45 feet in this area.

4 SUBSURFACE CONDITIONS

4.1 General

Subsurface materials and conditions at the site were investigated between August 21 and 23, 2024, with three borings designated B-1 through B-3, two cone penetration test (CPT) probes designated CPT-1 and CPT-2, and one dilatometer test (DMT) sounding designated DMT-1. The borings were advanced to depths of about 31.5 feet to 101.5 feet, the CPT probes to depths of about 62.5 feet to 87.6 feet, and the DMT sounding to a depth of about 53.2 feet below existing site grades. The approximate locations of the explorations completed for this investigation are shown on the Site Plan, Figure 2. Logs of the borings, CPT probes, and DMT sounding are provided on Figures 1A through 8A. The field and laboratory programs completed for this investigation are described in Appendix A. The terms and symbols used to describe the soils encountered in the borings are defined in Table 2A and on the attached legend. Guidelines for the classification of soils with the CPT probes and DMT soundings are provided in Tables 3A and 4A, respectively.

4.2 Soils

The soils disclosed by the explorations completed by GRI at the project site have been grouped into the following units, which are based on the stratigraphic order in which they were encountered:

a. PAVEMENT
b. FILL
c. SILT to Silty CLAY (Willamette Silt)
d. CLAY, SILT, and SAND (Hillsboro Formation)

The following paragraphs generally describe each soil unit and discuss the groundwater conditions at the site. Natural moisture contents, Atterberg-limits indices, fines contents, and other laboratory testing data are provided and discussed in Appendix A. Variations in these units may be present at the site that are not reflected in this report.

a. PAVEMENT

Exploration DMT-1 was advanced in an existing paved area and encountered about 4 inches of asphalt concrete (AC) pavement at the ground surface. The pavement is generally underlain by approximately 8 inches of crushed-rock base course (CRBC).

b. FILL

Silt that was interpreted to be fill was encountered at the ground surface in boring B-2 and extended to a depth of about 5 feet. In general, the silt fill is light brown mottled gray, has low plasticity, and contains a trace to some fine to coarse sand and angular to subangular gravel. Standard Penetration Resistance (SPT N-values) indicate the relative consistency of the silt fill is stiff.

c. SILT to Silty CLAY (Willamette Silt)

Silt to silty clay, interpreted to be Willamette Silt, was encountered at the ground surface in explorations B-1, B-3, CPT-1, and CPT-2, beneath the CRBC in exploration DMT-1, and beneath the fill in boring B-2. The silt to silty clay extends to depths ranging from about 27 feet to 32 feet. In general, the silt to silty clay is brown with varying degrees of gray mottling and grades to gray below a depth of about 20 feet, is typically low to medium plasticity, and contains a trace of fine sand. SPT N-values, Torvane shear-strength values, CPT tip-resistance values, and DMT constrained modulus values indicate the relative consistency of the silt to silty clay soils is soft to very stiff and is typically medium stiff.

Four one-dimensional consolidation tests were performed on samples of silt to silty clay soils obtained at depths of 8 feet and 21.2 feet in boring B-1, 15.8 feet in boring B-2, and 24 feet in boring B-3. The one-dimensional consolidation tests indicate the soils are overconsolidated and exhibit low compressibility in the preconsolidated range of pressures and moderate compressibility in the normally consolidated range of pressures (see Figures 11A through 14A). Monotonic and cyclic direct simple shear (CDSS) testing was also completed on samples of the silt to silty clay obtained at depths of about 20 feet to 22 feet in boring B-1. The CDSS test results indicate the soils generate low levels of excess pore pressures in response to cyclic loading (see Figure 16A). Based on comparisons with regional trends, the behavior is in general alignment with Willamette Silt behavior and should be considered clay-like from a liquefaction evaluation standpoint.

d. CLAY, SILT, and SAND (Hillsboro Formation)

Interbedded layers of clay, silt, and sand soils, interpreted to be the Hillsboro Formation, were encountered beneath Willamette Silt in all the explorations and extended to the maximum depth explored of about 101.5 feet. In general, the unit consists of medium to high plasticity clay that is interbedded with 5-foot to 20-foot layers of sandy silt to silty sand soils between depths of 55 feet and 100 feet. The clay is generally gray with varying degrees of yellow, brown and red mottling, is typically medium to high plasticity, and contains a trace to some fine sand. The silt is generally sandy with fine to coarse sand, light brown with varying degrees of red mottling, and is typically nonplastic to low plasticity. The sand is generally fine to coarse grained, silty, and light brown with varying degrees of red mottling. Up to a trace of subrounded to rounded gravel was encountered at various depths throughout the unit. SPT N-values, Torvane shear strength values, CPT tip-resistance values, and DMT constrained modulus values indicate the relative consistency of the silt and clay soils is medium stiff to hard and the relative density of the sand soils is medium dense.

All of the explorations were terminated in the Hillsboro Formation at depths between about 31.5 feet and 101.5 feet.

4.3 Groundwater

Borings B-1 through B-3 were drilled using mud-rotary drilling techniques, which do not allow direct measurement of groundwater levels at the time of drilling. To allow measurement and periodic monitoring of groundwater levels at the site, a vibrating-wire piezometer (VWP) was installed at a depth of about 51.7 feet below the ground surface in boring B-3. The following table summarizes the groundwater depth measurements from boring B-3.

Table 4-1: BORING B-3 GROUNDWATER DEPTH MEASUREMENT

Date	Depth, feet
10/18/2024	17.3

Based on the VWP readings, we anticipate that the groundwater level in the project area typically occurs at depths between about 10 feet and 20 feet throughout the year. However, perched groundwater conditions may approach the ground surface during the wet winter and spring months or following periods of prolonged or intense precipitation.

4.4 Infiltration Testing

On August 23, 2024, infiltration testing was completed in two boreholes, designated I-1 and I-2, at depths of about 5.5 feet below existing site grades. The approximate locations of the infiltration tests are shown on Figure 2. Details regarding the infiltration testing methods are provided in Appendix A. The unfactored, field-measured infiltration rates recorded at specific depths within specific soil units are tabulated below.

Table 4-2: INFILTRATION TEST RESULTS

Test No.	Depth of Infiltration Test, feet	Average Field Infiltration Rate, inches/hour	Soil Classification
I-1	5	<0.25	SILT, trace sand; ML; brown; low plasticity; moist; fine sand
I-2	5	<0.25	SILT, trace sand; ML; brown; low plasticity; moist; fine sand

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 General

Subsurface explorations completed for this investigation indicate the site is mantled in localized areas with up to 5 feet of silt fill soils, which are underlain by silt to silty clay soils interpreted to be Willamette Silt. In general, Willamette Silt soils were encountered at the ground surface across the majority of the site and extend to depths of about 27 feet to 30 feet. The Willamette Silt soils are underlain by interbedded layers of clay, silt, and sand soils interpreted to be the Hillsboro Formation, which extended to the maximum depth explored of about 101.5 feet. We anticipate that the groundwater level at the site typically occurs at depths of about 10 feet to 20 feet throughout the year; however, perched groundwater may approach the ground surface following periods of intense or prolonged precipitation.

In our opinion, structural loads for the proposed additions can be supported by conventional spread foundations established in compacted structural fill underlain by firm, undisturbed, native soil. The primary geotechnical considerations associated with construction of the additions include the presence of fine-grained soils at the ground surface that are extremely moisture sensitive and the presence of fill soils extending up to 5 feet below the ground surface in localized areas that are not suitable for foundation support. Project plans, specifications, and bid items for the project should address the risk and uncertainties associated with perched groundwater conditions and fill soils mantling localized areas of the site.

The following sections of this report provide our conclusions and recommendations for use in the design and construction of the proposed improvements.

5.2 Seismic Considerations

5.2.1 General

We understand the improvements will be constructed in accordance with the 2022 OSSC, which references ASCE 7-16 for seismic design. A site-specific seismic hazard evaluation was completed for the project to fulfill the requirements of amended section 1803 of the 2022 OSSC for special-occupancy structures. A total stress site-response analysis was completed as part of our site-specific seismic hazard evaluation to evaluate the effect of a code-based seismic event on the site and to better evaluate the liquefaction/cyclic softening hazard. Details of the site-specific seismic hazard evaluation and site response analysis are provided in Appendix B.

5.2.2 Code Background

The ASCE 7-16 seismic hazard levels are based on a Risk-Targeted Maximum Considered Earthquake (MCE_R) with the intent of including the probability of structural collapse. Based on generalized building fragility curves, the seismic design of a structure using the probabilistic MCE_R represents a targeted risk level of 1% in 50-year probability of collapse in the direction of maximum horizontal response. In general, these risk-targeted ground motions are developed by applying adjustment factors of directivity and risk coefficients to the 2% probability of exceedance in 50 years (2,475-year return-period hazard level) ground motions developed from the 2018 U.S. Geological Survey (USGS) probabilistic seismic hazard maps. The risk-targeted probabilistic values are also subject to a deterministic check, which is computed from the models of earthquake sources and ground-motion propagation that form the basis of the 2018 USGS National Seismic Hazard Maps. ASCE 7-16 defines the site-specific deterministic MCE_R ground motions in terms of 84th-percentile, 5%-damped response spectral acceleration in the direction of maximum horizontal response. The MCE_R ground motions are taken as the lesser of the probabilistic and deterministic spectral accelerations.

5.2.3 *Site-Response and Design Acceleration Parameters*

The ASCE methodology uses two bedrock spectral response parameters, S_S and S_1, corresponding to periods around 0.2 seconds and 1.0 seconds, to develop the MCE_R response spectrum. To establish the ground-surface MCE_R spectrum, these bedrock spectral parameters are adjusted for site class using the short- and long-period site coefficients, F_a and F_v, in accordance with Section 11.4.4 of ASCE 7-16, which includes new seismic site coefficients to adjust the mapped values for soil properties.

The S_S and S_1 parameters for the site located at the approximate latitude and longitude coordinates of 45.6149°N and 123.1131°W are 0.92 g and 0.46 g, respectively, for Site Class B/C, or bedrock conditions. Based on our review of Section 20.4.1 of ASCE 7-16, the site is classified as Site Class D based on an estimated shear-wave velocity (V_{S30}) of about 830 feet per second in the upper 100 feet of the soil profile. Site coefficients F_a and F_v of 1.13 and 1.88, respectively, were used to develop the Site Class D MCE_R-level spectrum in accordance with Section 11.4 of ASCE 7-16. However, Section 11.4.8 of ASCE 7-16 requires that a ground-motion hazard or site response analysis be completed for structures on Site Class D sites when the S_1 parameter is greater than or equal to 0.2 g. The code provides an exception that waives the ground-motion hazard or site response analysis if the S_{M1} parameter is increased by 50% in accordance with Supplement 3 issued for Chapter 11 of ASCE 7-16. This exception will increase the seismic base shear for design of the improvements; therefore, a site response analysis was completed as part of our site-specific seismic hazard evaluation for this project.

The recommended MCE_R- and design-level spectral response parameters based on site response modeling for Site Class D conditions are provided below, in Table 5-1. These values assume that dynamic seismic design of the structure will be completed using the Equivalent Lateral Force (ELF) design procedure that meets the intent of ASCE 7-16 with the proposed 2022 OSSC modifications. Given that a site response analysis was completed for this project, the provisions of Supplement 3 issued for Chapter 11 of ASCE 7-16 do not apply. We should be contacted if seismic design will be completed using modal response-spectrum analysis or nonlinear response history analysis methodologies.

Table 5-1: RECOMMENDED SEISMIC DESIGN PARAMETERS FOR ELF, 5% DAMPING

Seismic Parameter	Recommended Value
Site Class	D
MCE_R Spectral Response Acceleration Parameter at Short Periods, S_{MS}	1.10 g
MCE_R Spectral Response Acceleration Parameter at 1.0-Second Period, S_{M1}	0.79 g
Design Spectral Response Acceleration Parameter at Short Periods, S_{DS}	0.74 g
Design Spectral Response Acceleration Parameter at 1.0-Second Period, S_{D1}	0.53 g

Abbreviations: ELF = Equivalent Lateral Force; MCE_R = Risk-Targeted Maximum Considered Earthquake

5.2.4 Liquefaction and Cyclic Softening

5.2.4.1 General

Seismic analyses were completed to evaluate the potential for liquefaction/cyclic softening at the project site for a code-based seismic event. A site-specific site-response analysis was completed as part of our analyses to develop project-specific profiles of cyclic stress ratio (CSR) and normalized stress time histories. The procedure and results of our site-response analysis are provided in Appendix B.

Liquefaction is a process by which saturated, granular materials, such as sand and, to a somewhat lesser extent, non-plastic and low-plasticity silts, temporarily lose strength during and immediately after a seismic event. This degradation in soil properties may be substantial and abrupt, particularly in loose sands. Liquefaction occurs as seismic shear stresses propagate through saturated soil and distort the soil structure, causing loosely packed groups of particles to contract or collapse. If drainage is impeded and cannot occur quickly, the collapsing soil structure causes the pore-water pressure to increase between the soil grains. If the pore-water pressure increases to a level approaching the weight of the overlying soil, the soil temporarily behaves as a viscous liquid rather than a solid. After liquefaction is triggered, there is an increased risk of settlement, loss of bearing capacity, lateral spreading, and/or slope instability, particularly along waterfront areas. Liquefaction-induced settlement occurs as the elevated pore-water pressures dissipate and the soil consolidates after the earthquake.

The cyclic behavior of fine-grained material is generally different from that of granular material; therefore, the term “cyclic softening” is used to differentiate the behavior of fine-grained materials from liquefaction. Cyclic softening describes a relatively gradual and progressive increase in shear strain with seismic load cycles. Excess pore-water pressures may increase due to cyclic loading but will generally not approach total overburden stress. Shear strains accumulate with additional loading cycles, but an abrupt or sudden decrease in shear stiffness is not typically observed. Settlement due to post-seismic consolidation can occur, particularly in lower-plasticity silts; however, settlement does not generally occur to the same degree as sandy soils. Large shear strains can develop, and strength loss related to soil sensitivity may occur in some fine-grained soils.

5.2.4.2 Cyclic Resistance

Laboratory tests and field observations have shown that soil gradation, plasticity, and stress history can have a profound effect on the cyclic behavior of a soil. Idriss and Boulanger (2008) note the importance of these fundamental characteristics for geotechnical engineering analysis and have suggested classifying soils into two primary groups depending on their seismic behavior: sand-like and clay-like materials. Sand-like soils may liquefy, while clay-like soils may experience cyclic softening. A third group, termed "transitional," may exhibit cyclic behavior similar to sand-like and clay-like soils.

The cyclic resistance of soils is dependent on several factors, including the number of loading cycles, relative density, confining stress, plasticity, natural water content, stress history, age, depositional environment (fabric), and composition. For sand-like soils, the cyclic resistance is typically evaluated using SPT N-values or CPT tip-resistance values normalized for overburden pressures and corrected for factors that influence cyclic resistance, such as fines content. For clay-like soils, the cyclic resistance is typically evaluated using estimates of the undrained shear strength, overconsolidation ratio, and sensitivity or directly from cyclic laboratory tests. In practice, the cyclic resistance of these soils is commonly evaluated using simplified correlations based on in-situ testing in conjunction with laboratory index testing. However, more advanced laboratory testing, such as CDSS programs, can be used to estimate cyclic resistance and site-specific soil behavior more accurately, as well as calibrate the simplified methods for a specific soil deposit.

To supplement the practice-oriented approaches and better understand the seismic behavior of the Willamette Silt soils at the site, a laboratory-testing program was performed to evaluate the cyclic resistance, degradation potential, and post-cyclic behavior of these soils. The laboratory testing program foci included static and CDSS tests, along with supporting standard index and consolidation tests. The laboratory test results indicate the Willamette Silt soils generate excess pore pressures in response to cyclic loading and are subject to clay-like behavior at the strain levels estimated from our site-response analysis. In this regard, our analyses considered soils classified as transitional (e.g., fine-grained soils with a soil behavior index [I_c] values between 2.4 and 2.6) to behave in a clay-like manner. Therefore, Willamette Silt soils with I_c values greater than 2.4 are generally not considered susceptible to liquefaction at the strain levels estimated from our site-response analysis but will likely undergo some strength loss due to cyclic softening. Willamette Silt soils having an I_c value of less than about 2.4 are generally considered to exhibit sand-like behavior and are generally considered susceptible to liquefaction. This was based on the laboratory testing program and is consistent with our experience with similar laboratory testing programs and the recommendations of Idriss and Boulanger (2008).

5.2.4.3 Analysis Approach and Methodology

The potential for liquefaction and cyclic softening in the project area was evaluated using the software program CLiq, developed by GeoLogismiki of Neo Souli, Greece. The standard of practice method for liquefaction/cyclic softening analysis (often termed the “Simplified Procedure”) compares estimates of the cyclic shear stresses induced within a soil profile during an earthquake, designated the CSR, with the cyclic resistance of the soil, designated the cyclic resistance ratio (CRR). The factor of safety (FS) against liquefaction or cyclic softening is calculated as the ratio of the CRR to the CSR on a layer-by-layer basis within the soil profile. As the FS decreases to 1.0, there is an increased risk of liquefaction or soil-strength loss.

For this procedure, the CSR profiles were developed based on the results of site-specific site-response modeling performed using the computer program DEEPSOIL, which is discussed further in Appendix B. The DEEPSOIL analyses normally yield the maximum computed shear stress normalized by the initial vertical effective stress (τ_{max}/σ'_v). The DEEPSOIL normalized shear stress was multiplied by 0.65 to convert to an equivalent uniform value (i.e., representative value) of CSR. The CPT explorations and results of the laboratory testing program (which included fines content, consolidation testing, Atterberg-limits indices, and CDSS testing) were used to evaluate the cyclic shear resistance and develop a CRR profile of the soils present below the groundwater level at the site. For the clay-like silt soils, the cyclic resistance was estimated using the stress history and normalized soil engineering properties (SHANSEP) methods presented by Idriss and Boulanger (2008), which are based on the past stress history, overburden stress, and undrained shear strength of the soil. Undrained shear strengths were estimated based on the static and CDSS testing program results and CPT explorations using correlations developed by Robertson (2009). For the sand-like soils present below the silt soils, the cyclic resistance was estimated using CPT tip resistance values normalized for overburden stress and corrected for factors that influence cyclic resistance, such as fines content.

The potential for liquefaction/cyclic softening and subsequent seismically induced settlement at the site was evaluated using the methods recommended by Idriss and Boulanger (2008), with subsequent revisions (2014). The USGS (2018) National Seismic Hazard Maps Project was used to determine the contributing earthquake magnitudes that represent the seismic exposure of the site for the Maximum Considered Earthquake Geometric Mean hazard level. A crustal event on the Gales Creek fault zone and an event on the Cascadia Subduction Zone (CSZ) were determined to represent the sources of seismic shaking. For our analysis, we considered a moment magnitude (MW) 6.8 crustal earthquake, a MW 9 CSZ earthquake, and a groundwater depth of about 15 feet below the ground surface, which corresponds to average groundwater level at the site.

5.2.4.4 Seismically Induced Vertical Settlement

The potential for liquefaction and cyclic softening was evaluated using the CSR profile obtained from site-response modeling, the CPT tip-resistance values to estimate the CRR of the sand-like soils, and the CDSS test results to estimate the CRR of the transitional and clay-like soils. The results of our evaluation indicate the silt soils below the groundwater level at the project site will exhibit limited cyclic softening at the maximum strain levels determined from our site-response analysis. However, our analysis indicates localized interbedded layers of sandy silt to silty soils present between depths of 55 feet and 100 feet at the project site are susceptible to liquefaction during a code-based seismic event.

Estimated dynamic settlements were evaluated using the methodology developed by Idriss and Boulanger (2008), with subsequent revisions (2014). The methodology is based on the strain potential approach detailed by Ishihara and Yoshimine (1992) and later updated by Yoshimine et al. (2006). Our analysis indicates about 1 inch to 2 inches of seismically induced settlement and less than 1 inch of differential seismic settlement over 50 feet could occur due to liquefaction and cyclic softening following a code-based seismic event. The seismically induced settlement will likely occur during and after earthquake shaking. Our estimate of differential seismic settlement over 50 feet is based on our analysis of the variability in subsurface conditions encountered at each exploration location and the thickness of non-liquefiable silt and clay soils above the interbedded layers of liquefiable sandy silt to silty sand soils. This is consistent with the standard of practice and recommendations of Montgomery and Boulanger (2016).

5.2.5 Lateral Spreading

Lateral spreading is a liquefaction-related seismic hazard that may adversely impact some sites. Areas subject to lateral spreading are underlain by soils susceptible to liquefaction and are sloping sites or flat sites adjacent to an open face, such as waterfront areas. The lateral spreading potential at the site is considered negligible due to the relatively flat topography and absence of an open face.

5.2.6 Other Seismic Hazards

The USGS considers the Gales Creek Fault Zone, located about 9.8 kilometers from the site, to be the closest crustal fault source contributing to the overall seismic hazard at the site. Unless occurring on a previously unmapped or unknown fault, it is our opinion that the risk of ground rupture at the site is low. The risk of damage by a tsunami and/or seiche at the site is absent.

5.3 Earthwork

5.3.1 General

The site is mantled with fine-grained silt soils that are moisture sensitive and may be susceptible to softening or disturbance from construction activities during wet conditions. It is our opinion that earthwork can be completed most economically during the dry summer months, which typically extend from June to mid-October. To reduce the risk of disturbing the moisture-sensitive, fine-grained soils, excavation should be completed using track-mounted hydraulic excavators. The excavation should be finished using a smooth-edged bucket to produce a firm, undisturbed surface. It may also be necessary to construct granular haul roads concurrently with excavation to minimize subgrade disturbance. If the subgrade is disturbed during construction, soft disturbed soils should be overexcavated to firm soil and backfilled with granular structural fill.

If construction occurs during wet-ground conditions, gravel work pads and haul roads will be required to facilitate construction access on site and provide a firm working surface for construction activities. In our opinion, a 12- to 18-inch-thick, granular work pad should be sufficient to reduce subgrade disturbance by lighter construction equipment and limited traffic by dump trucks. Haul roads and other high-density traffic areas will require a minimum of 18 inches to 24 inches of fragmental rock, up to 6-inch nominal size, to reduce the risk of subgrade deterioration. The use of a geotextile fabric over the subgrade may reduce maintenance during construction. Haul roads can also be constructed by placing a thickened section of pavement base course and subsequently spreading and grading the excess crushed rock base after earthwork is complete.

5.3.2 Site Preparation

Demolition of existing improvements within the limits of the proposed improvements should include removal of existing pavements, foundations, construction debris, tanks, and underground utilities (if present). The ground surface within all building areas, paved areas, walkways, and areas to receive structural fill should be stripped of existing vegetation, surface organics, and loose surface soils and fill. All demolition debris, trees, brush, and surficial organic material should be removed from within the limits of the proposed improvements. Excavations required during demolition or to remove underground utility remnants, unsuitable soils, brush, and trees should be backfilled with structural fill. Organic strippings should be disposed of offsite or stockpiled on site for use in landscaped areas.

Following stripping or excavation to subgrade elevation, the exposed subgrade should be evaluated by a qualified member of GRI's geotechnical engineering and geology staff. Proof rolling with a loaded dump truck may be part of this evaluation. Any soft areas or areas of unsuitable material disclosed by the evaluation should be overexcavated to firm material and backfilled with structural fill. Due to previous development at the site and the presence of fill soils in localized areas, it should be anticipated some overexcavation of subgrade will be required.

5.3.3 Site Grading

Final grading across the project should provide for positive drainage of surface water away from the building and any exposed slopes to reduce the potential for erosion. Permanent cut and fill slopes should be not steeper than 2H:1V (Horizontal to Vertical) and protected with vegetation to reduce the risk of surface erosion due to rainfall.

5.3.4 Prior Site Development

Site improvements within previously developed areas include the risk of encountering undocumented or poorly documented improvements and infrastructure, such as clay drainage tiles or wells. The possibility does exist to encounter existing underground improvements or relatively thick sections of undocumented fill soils. Where encountered, existing improvements, infrastructure, and undocumented fill soils located within the footprint of the proposed additions must be overexcavated and replaced with compacted structural fill.

5.4 Excavation

5.4.1 General

We anticipate excavations to found the building additions will be minimal and the depth of utility trenches will be on the order of 5 feet to 10 feet deep. The method of excavation and design of excavation support are the responsibilities of the contractor and are subject to applicable local, state, and federal safety regulations, including the current Occupational Safety and Health Administration (OSHA) excavation and trench safety standards. The means, methods, and sequencing of construction operations and site safety are also the contractor's responsibilities. The information provided below is for the use of our client and should not be interpreted to imply that we are assuming responsibility for the contractor's actions or site safety.

5.4.2 Groundwater Management

Depending on the time of year the work is completed, perched groundwater may be encountered in the utility trench excavations. Groundwater seepage, running-soil conditions, and unstable excavation sidewalls or excavation subgrades, if encountered during construction, will require dewatering of the excavation and sidewall support. The impact of these conditions can be reduced by completing excavations during the summer months, when perched groundwater levels are lowest, and by limiting the depths of the excavations.

We anticipate that groundwater seepage, if encountered, can generally be controlled by pumping from sumps. To facilitate dewatering, it will be necessary to overexcavate the base of the excavations to permit the installation of a granular working blanket. We estimate the required thickness of the granular working blanket will be on the order of 1 foot or as required to maintain a stable excavation base. The required depth of overexcavation will depend on the conditions exposed in the excavations and the effectiveness of the contractor's dewatering efforts. The thickness of the granular blanket must be evaluated based on field observations during construction. We recommend the use of relatively clean, free-draining material, such as 2- to 4-inch-minus crushed rock, for this purpose. Depending on the soil conditions encountered, the use of a nonwoven geotextile fabric over the excavation base can be considered to assist in stability and dewatering.

5.4.3 Utility Excavations

In our opinion, there are three major considerations associated with the design and construction of new utilities:

1. Provide stable excavation sideslopes or support for trench sidewalls to minimize loss of ground.
2. Provide a safe working environment during construction.
3. Minimize post-construction settlement of the utility and ground surface.

According to current OSHA regulations, the fine-grained soils encountered near the ground surface in the explorations may be classified as Type B. In our opinion, trenches less than 4 feet deep that do not encounter perched groundwater may be cut vertically and left unsupported during the normal construction sequence, assuming trenches are excavated and backfilled in the shortest possible sequence. Excavations that encounter perched groundwater or are more than 4 feet deep should be laterally supported or alternatively provided with side slopes of 1H:1V or flatter. In our opinion, adequate lateral support may be provided by common methods, such as the use of a trench shield or hydraulic shoring systems.

5.5 Structural Fill

We anticipate minor amounts of structural fill will be placed for this project. In this regard, we recommend structural fill consist of imported granular material such as sand, sandy gravel, or crushed rock with a maximum size of 2 inches. Granular material used to construct structural fills during wet weather should not contain more than about 5% to 7% passing the No. 200 sieve (washed analysis). Granular fill should be placed in lifts and compacted with vibratory equipment to at least 95% of the maximum dry density determined in accordance with ASTM International (ASTM) D698. Appropriate lift thicknesses will depend on the type of compaction equipment used. For example, if hand-operated, vibratory-plate equipment is used, lift thicknesses should be limited to 6 inches to 8 inches. If smooth-drum vibratory rollers are used, lift thicknesses up to 12 inches are appropriate, and if backhoe- or excavator-mounted vibratory plates are used, lift thicknesses up to 2 feet may be acceptable.

All utility-trench excavations within building, pavement, and hardscape areas should be backfilled with relatively clean, granular material such as sand, sandy gravel, or crushed rock of up to 1½-inch maximum size and having less than 5% passing the No. 200 sieve (washed analysis). The bottom of the excavation should be thoroughly cleaned to remove loose materials, and the utilities should be underlain by a minimum 6-inch thickness of bedding material. The granular backfill material should be compacted to at least 95% of the maximum dry density determined by ASTM D698 in the upper 5 feet of the trench and at least 92% of this density below a depth of 5 feet. The use of hoe-mounted, vibratory-plate compactors is usually most efficient for this purpose. Flooding or jetting as a means of compacting the trench backfill should not be permitted.

On-site, fine-grained soils and site strippings may be used as fill in landscaped areas. These materials should be placed at about 90% of the maximum dry density, as determined by ASTM D698.

5.6 Foundation Support

5.6.1 General

Foundation loads for the additions are currently unavailable; however, we anticipate the maximum wall loads will be on the order of 3 kips/ft to 5 kips/ft. In our opinion, foundation support for the proposed additions can be provided by conventional column-type and continuous spread footings in accordance with the following design criteria. As discussed in Section 5.2.4.4 of this report, our liquefaction analyses indicate 1 inch to 2 inches of total seismic settlement and less than 1 inch of differential seismic settlement over 50 feet could occur due to liquefaction following a code-based seismic event. The thicknesses of non-liquefiable soils that mantle the site may reduce surface manifestation of the settlement and the magnitude of differential lateral displacement. Based on our seismic studies, it is our opinion that the seismic performance criteria provided in Table 2-1 of this report and Table 12.13-3 of ASCE 7-16 can be achieved without requiring mitigation such as ground improvement or deep foundations. However, GRI should be provided the building loads to review and confirm our foundation recommendations.

5.6.2 Foundation Subgrade and Base Course Preparation

All new conventional footings should be established in firm, undisturbed native soil or compacted granular structural fill underlain by native soil. All footing subgrades should be evaluated by a geotechnical engineer. Any areas of soft or unsuitable material should be overexcavated and backfilled with granular structural fill. Our experience indicates that fine-grained soils are easily disturbed by excavation and construction activities. Therefore, we recommend the footing subgrade be blanketed with a minimum 3-inch-thick layer of compacted crushed rock to provide a firm working surface. Relatively clean, ¾-inch-minus crushed rock is suitable for this purpose and should be compacted with a lightweight vibratory compactor.

5.6.3 Conventional Spread Foundations

The base of all new footings should be established at a minimum depth of 18 inches below the lowest adjacent finished grade. The footing width should not be less than 24 inches for isolated column footings and 18 inches for wall footings. New foundations established in accordance with these criteria can be designed on the basis of an allowable soil bearing pressure of 2,500 pounds per square foot, which assumes the footings will be established in compacted granular structural fill underlain by firm, undisturbed native soil. This value applies to the total of dead load and/or frequently applied live loads and can be increased by one-third for the total of all loads: dead, live, and wind or seismic. We estimate the total settlement of spread footings designed in accordance with the recommendations presented above will be less than 1 inch for maximum wall loads up to 5 kips/ft. Differential settlements between adjacent comparably loaded footings should be less than half the total settlement. Our experience indicates these settlements will occur rapidly, with the majority of the settlement occurring during construction.

5.6.4 Lateral Foundation Resistance

Horizontal shear forces can be resisted partially or completely by frictional forces developed between the base of the footings and the underlying soil and by soil passive resistance. The total frictional resistance between the footing and the soil is the normal force times the coefficient of friction between the soil and the base of the footing. We recommend an ultimate value of 0.4 for the coefficient of friction for footings cast on granular material. The normal force is equal to the sum of the vertical forces (dead load plus real live load). If additional lateral resistance is required, passive earth pressures against embedded footings can be computed based on an equivalent fluid having a unit weight of 250 pounds per cubic foot. This design passive earth pressure would be applicable only if the foundation is cast neat against undisturbed soil or if backfill for the foundations is placed as granular structural fill and is based on the assumption that up to ½ inch of lateral movement of the structure will occur in order for the soil to develop this resistance. This value is also based on the assumption that the ground surface in front of the foundation is horizontal (i.e., does not slope downward away from the toe of the footing).

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5.7 Subdrainage/Floor Support

To provide a capillary break and reduce the risk of damp floors, slab-on-grade floors established at or above adjacent final site grades should be underlain by a minimum of 8 inches of free-draining, clean, angular rock capped with a 2-inch-thick layer of ¾-inch-minus crushed rock to improve workability, for a total rock section of 10 inches. The 8-inch free-draining section should consist of angular rock such as 1½- to ¾-inch crushed rock with less than 2% passing the No. 200 sieve (washed analysis). The slab base course section should be placed in one lift and compacted to at least 95% of the maximum dry density (ASTM D698) or until well keyed. In areas where floor coverings will be provided or moisture-sensitive materials stored, it would be appropriate to also install a vapor-retarding membrane. The membrane should be installed as recommended by the manufacturer. In addition, a foundation drain should be installed around the building perimeter to collect water that could potentially infiltrate beneath the foundations and should discharge to an approved storm drain.

Although the finished floor elevation for the addition is anticipated to be established at or above the adjacent surrounding site grades, if structures such as floors are established below final site grades, the structure should be provided with a subdrainage system. A subdrainage system will reduce the buildup of hydrostatic pressures on the floor slab and the risk of groundwater entering through embedded walls and floor slabs. GRI should be contacted if embedded structures are being considered.

In our opinion, it is appropriate to assume a coefficient of subgrade reaction, k, of 175 pounds per cubic inch to characterize the subgrade support for point loading with 10 inches of compacted crushed rock beneath the floor slab.

5.8 Retaining Walls

5.8.1 General

We anticipate the new additions will be established near existing site grades; however, retaining walls with a maximum height of 5 feet may be required to maintain final building grades. If required, we anticipate the retaining walls will consist of cast-in-place concrete walls that support the exterior walls of each addition; however, we should be contacted if other types of retaining walls are being considered for this project. GRI should review the final retaining wall plans developed by the structural engineer once they become available to evaluate whether additional considerations, such as impacts on existing infrastructure and improvements, are warranted.

5.8.2 Foundation Design Criteria

Foundation design and subgrade preparation should conform to the recommendations provided in the Foundation Design Criteria section of this report. For areas where the ground surface in front of the wall will be nearly horizontal, we recommend embedding the toe of retaining walls at least 1.5 feet below the finished grade. For walls constructed on slopes up to 3H:1V or flatter, we recommend the embedment depth be increased to 2 feet. To provide more uniform support, retaining walls supported on spread foundations should be founded on a minimum 6-inch-thick section of compacted crushed rock. The crushed rock section may need to be increased to 2 feet or 3 feet for retaining walls founded on soft soils; however, the actual thickness of the crushed rock section in soft areas should be determined based on field observations during construction.

5.8.3 Lateral Earth Pressures

Design lateral earth pressures for retaining walls depend on the type of construction (i.e., the ability of the wall to yield). Possible conditions are 1) a wall that is laterally supported at its base and top and therefore is unable to yield to the active state and 2) a retaining wall, such as a typical cantilever or gravity wall, that yields to the active state by tilting about its base. Conventional basement walls and cantilevered retaining walls are examples of non-yielding and yielding walls, respectively.

For completely drained, horizontal backfill, yielding and non-yielding walls may be designed based on equivalent fluid unit weights of 35 pounds per cubic foot (pcf) and 55 pcf, respectively. To account for seismic loading, the earth pressures should be increased by 10 pcf and 20 pcf for yielding and non-yielding walls, respectively, with horizontal backfill. These earth pressures assume the walls are fully drained (i.e., hydrostatic pressure cannot build up on the back of the wall). This results in a triangular distribution with the resultant acting at ⅓H up from the base of the wall, where H is the height of the wall in feet. Additional lateral loading due to surcharge loads can be evaluated using the criteria shown on the Surcharge-Induced Lateral Pressure, Figure 3.

The lateral earth pressure criteria presented above are appropriate if the retaining walls are fully drained. We recommend the installation of a permanent drainage system behind all the retaining walls. The drainage system can either consist of a drainage blanket of crushed rock or continuous drainage panels between the retained soil/backfill and the face of the wall. The drainage blanket should have a minimum width of 12 inches and consist of crushed drain rock that contains less than 2% fines content (washed analysis). A typical drainage system for retaining walls is shown on the Typical Subdrainage Detail, Figure 4. The drainage blanket or drainage panels should extend to the base of the wall, where water should be collected in a perforated pipe and discharged to a suitable outlet, such as a sump or approved storm drain. In addition, the wall design should include positive drainage measures to prevent the ponding of surface water behind the top of the wall.

Overcompaction of the backfill behind walls should be avoided. Heavy compactors and large pieces of construction equipment should not operate within 5 feet of any retaining wall to avoid the buildup of excessive lateral pressures. Compaction close to the walls should be accomplished with hand-operated vibratory-plate compactors. Overcompaction of backfill could significantly increase lateral earth pressures behind walls and cause damage to cast-in-place concrete retaining walls.

5.9 Pavement Design

We anticipate any new paved areas included in this project will be subjected primarily to automobile and light truck traffic, with occasional heavy truck traffic. We anticipate the majority of new pavement will consist of AC; however, areas subjected to repeated heavy truck traffic, such as loading dock or trash enclosure areas, will be paved with portland cement concrete (PCC) pavement.

Traffic estimates for paved areas included in this project are currently unknown. Based on our experience with similar projects and subgrade soil conditions, the recommended pavement sections are provided in Tables 5-2 and 5-3, below. Crushed-rock base course is denoted as CRBC in the tables below and should consist of 1- or ¾-inch-minus crushed rock conforming to the imported granular fill recommendations provided in the Structural Fill section of this report.

Table 5-2: RECOMMENDED AC PAVEMENT SECTIONS

Area/Traffic Loading	CRBC Thickness, inches	AC Thickness, inches
Areas Subject to Repeated Heavy Truck Traffic (Trash Enclosure and Service Areas)	12	5
Areas Subject to Primarily Automobile Traffic (Vehicle Drive Lanes)	12	4
Areas Subject to Automobile Parking (Parking Stalls)	8	3

Abbreviations: AC = asphalt concrete; CRBC = crushed-rock base course

Table 5-3: RECOMMENDED PCC PAVEMENT SECTIONS

Area/Traffic Loading	CRBC Thickness, inches	PCC Thickness, inches
Areas Subject to Repeated Heavy Truck Traffic (Trash Enclosure and Service Areas)	6	6

Abbreviations: PCC = portland cement concrete; CRBC = crushed-rock base course

Note: The recommended pavement sections should be considered minimum thicknesses and underlain by a WSF 200 woven geotextile fabric or comparable product.

It should be assumed that some maintenance will be required over the life of the pavement (15 years to 20 years). The recommended pavement sections are based on the assumption that pavement construction will be accomplished during the dry season and after the construction of the building has been completed. If wet-weather pavement construction is considered, it will likely be necessary to increase the thickness of the CRBC to support construction equipment and protect the subgrade from disturbance, as discussed in the Earthwork section of this report. The indicated sections are not intended to support construction traffic such as forklifts, dump trucks, or concrete trucks. Pavements subject to construction traffic should be protected and may require repair.

For the above-indicated sections, drainage is an essential aspect of pavement performance. We recommend all paved areas be provided positive drainage to remove surface water and water within the base course; subgrade should be sloped to a minimum of 0.5% slope to aid in drainage. This will be particularly important in cut sections or at low points within the paved areas, such as at catch basins. Effective methods to prevent saturation of the base-course materials include providing weepholes in the sidewalls of catch basins, subdrains in conjunction with utility excavations, and separate trench-drain systems. To help ensure quality materials and construction practices, we recommend the pavement work conform to current Oregon Department of Transportation standards.

Prior to placing base-course materials, all pavement areas should be proof rolled with a fully loaded dump truck. Any soft areas detected by the proof rolling should be overexcavated to firm ground and backfilled with compacted structural fill.

Provided the pavement section is installed in accordance with the above recommendations, it is our opinion the site-access areas will support infrequent traffic by an emergency vehicle with a gross vehicle weight of up to 75,000 pounds. For the purposes of this evaluation, “infrequent” can be defined as once a month or less. If the frequency of emergency vehicle traffic exceeds this preliminary assumption, GRI should be contacted to review our pavement recommendations.

5.10 On-Site Disposal of Stormwater

The unfactored, field-measured infiltration rate for the Willamette Silt soils that mantle the site is less than 0.25 inches per hour; therefore, it is our opinion the near-surface soils do not meet the requirements for on-site stormwater disposal. If pervious concrete or permeable pavers are selected for the site, the subsurface detention reservoir must be properly sized to meet the hydrological needs such as the volume of water stored based on a design storm and the maximum detention time. Based on the infiltration test results, in our opinion, the site will require an underdrain pipe to move the stormwater to an approved outlet structure.

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6 DESIGN REVIEW AND CONSTRUCTION SERVICES

We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GRI should be retained to review all geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in our report. To observe compliance with the intent of our recommendations, the design concepts, and the plans and specifications, it is our opinion all construction operations pertaining to earthwork and foundation installation should be observed by a GRI representative. Our construction-phase services will allow for timely design changes if site conditions are encountered that are different from those described in our report. If we do not have the opportunity to confirm our interpretations, assumptions, and analyses during construction, we cannot be responsible for the application of our recommendations to subsurface conditions different from those described in this report.

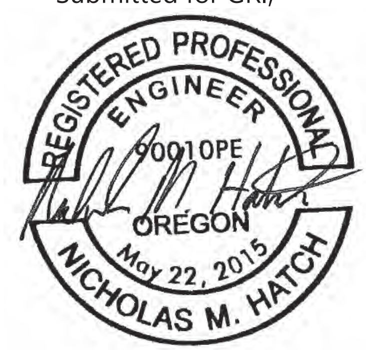
7 LIMITATIONS

This report has been prepared to aid the project team in the design of this project. The scope is limited to the specific project and location described within this report, and our description of the project represents our understanding of the significant aspects of the project relevant to earthwork, design, and construction of the proposed improvements. If any changes in the design and location of the project elements as outlined in this report are planned, we should be given the opportunity to review the changes and modify or reaffirm the preliminary conclusions and recommendations of this report in writing.

The conclusions and recommendations in this report are based on the data obtained from the subsurface explorations at the locations shown on Figure 2 and other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in subsurface conditions may exist between exploration locations. This report does not reflect variations that may occur between these explorations. The nature and extent of variation may not become evident until construction. If during construction, subsurface conditions differ from those encountered in the explorations, we should be advised at once so we can observe and review these conditions and reconsider our recommendations where necessary.

We have included as Appendix C the Geoprofessional Business Association guidance document “Important Information about This Geotechnical-Engineering Report” to assist you and others in understanding the use and limitations of this report. We recommend you read this document.

Submitted for GRI,



RENEWS: 06/2025

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This document has been submitted electronically.

8 REFERENCES

American Society of Civil Engineers (ASCE) , 2016, ASCE 7-16, Minimum design loads for buildings and other structures: ASCE, Reston, Virginia.

ASCE, n.d., ASCE Hazard Report lookup by latitude, longitude, accessed September 4, 2024, from: https://ascehazardtool.org

Idriss, I. M., and Boulanger, R. W., 2008, Soil liquefaction during earthquakes: Earthquake Engineering Research Institute, EERI MNO-12.

Idriss, I. M., and Boulanger, R. W., 2014, CPT and SPT based liquefaction triggering procedures: Department of Civil & Environmental Engineering, College of Engineering, University of California at Davis, Report No. UCD/CGM-14/01.

Ishihara, K., and Yoshimine, M., 1992, Evaluation of settlements in sand deposits following liquefaction during earthquakes, *Soils and Foundations*, vol. 32, no. 1, pp. 173–188.

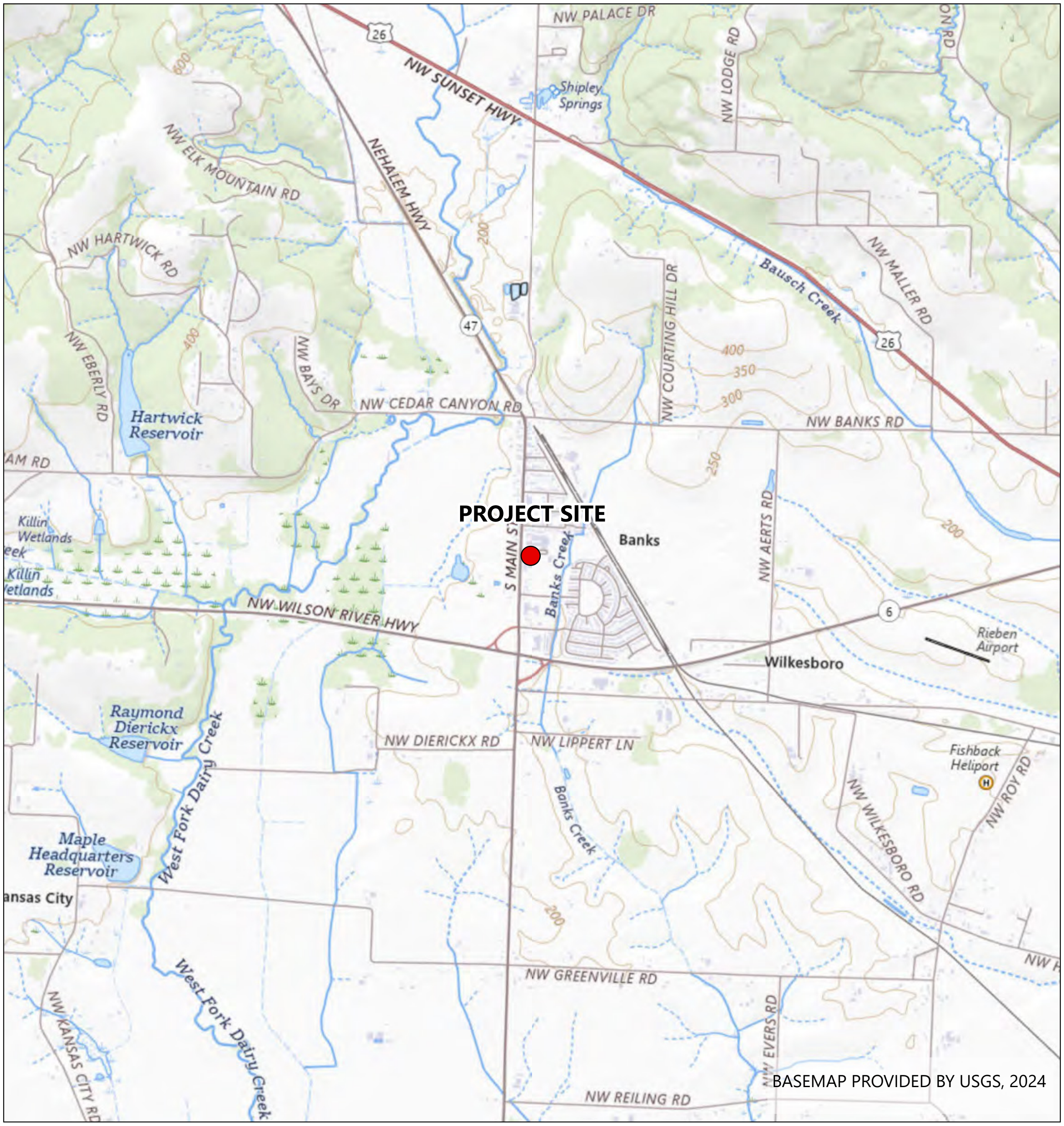
Montgomery, J., and Boulanger, R. W., 2016, Effects of spatial variability on liquefaction-induced settlement and lateral spreading, *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 143.

Robertson, P., 2009, Interpretation of cone penetration tests – A unified approach, *Canadian Geotechnical Journal*, vol. 46, pp. 1,337–1,355. 10.1139/T09-065.

Wells, R. E., Haugerud, R. A., Niem, A. R., Niem, W. A., Ma, L., Evarts, R. C., O'Connor, J. E., Madin, I. P., Sherrod, D. R., Beeson, M. H., et al., 2020, Geologic map of the greater Portland metropolitan area and surrounding region, Oregon and Washington: U.S. Geological Survey Scientific Investigations Map 3443, pamphlet 55 pp., 2 sheets, scale 1:63,360, https://doi.org/10.3133/sim3443.

U.S. Geological Survey (USGS), 2018, Unified hazard tool, conterminous U.S., accessed October 7, 2024, from USGS website: https://earthquake.usgs.gov/nshmp/disagg

Yoshimine, M., Nishizaki, H., Amano, K., and Hosono, Y., 2006, Flow deformation of liquefied sand under constant shear load and its application to analysis of flow slide of infinite slope, *Soil Dynamics and Earthquake Engineering*, vol. 26, nos. 2-4, pp. 253–264



N

0 0.5 1

1 INCH = 0.5 MILES

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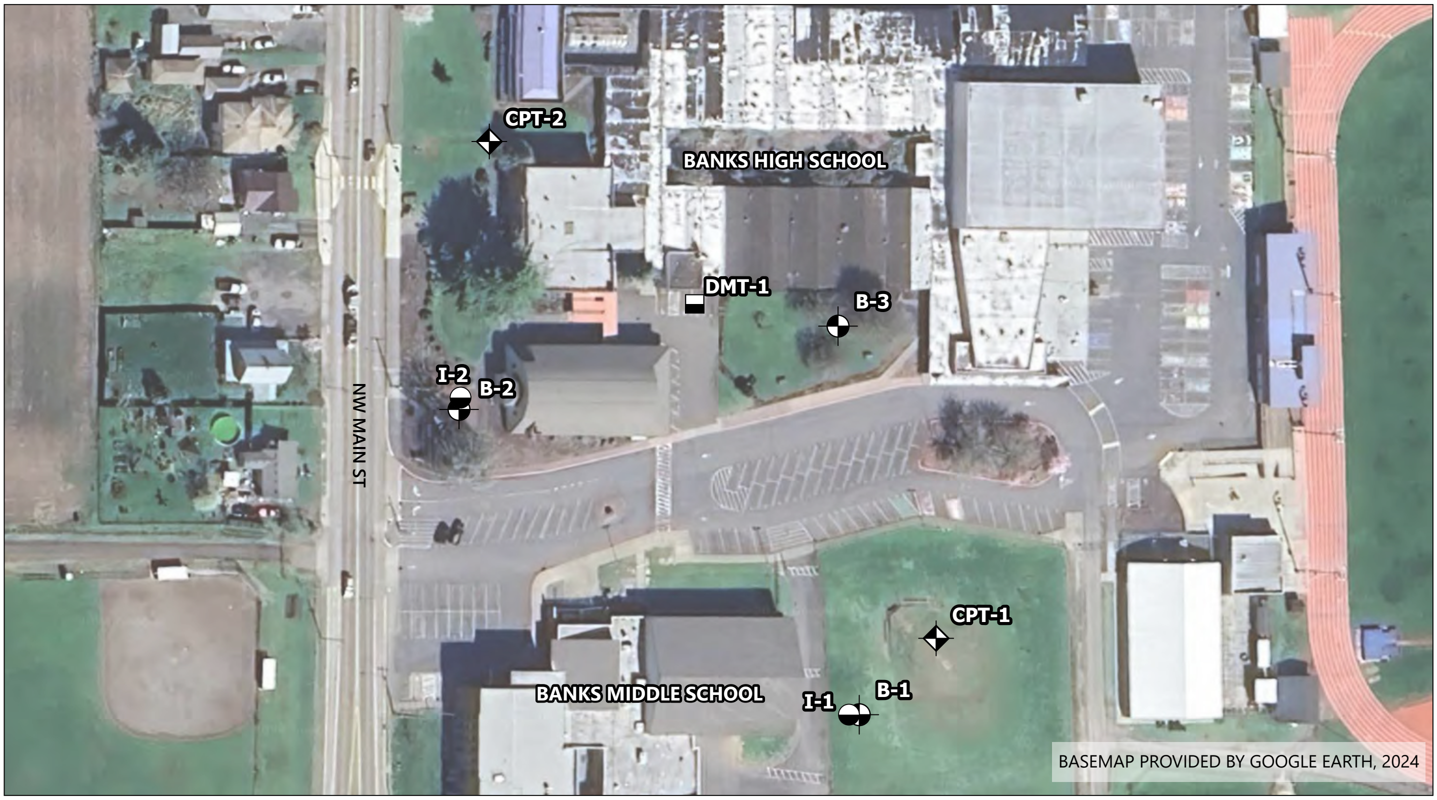
BANKS MIDDLE AND HIGH SCHOOL IMPROVEMENTS

VICINITY MAP

NOV. 2024 JOB NO. 6988-A FIG. 1



LEGEND:

- APPROXIMATE LOCATION OF BORING COMPLETED BY GRI
- APPROXIMATE LOCATION OF CONE PENETRATION TEST COMPLETED BY GRI
- APPROXIMATE LOCATION OF DILATOMETER TEST COMPLETED BY GRI
- APPROXIMATE LOCATION OF INFILTRATION TEST COMPLETED BY GRI

N

0 100 200

1 INCH = 100 FEET

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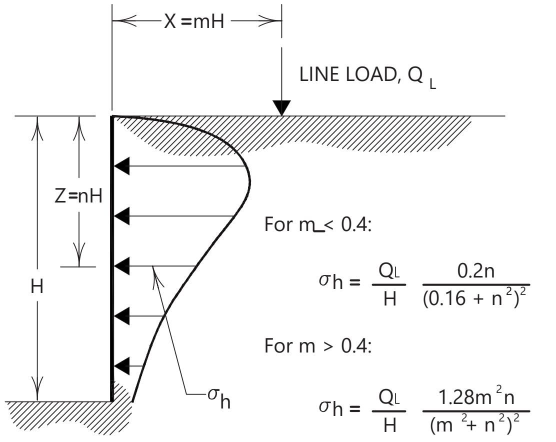
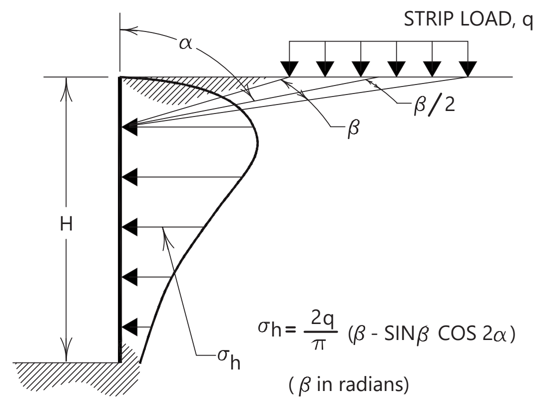
SITE PLAN

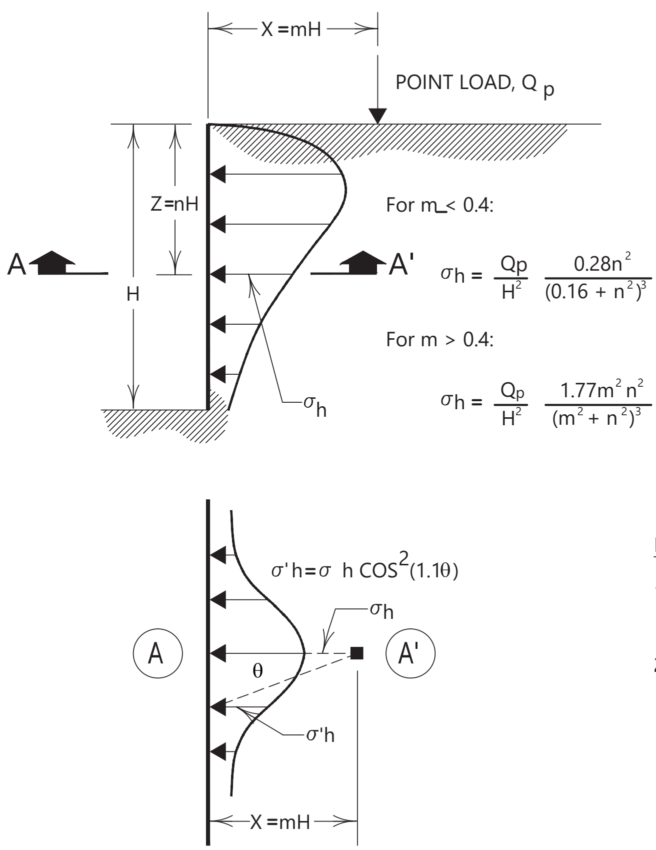
NOV. 2024 | JOB NO. 6988-A | FIG. 2



LINE LOAD PARALLEL TO WALL



STRIP LOAD PARALLEL TO WALL



DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

NOTES:

1. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.

2. LATERAL PRESSURES FROM ANY COMBINATION OF THE ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.

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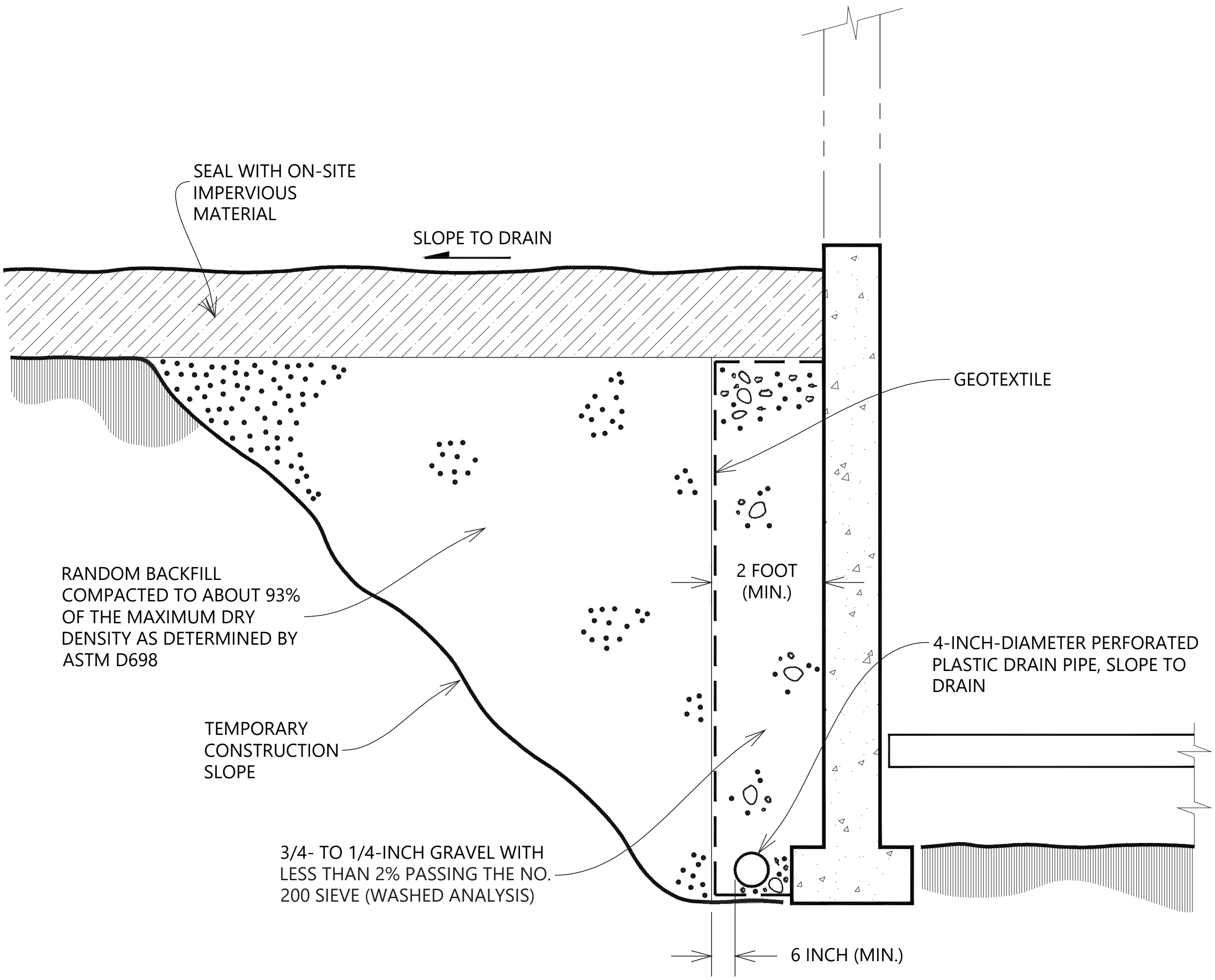
SURCHARGE-INDUCED LATERAL PRESSURE

NOV. 2024 | JOB NO. 6988-A | FIG. 3



NOT TO SCALE

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TYPICAL SUBDRAINAGE DETAIL

NOV. 2024 JOB NO. 6988-A FIG. 4

APPENDIX A

Field Explorations and Laboratory Testing

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APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

A.1 FIELD EXPLORATIONS

A.1.1 General

Subsurface explorations and conditions at the site were investigated between August 21 and 23, 2024, with three borings, designated B-1 through B-3, two cone penetration test (CPT) probes, designated CPT-1 and CPT-2, and one dilatometer test (DMT) sounding, designated DMT-1. The approximate locations of the field explorations completed for this investigation are shown on the Site Plan, Figure 2. Logs of the borings, CPT probes, and DMT sounding are provided on Figures 1A through 3A, Figures 4A through 7A, and Figure 8A, respectively. The field exploration work was coordinated and documented by an experienced member of GRI's geotechnical engineering staff, who maintained a log of the materials and conditions disclosed during the course of work.

A.1.2 Borings

Borings B-1 through B-3 were advanced to depths ranging between about 31.5 feet and 101.5 feet with mud-rotary drilling techniques using a truck-mounted drill rig provided and operated by Western States Soil Conservation, Inc. of Hubbard, Oregon. Disturbed soil samples were obtained from the borings at 2.5-foot intervals of depth in the upper 15 feet and at 5-foot intervals below this depth. Disturbed soil samples were obtained using a 2-inch outside-diameter standard split-spoon sampler. Standard Penetration Tests (SPTs) were conducted by driving the sampler into the soil a distance of 18 inches using a 140-pound hammer dropped 30 inches. The number of blows required to drive the standard split-spoon sampler the last 12 inches is known as the Standard Penetration Resistance, or SPT N-value. The SPT N-values provide a measure of relative density of granular soils and the relative consistency of cohesive soils. Samples obtained from the borings were placed in airtight bags and returned to our laboratory for further classification and testing.

In addition, relatively undisturbed samples were collected by pushing a 3-inch outside-diameter Shelby tube into the undisturbed soil a maximum distance of 24 inches using the hydraulic ram of the drill rig. The soil exposed at the end of the Shelby tube was examined and classified in the field. After classification, the tubes were sealed with rubber caps and returned to our laboratory for further examination and testing.

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Logs of the borings are provided on Figures 1A through 3A. The log presents a summary of the various types of materials encountered in the boring and notes the depth at which the materials and/or characteristics of the materials change. To the right of the summary, the numbers and types of samples taken during the drilling operation are indicated. Farther to the right, SPT N-values, moisture contents, Atterberg limits and percent material passing the No. 200 sieve are shown graphically. The terms used to describe the materials encountered in the borings are defined in Table 2A and the attached legend.

A.1.3 Cone Penetration Test Probes

Two CPT probes, designated CPT-1 and CPT-2, were advanced to depths of about 62.5 feet and 87.6 feet using a truck-mounted CPT rig provided and operated by Oregon Geotechnical Explorations, Inc., of Kaiser, Oregon. During a CPT, a steel cone is forced vertically into the soil at a constant rate of penetration. The force required to cause penetration at a constant rate can be related to the bearing capacity of the soil immediately surrounding the point of the penetrometer cone. This force is measured and recorded every 2 inches. In addition to the cone measurements, measurements are obtained of the magnitude of force required to force a friction sleeve attached above the cone through the soil. The force required to move the friction sleeve can be related to the undrained shear strength of fine-grained soils. The dimensionless ratio of sleeve friction to point-bearing capacity provides an indicator of the type of soil penetrated. The cone penetration resistance and sleeve friction can be used to evaluate the relative consistency of cohesionless and cohesive soils, respectively. In addition, a piezometer fitted between the cone and the sleeve measures changes in water pressure as the probe is advanced and can also be used to measure the depth to the top of the groundwater table. The probe was also operated using an accelerometer fitted to it, which allows measurement of the arrival time of shear waves from impulses generated at the ground surface. This allows the calculation of shear-wave velocities for the surrounding soil profile.

Logs of the CPT probes and shear-wave velocity measurements recorded in CPT-1 and CPT-2 are provided on Figures 4A through 7A. The CPT logs present a graphical summary of the tip resistance, local (sleeve) friction, friction ratio, pore pressure, and soil behavior type index. The terms used to describe the soils encountered in the probe are defined in Table 3A.

A.1.4 Dilatometer Test Sounding

One DMT sounding, designated DMT-1, was advanced to a depth of about 53.2 feet using a truck-mounted rig provided and operated by Oregon Geotechnical Explorations, Inc., of Kaiser, Oregon. DMT soundings provide additional geotechnical information to characterize the subsurface materials. The DMT is performed by pushing a blade-shaped instrument into the soil. The blade is equipped with an expandable membrane on one side that is pressurized until the membrane moves horizontally into the surrounding soil. Readings of pressure required to move the membrane to a point that is flush with the blade (P_0) and to a point 1.1 millimeters into the surrounding soil (P_1) are recorded. The test sequence was performed at 8-inch intervals to obtain a comprehensive soil profile. A material index (I_D), a horizontal stress index (K_D), and a dilatometer modulus (E_D) are obtained directly from the dilatometer data. The constrained modulus (M) is then obtained from the DMT data. The terms used to describe the materials encountered in the DMT are defined in Table 4A.

DMT results are summarized on Figure 8A. The results show the dilatometer pressure readings (P_0, P_1) and three dilatometer-derived parameters: horizontal stress index (K_D), material index (I_D), and constrained modulus (M).

A.1.5 Infiltration Testing

Falling-head infiltration testing was completed at the site on August 23, 2024, in general conformance with the City of Portland 2020 *Stormwater Management Manual* using the encased falling-head method outlined in Section 2.3.2 of the manual. The test locations were designated I-1 and I-2 and completed in shallow boreholes at depths of about 5.5 feet below existing site grades using hand-auger techniques. The borehole was augured to the depth of the infiltration test and withdrawn. A 4-inch-diameter PVC pipe was seated firmly into the base of the borehole and filled with water to a height of approximately 1 foot above the base of the hole. After soaking for a minimum of one hour, infiltration testing was conducted by reestablishing the water level in the pipe to the target height and recording the drop in water level over one hour or until the water completely drained, whichever occurred first. The average unfactored, field-measured infiltration rates are provided in Table 1A, below.

Table 1A: INFILTRATION TEST RESULTS

Test No.	Depth of Infiltration Test, feet	Average Field Infiltration Rate, inches/hour	Soil Classification
I-1	5	<0.25	SILT, trace sand; ML; brown; low plasticity; moist; fine sand
I-2	5	<0.25	SILT, trace sand; ML; brown; low plasticity; moist; fine sand

A.2 LABORATORY TESTING

A.2.1 General

The samples obtained from the borings were examined in our laboratory, where the physical characteristics of the samples were noted and the field classifications modified where necessary. At the time of classification, the natural moisture content of each sample was determined. Additional testing included dry oven weight, Atterberg limits, one-dimensional consolidation, grain-size analyses, and cyclic direct simple shear (CDSS) testing. A summary of the laboratory test results is provided on Table 5A. The following sections describe the testing program in more detail.

A.2.2 Natural Moisture Contents

Natural moisture content determinations were made in conformance with ASTM International (ASTM) D2216. The results are summarized on Figures 1A through 3A and in Table 5A.

A.2.3 Grain-Size Analysis – Washed Sieve Method

To assist in classification of the soils, samples of known dry weight were washed over a No. 200 sieve. The material retained on the sieve is oven-dried and weighed. The percentage of material passing the No. 200 sieve is then calculated. The results are summarized on Figures 1A through 3A and in Table 5A.

A.2.4 Torvane Shear Strength

The approximate undrained shear strength of the fine-grained soils was determined using a Torvane shear device. The Torvane is a hand-held apparatus with vanes that are inserted into the soil. The torque required to fail the soil in shear around the vanes is measured using a calibrated spring. The results of the Torvane shear-strength tests are summarized on Figures 1A through 3A and in Table 5A.

A.2.5 Undisturbed Unit Weight

The unit weight, or density, of the undisturbed soil samples was determined in the laboratory in conformance with ASTM D2937. The results are summarized on Figures 1A through 3A and in Table 5A.

A.2.6 Atterberg Limits

Atterberg-limits testing was performed for selected samples of fine-grained soils in conformance with ASTM D4318. The test results are summarized on the Plasticity Charts, Figures 9A and 10A, on Figures 1A through 3A, and in Table 5A.

A.2.7 One-Dimensional Consolidation

Four one-dimensional consolidation tests were performed in conformance with ASTM D2435 on relatively undisturbed soil samples extruded from Shelby tubes. These tests provide data on the compressibility of underlying fine-grained soils, which are necessary for settlement studies. The results of the one-dimensional consolidation tests are summarized in Figures 11A through 14A. The initial dry unit weights and moisture contents of the samples are shown on the figures.

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A.2.8 Monotonic Direct Simple Shear Testing

A single-stage, consolidated, undrained monotonic direct-simple shear (MDSS) test with pore pressure measurements was performed in conformance with ASTM D6528 on a relatively undisturbed soil sample extruded from Shelby tubes. The MDSS test provides data on the peak shear strength and associated shear strain of the fine-grained soils selected for testing. Results of the testing are included on Figure 15A.

A.2.9 Cyclic Direct Simple Shear Testing

A strain-controlled CDSS test was performed in conformance with standard of practice and ASTM D6528 on relatively undisturbed soil samples extruded from a Shelby tube. The test provides data on the static and cyclic shear resistance, degradation potential, and post-cyclic behavior of the underlying fine-grained soils, which are necessary for seismic studies. Results of the testing are included on Figure 16A.

Table 2A

GUIDELINES FOR DESCRIPTION OF SOIL[1]

Description of Relative Density for Cohesionless (Coarse-Grained) Soils

Relative Density	Standard Penetration Resistance (N-values) blows/foot (ft)	3-inch Sampler, 140-lb hammer approx. N-Value (blows/ft)[2]	3-inch Sampler, 300-lb hammer approx. N-Value (blows/ft)[1]
Very Loose	0 - 4	0 – 10	0 – 5
Loose	4 - 10	10 – 24	5 – 11
Medium Dense	10 - 30	24 – 73	11 – 34
Dense	30 - 50	73 – 122	34 – 57
Very Dense	over 50	over 122	over 57

Description of Relative Consistency for Cohesive (Fine-Grained) Soils

Relative Consistency	Standard Penetration Resistance (N-values) blows/ft	3-inch Sampler, 140 lb hammer approx. N-Value (blows/ft)[1]	3-inch Sampler, 300 lb hammer approx. N-Value (blows/ft)[2]	Torvane or Undrained Shear Strength, tsf
Very Soft	0 - 2	0 – 3	0 – 1	less than 0.125
Soft	2 - 4	3 – 6	1 – 3	0.125 - 0.25
Medium Stiff	4 - 8	6 – 12	3 – 6	0.25 - 0.50
Stiff	8 - 15	12 – 23	6 – 11	0.50 - 1.0
Very Stiff	15 - 30	23 – 46	11 – 22	1.0 - 2.0
Hard	30 – 60	46 – 92	22 – 43	over 2.0
Very Hard	over 60	over 92	over 43	

Grain-Size Classification

Boulders:
>12 inches

Cobbles:
3 inches - 12 inches

Gravel:
¼ inch - ¾ inch (fine)
¾ inch - 3 inches (coarse)

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

Silt/Clay:
Pass No. 200 sieve

Modifier for Subclassification

Adjective	Primary Constituent SAND or GRAVEL	Primary Constituent SILT or CLAY
	Percentage of Other Material (By Weight)	
trace:	<15 (sand, gravel)	<15 (sand, gravel)
some:	15 - 30 (sand, gravel)	15 - 30 (sand, gravel)
sandy, gravelly:	30 - 50 (sand, gravel)	30 - 50 (sand, gravel)
trace:	<5 (silt, clay)	*Relationship of clay and silt determined by plasticity index test*
some:	5 - 12 (silt, clay)	
silty, clayey:	12 - 50 (silt, clay)	

1. Soil descriptions are developed using visual-manual procedures (ASTM D2488) and generally follow ODOT Geotechnical Design Manual (Chapter 5) guidelines.
2. Oversized sampler (OD = 3 inches, ID = 2.4 inches) blow counts converted to SPT N-Value using equations provided by Burmister, D.M., 1948, The importance and practical use of relative density in soil mechanics: Proceedings of ASTM, v. 48:1249.

Table 3A

SOIL CHARACTERIZATION
BASED ON CONE PENETRATION TEST (CPT)

Description of Relative Consistency for Cohesive (Fine-Grained) Soils

Cone Tip Resistance, tsf	Relative Consistency
<5	Very Soft
5 - 15	Soft to Medium Stiff
15 - 30	Stiff
30 - 60	Very Stiff
>60	Hard

Description of Relative Density for Cohesionless (Coarse-Grained) Soils

Cone Tip Resistance, tsf	Relative Density
<20	Very Loose
20 - 40	Loose
40 - 120	Medium Dense
120 - 200	Dense
>200	Very Dense

Reference

Kulhawy, F. H., and Mayne, P. W., 1990, Manual on Estimating Soil Properties for Foundation Design, Electric Power Research Institute, EL-6800.

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Table 4A

SOIL CHARACTERIZATION
BASED ON MARCHETTI FLAT PLATE DILATOMETER TEST (DMT)

Description of Relative Consistency for Cohesive (Fine-Grained) Soils

	Soil Type[a]	
	CH, CL	ML, MH
	DMT Constrained Modulus (M_{DMT}), tsf	
Relative Consistency	I_D[b] < 0.6	0.6 < I_D[b] < 1.8
Very Soft	0 - 30	0 - 50
Soft	30 - 60	50 - 100
Medium Stiff	60 - 100	100 - 200
Stiff	100 - 175	200 - 375
Very Stiff	175 +	375 +

Description of Relative Density for Cohesionless (Coarse-Grained) Soils

	Soil Type[a]	
	SM, SC	SP, SW
	DMT Constrained Modulus (M_{DMT}), tsf	
Relative Density	1.8 < I_D[b] < 3.3	3.3 < I_D[b]
Very Loose	0 - 75	0 - 100
Loose	75 - 150	100 - 200
Medium Dense	150 - 300	200 - 425
Dense	300 - 550	425 - 850
Very Dense	550 +	850 +

Notes:

a. Unified Soil Classification System
b. I_D = Material Index

Table 5A

SUMMARY OF LABORATORY RESULTS

Sample Information						Atterberg Limits			
Location	Sample	Depth, ft	Elevation, ft	Moisture Content, %	Dry Unit Weight, pcf	Liquid Limit, %	Plasticity Index, %	Fines Content, %	Soil Type
B-1	S-1	2.5	--	34	--	--	--	--	SILT
	S-2	5.0	--	41	--	--	--	97	SILT
	S-3	7.5	--	38	--	39	12	--	SILT
		8.3	--	38	85	--	--	98	SILT
	S-4	9.5	--	41	--	--	--	97	SILT
	S-6	14.5	--	41	--	39	15	--	Silty CLAY
	S-7	20.0	--	37	--	40	16	--	Silty CLAY
		20.8	--	37	85	--	--	--	Silty CLAY
	S-7	21.2	--	35	88	--	--	99	Silty CLAY
	S-8	22.0	--	34	--	--	--	99	Silty CLAY
	S-10	27.0	--	30	--	--	--	--	CLAY
	S-11	30.0	--	28	--	51	33	97	CLAY
B-2	S-1	2.5	--	50	--	--	--	--	FILL
	S-2	5.0	--	37	--	--	--	--	SILT
	S-3	7.5	--	40	--	--	--	97	SILT
	S-5	12.0	--	39	--	--	--	95	SILT
	S-6	15.0	--	34	--	34	6	--	SILT
		15.5	--	34	89	--	--	93	SILT
	S-7	17.0	--	41	--	--	--	--	SILT
	S-8	20.0	--	35	--	41	14	95	SILT
	S-10	27.0	--	27	--	--	--	97	CLAY
	S-12	31.5	--	28	--	--	--	--	CLAY
B-3	S-1	2.5	--	31	--	--	--	--	SILT
	S-2	5.0	--	38	--	--	--	99	SILT
	S-3	7.5	--	37	--	--	--	--	SILT
	S-5	12.0	--	41	--	--	--	98	SILT
	S-6	15.0	--	38	--	--	--	--	SILT
	S-7	20.0	--	38	--	--	--	98	Silty CLAY
	S-8	22.5	--	29	--	30	13	--	Silty CLAY
		23.0	--	29	95	--	--	--	Silty CLAY
		23.8	--	28	97	--	--	97	Silty CLAY
	S-9	24.5	--	26	--	33	18	97	Silty CLAY
	S-11	32.0	--	27	--	--	--	--	CLAY
	S-12	35.0	--	34	--	79	52	--	CLAY
	S-14	42.0	--	47	--	--	--	--	CLAY
	S-15	45.0	--	35	--	48	26	78	Silty CLAY
	S-17	52.0	--	40	--	--	--	78	Silty CLAY
	S-18	55.0	--	36	--	--	--	66	Sandy SILT
	S-19	60.0	--	39	--	--	--	33	Silty SAND
	S-21	67.0	--	43	--	--	--	--	Sandy CLAY

Page 1 of 2

Table 5A

SUMMARY OF LABORATORY RESULTS

Sample Information						Atterberg Limits			
Location	Sample	Depth, ft	Elevation, ft	Moisture Content, %	Dry Unit Weight, pcf	Liquid Limit, %	Plasticity Index, %	Fines Content, %	Soil Type
B-3	S-22	70.0	--	41	--	--	--	58	Sandy SILT
	S-23	75.0	--	47	--	--	--	--	Sandy SILT
	S-24	80.0	--	39	--	--	--	41	Silty SAND
	S-25	90.0	--	35	--	--	--	--	Silty SAND
	S-26	100.0	--	50	--	--	--	--	CLAY

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Page 2 of 2

BORING AND TEST PIT LOG LEGEND

SOIL SYMBOLS

Symbol	Typical Description
	LANDSCAPE MATERIALS
	FILL
	GRAVEL; clean to some silt, clay, and sand
	Sandy GRAVEL; clean to some silt and clay
	Silty GRAVEL; up to some clay and sand
	Clayey GRAVEL; up to some silt and sand
	SAND; clean to some silt, clay, and gravel
	Gravelly SAND; clean to some silt and clay
	Silty SAND; up to some clay and gravel
	Clayey SAND; up to some silt and gravel
	SILT; up to some clay, sand, and gravel
	Gravelly SILT; up to some clay and sand
	Sandy SILT; up to some clay and gravel
	Clayey SILT; up to some sand and gravel
	CLAY; up to some silt, sand, and gravel
	Gravelly CLAY; up to some silt and sand
	Sandy CLAY; up to some silt and gravel
	Silty CLAY; up to some sand and gravel
	PEAT

BEDROCK SYMBOLS

Symbol	Typical Description
	BASALT
	MUDSTONE
	SILTSTONE
	SANDSTONE

SURFACE MATERIAL SYMBOLS

Symbol	Typical Description
	Asphalt concrete PAVEMENT
	Portland cement concrete PAVEMENT
	Crushed rock BASE COURSE

SAMPLER SYMBOLS

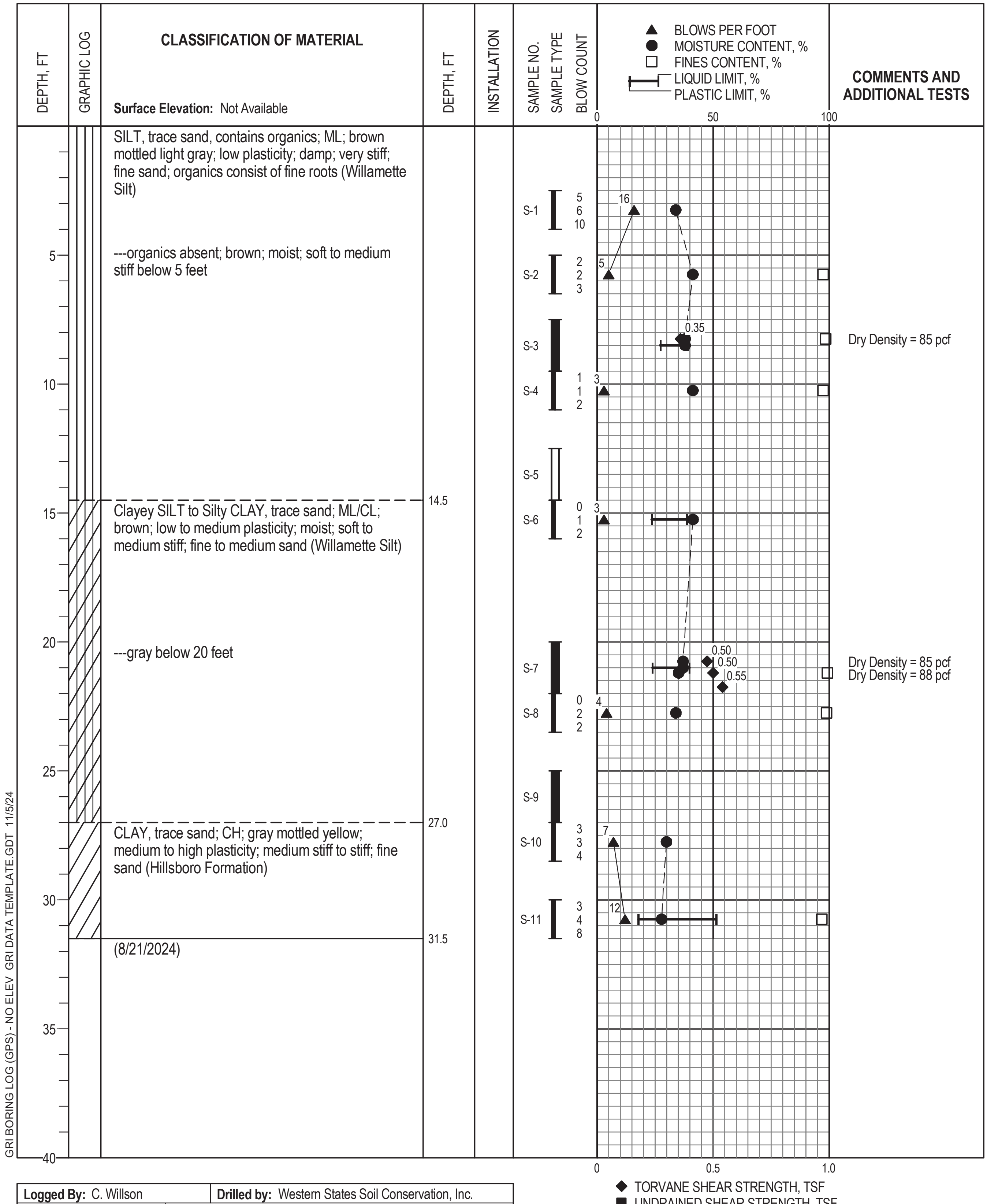
Symbol	Sampler Description
	2.0 in. O.D. split-spoon sampler and Standard Penetration Test with recovery (ASTM D1586)
	Shelby tube sampler with recovery (ASTM D1587)
	3.0 in. O.D. split-spoon sampler with recovery (ASTM D3550)
	Grab Sample
	Rock core sample interval
	Sonic core sample interval
	Push probe sample interval

INSTALLATION SYMBOLS

Symbol	Symbol Description
	Flush-mount monument set in concrete
	Concrete, well casing shown where applicable
	Bentonite seal, well casing shown if applicable
	Filter pack, machine-slotted well casing shown where applicable
	Grout, vibrating-wire transducer cable shown where applicable
P	Vibrating-wire pressure transducer
	1-in.-diameter solid PVC
	1-in.-diameter hand-slotted PVC
	Grout, inclinometer casing shown where applicable

FIELD MEASUREMENTS

Symbol	Typical Description
	Groundwater level during drilling and date measured
	Groundwater level after drilling and date measured
	Rock/sonic core or push probe recovery (%)
	Rock quality designation (RQD, %)

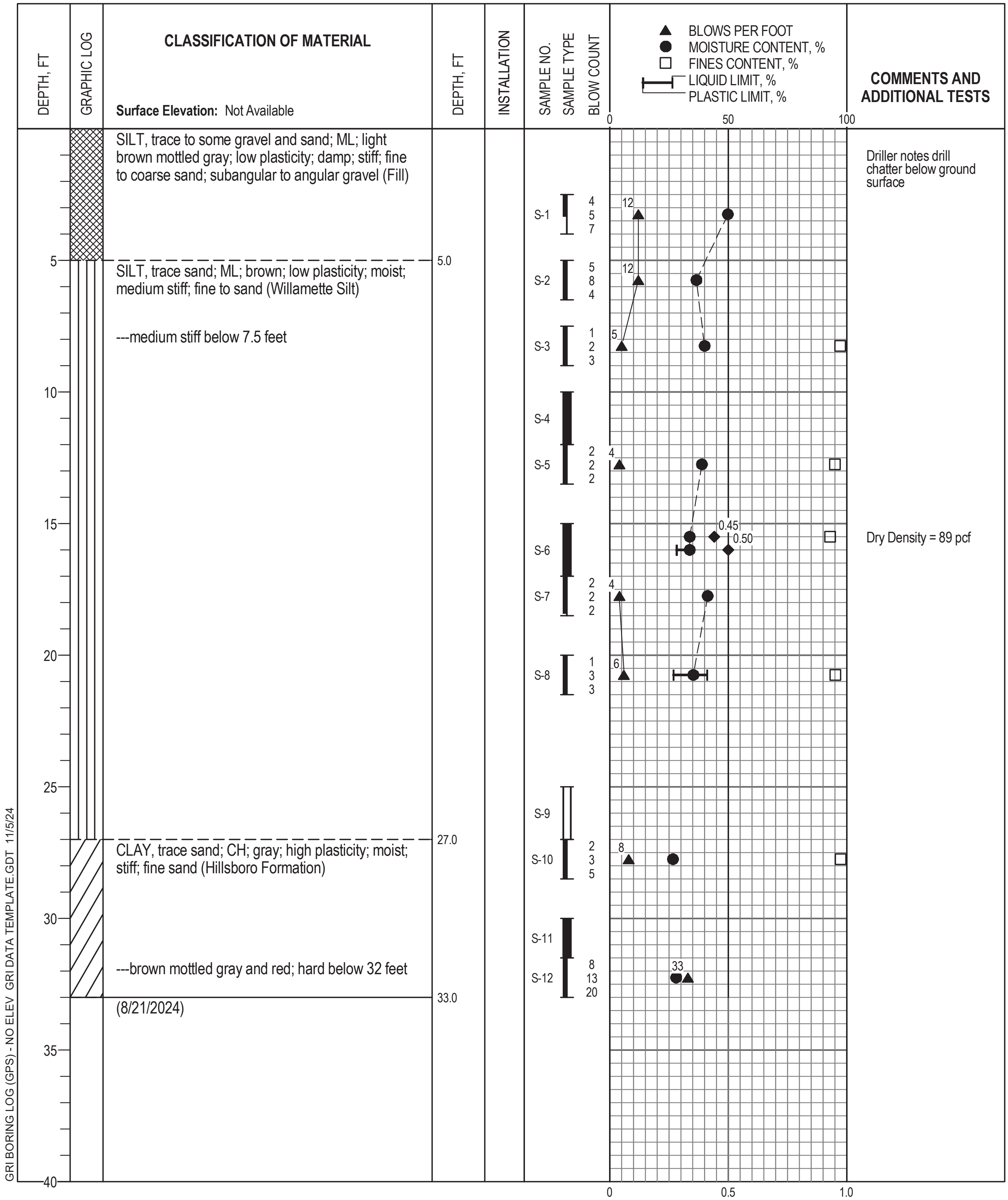


Logged By: C. Willson	**Drilled by:** Western States Soil Conservation, Inc.	
Date Started: 8/21/24	**GPS Coordinates:** 45.614119° N -123.113077° W (WGS84)	
Drilling Method: Mud Rotary **Equipment:** CME 75 HT Truck-Mounted Drill Rig **Hole Diameter:** 5 in.		**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.8
Note: See Legend for Explanation of Symbols		

GRI

BORING B-1

NOV. 2024 JOB NO. 6988-A FIG. 1A



◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF

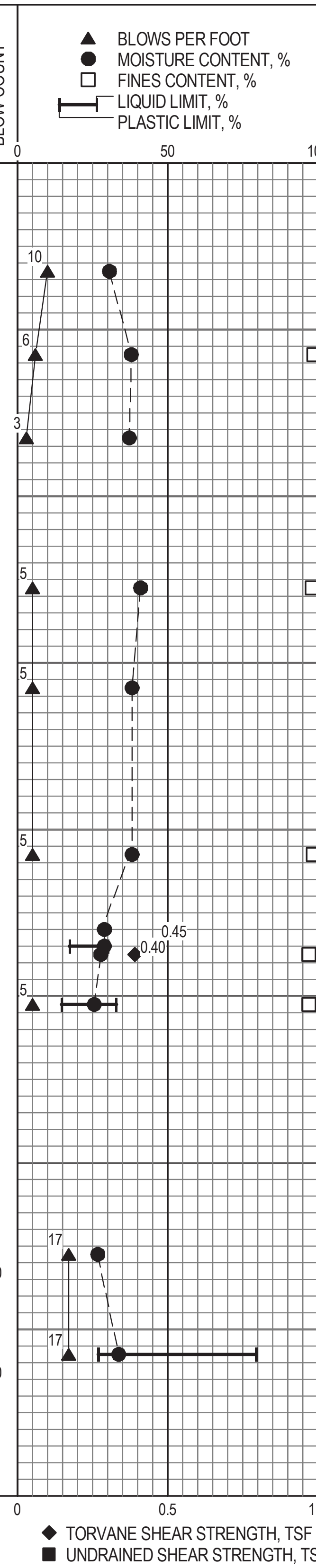
Logged By: C. Willson	Drilled by: Western States Soil Conservation, Inc.
Date Started: 8/21/24	GPS Coordinates: 45.614713° N -123.114201° W (WGS84)
Drilling Method: Mud Rotary Equipment: CME 75 HT Truck-Mounted Drill Rig Hole Diameter: 5 in.	Hammer Type: Auto Hammer Weight: 140 lb Drop: 30 in. Energy Ratio: 0.8
Note: See Legend for Explanation of Symbols	

GRI

BORING B-2

NOV. 2024 JOB NO. 6988-A FIG. 2A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL Surface Elevation: Not Available	DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		SILT, trace sand, contains organics; ML; brown to brown mottled gray; low plasticity; damp; stiff; fine sand; organics consist of fine roots (Willamette Silt)			S-1		5 4 6	
5		---moist; soft to medium stiff; organics absent below 5 feet			S-2		2 3 3	
					S-3		2 1 2	
10					S-4			
					S-5		0 3 2	
15					S-6		2 2 3	∇ 15.4 ft (8/22/2024)
20		Clayey SILT to Silty CLAY, trace sand; ML/CL; gray to gray mottled brown; low to medium plasticity; moist; medium stiff; fine sand (Willamette Silt)	20.0		S-7		2 3 2	
					S-8			Dry Density = 95 pcf Dry Density = 97 pcf
25					S-9		2 2 3	
30		CLAY, trace sand; CH; gray to gray mottled yellow and red; high plasticity; very stiff; fine sand (Hillsboro Formation)	30.0		S-10			
					S-11		4 7 10	
35					S-12		4 7 10	
40								



(CONTINUED NEXT PAGE)

GRI BORING LOG (GPS) - NO ELEV GRI DATA TEMPLATE.GDT 11/5/24

Logged By: C. Willson	**Drilled by:** Western States Soil Conservation, Inc.	
Date Started: 8/22/24	**GPS Coordinates:** 45.614881° N -123.113143° W (WGS84)	
Drilling Method: Mud Rotary **Equipment:** CME 75 HT Truck-Mounted Drill Rig **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.8	
Note: See Legend for Explanation of Symbols		

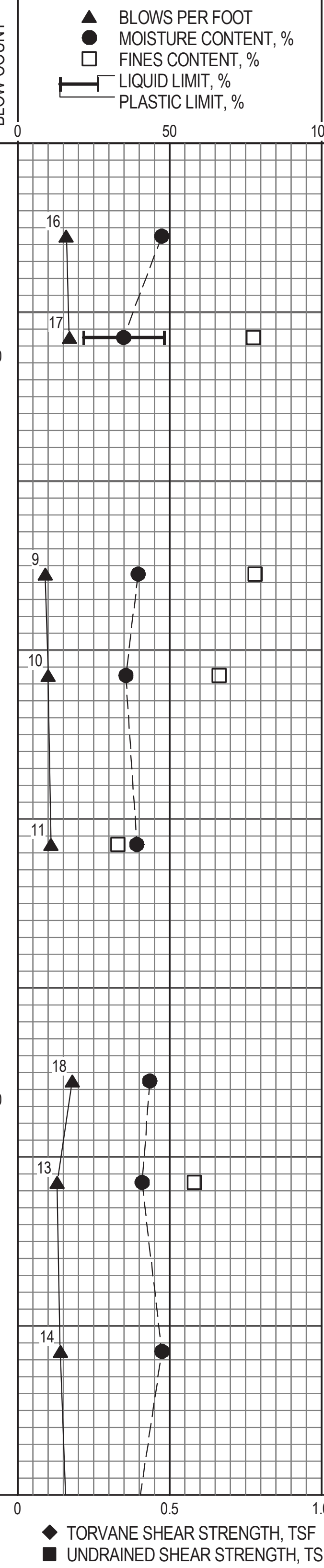
GRI

BORING B-3

NOV. 2024 JOB NO. 6988-A FIG. 3A

Surface Elevation: Not Available

DEPTH, FT	CLASSIFICATION OF MATERIAL	DEPTH, FT	INSTALLATION	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	CLAY, trace sand; CH; gray mottled red; high plasticity; moist; very stiff; fine sand (Hillsboro Formation)	40.0		S-13		
				S-14	6 7 9	
45	---some sand; CL; medium plasticity below 45 feet			S-15	4 7 10	
50			P	S-16		
				S-17	4 4 5	
55	Sandy SILT; ML; light brown; nonplastic to low plasticity; moist; stiff; fine to medium sand (Hillsboro Formation)			S-18	4 4 6	
60	Silty SAND; SM; light brown; nonplastic silt; moist; medium dense; fine to coarse sand (Hillsboro Formation)			S-19	4 4 7	
65				S-20		
	Sandy CLAY, trace gravel; CH; light brown and mottled red; high plasticity; moist; very stiff; subrounded to rounded gravel (Hillsboro Formation)	67.0		S-21	9 8 10	
70	Sandy SILT; ML; light brown mottled red; nonplastic to low plasticity; moist; stiff; fine sand (Hillsboro Formation)	70.0		S-22	6 6 7	
75	---trace gravel; fine to coarse sand; subrounded to rounded gravel below 75 feet			S-23	5 7 7	
80						



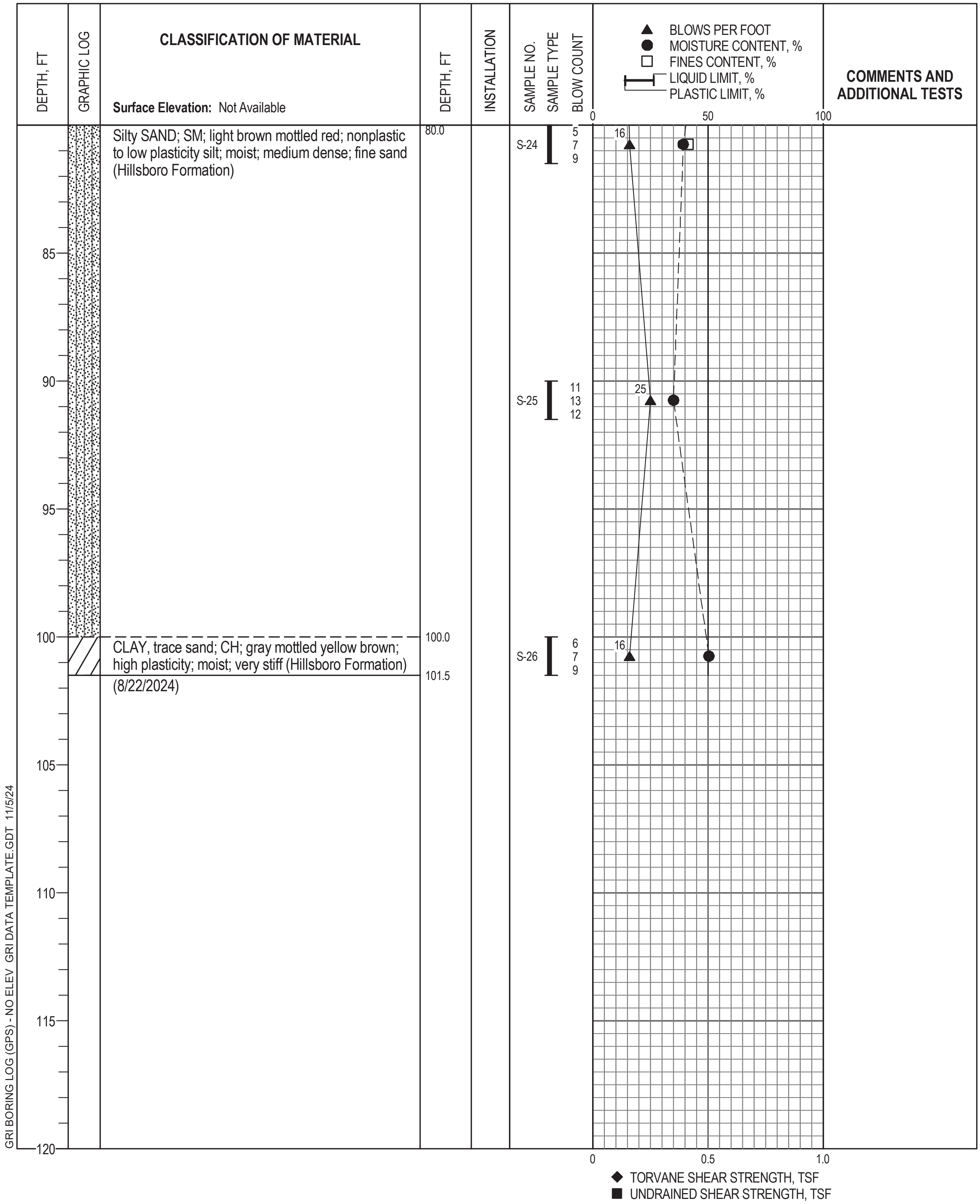
(CONTINUED NEXT PAGE)

GRI BORING LOG (GPS) - NO ELEV GRI DATA TEMPLATE.GDT 11/5/24

GRI

BORING B-3

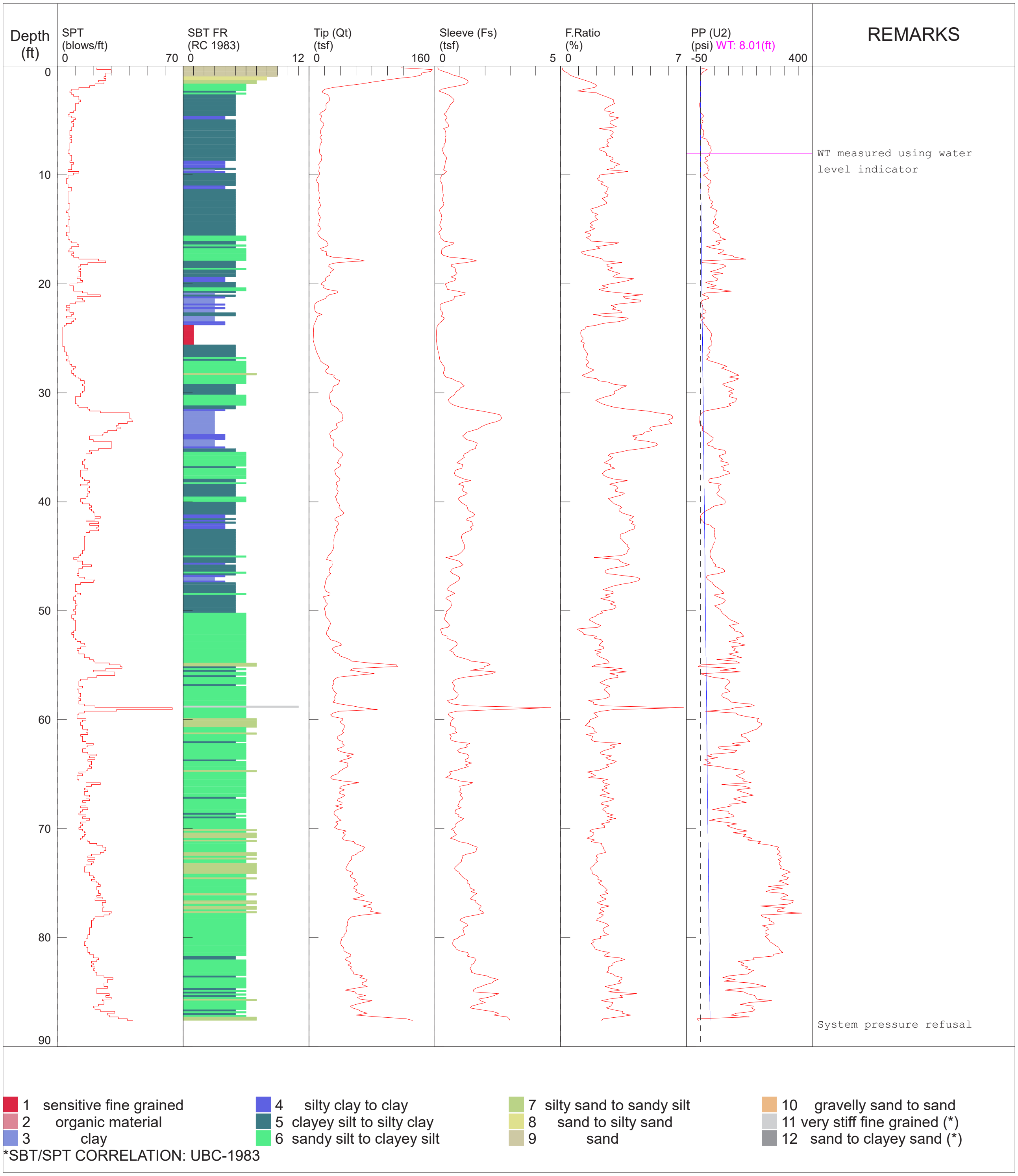
NOV. 2024 | JOB NO. 6988-A | FIG. 3A



GRI

BORING B-3

NOV. 2024 | JOB NO. 6988-A | FIG. 3A

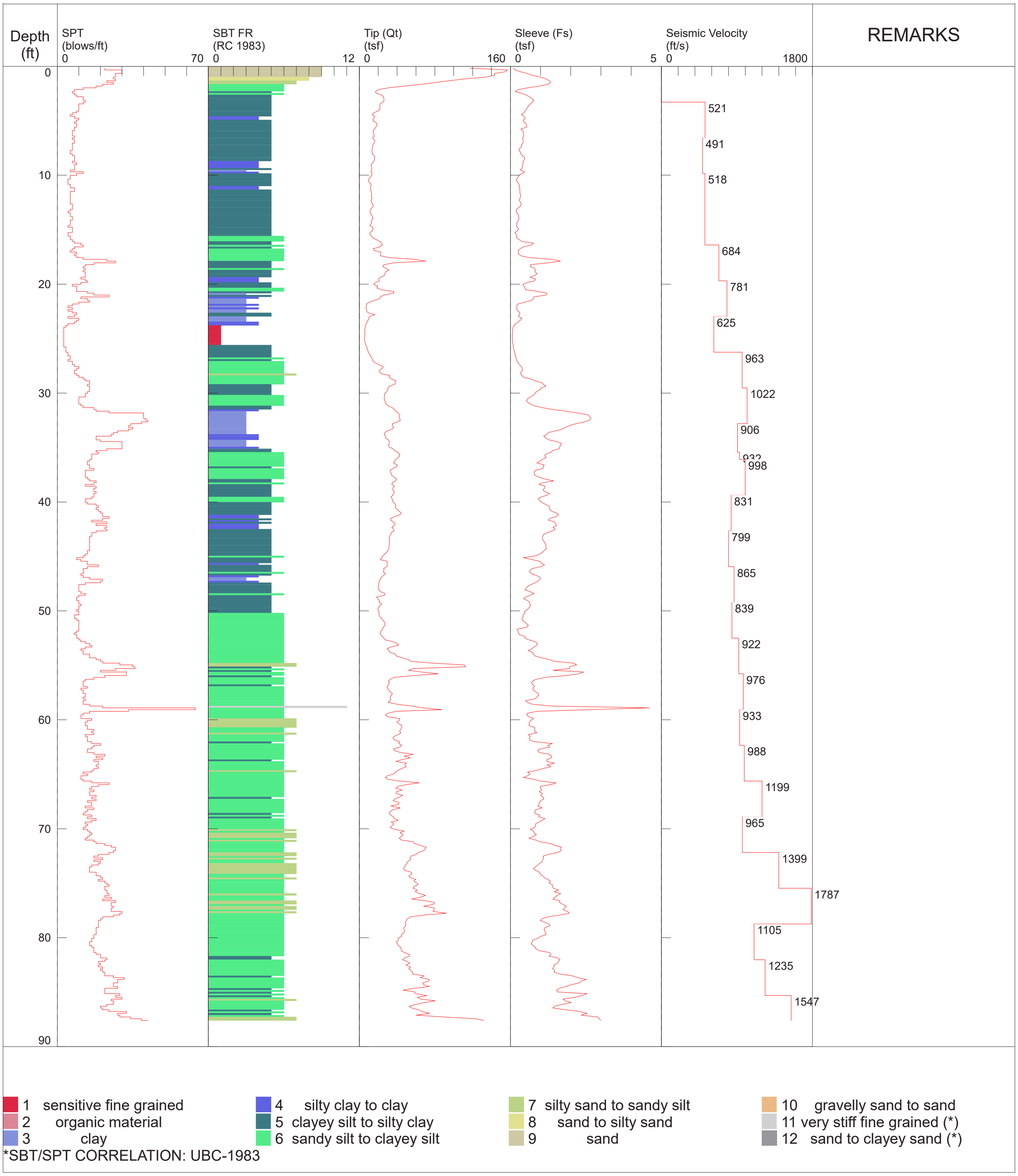


Operator:	Oregon Geotechnical Explorations, Inc.
Cone ID:	DDG1654
Test Date:	8/23/2024
Total Depth:	87.598 Feet

GRI

CONE PENETRATION TEST CPT-1

NOV. 2024 JOB NO. 6988-A FIG. 4A

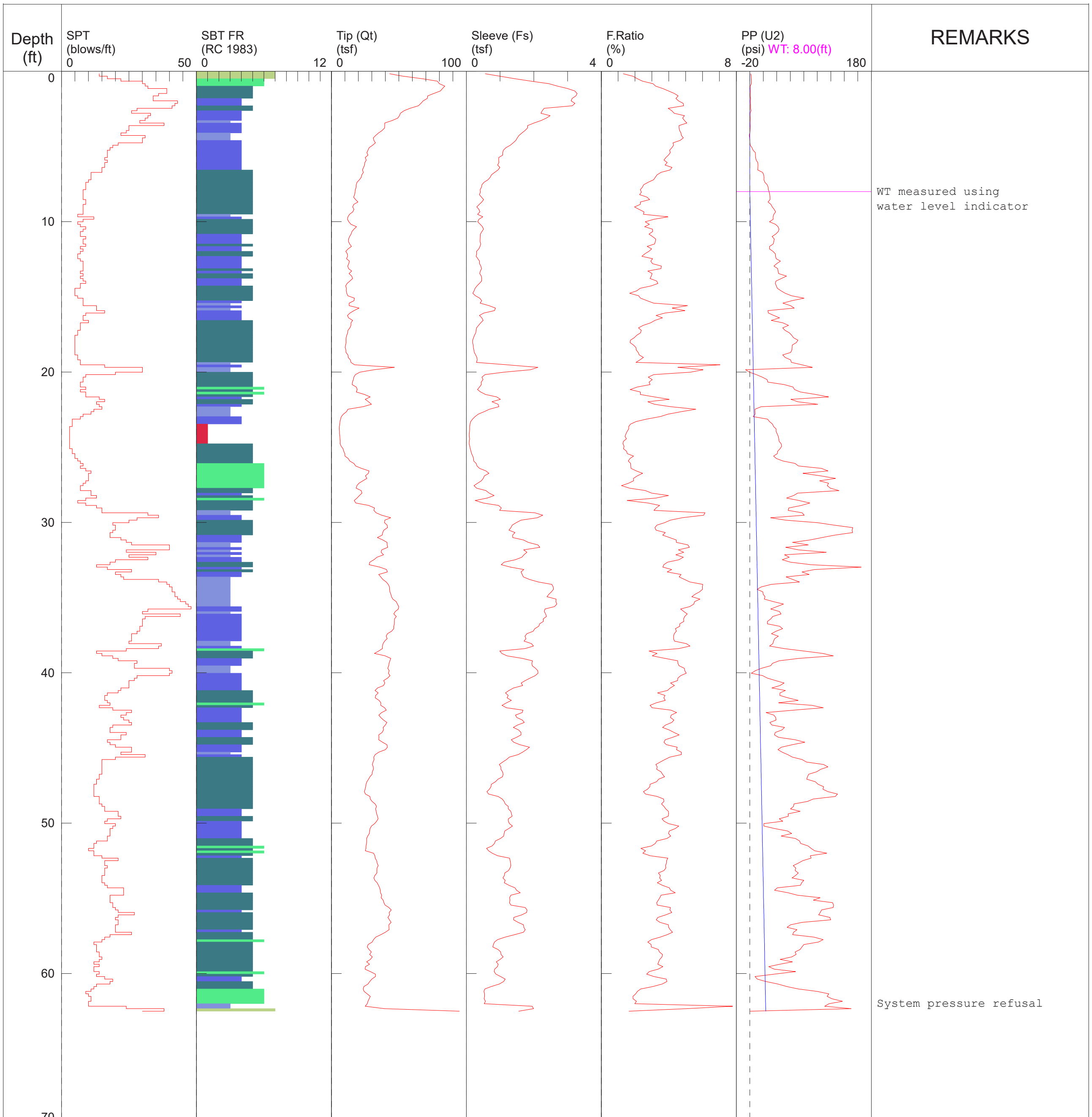


Operator:	Oregon Geotechnical Explorations, Inc.
Cone ID:	DDG1654
Test Date:	8/23/2024
Total Depth:	87.598 Feet

GRI

CONE PENETRATION TEST
CPT-1
(SEISMIC VELOCITY PROFILE)

NOV. 2024 JOB NO. 6988-A FIG. 5A



1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

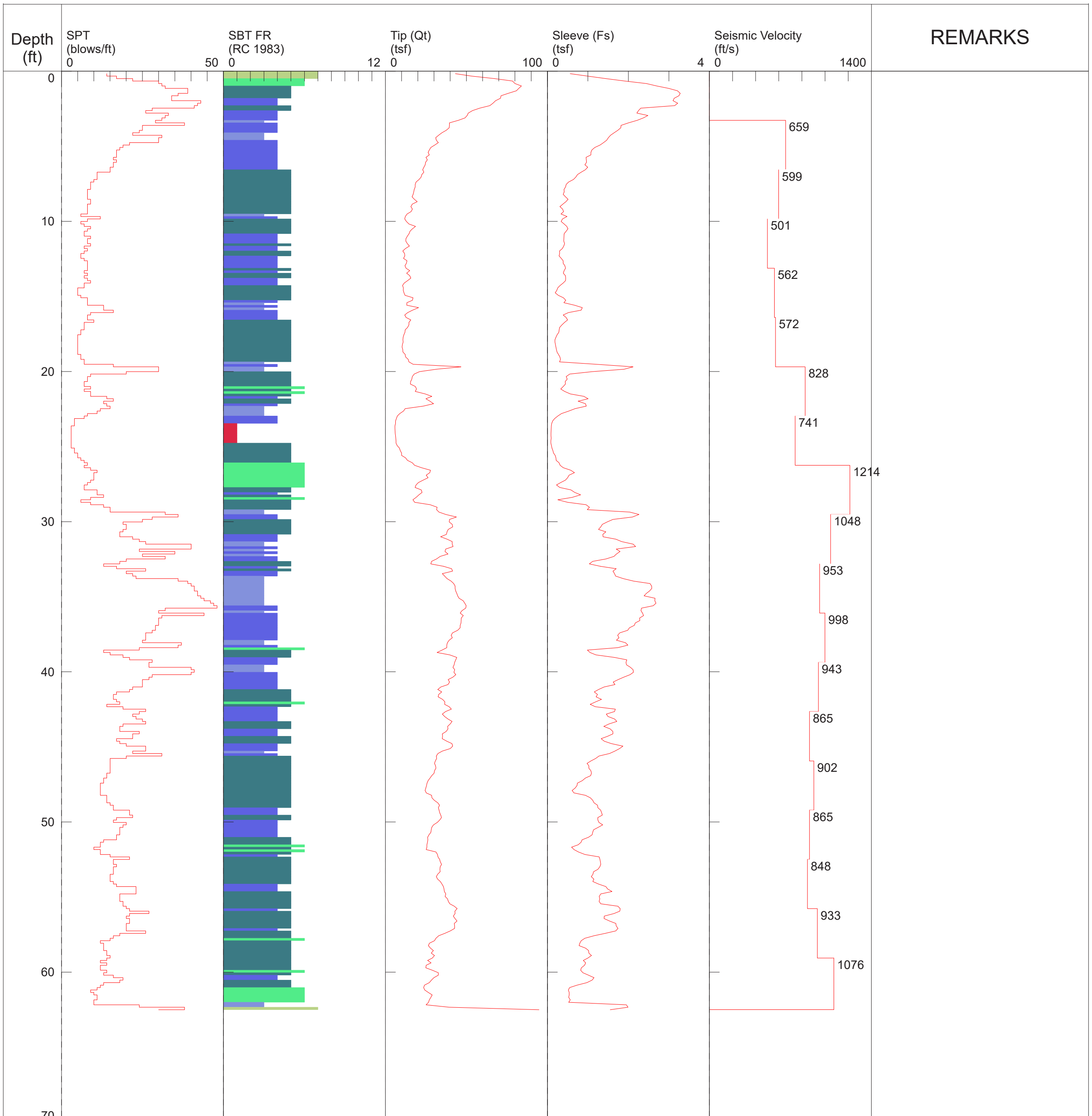
*SBT/SPT CORRELATION: UBC-1983

Operator:	Oregon Geotechnical Explorations, Inc.
Cone ID:	DDG1654
Test Date:	8/23/2024
Total Depth:	62.500 Feet

GRI

CONE PENETRATION TEST CPT-2

NOV. 2024 JOB NO. 6988-A FIG. 6A



1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

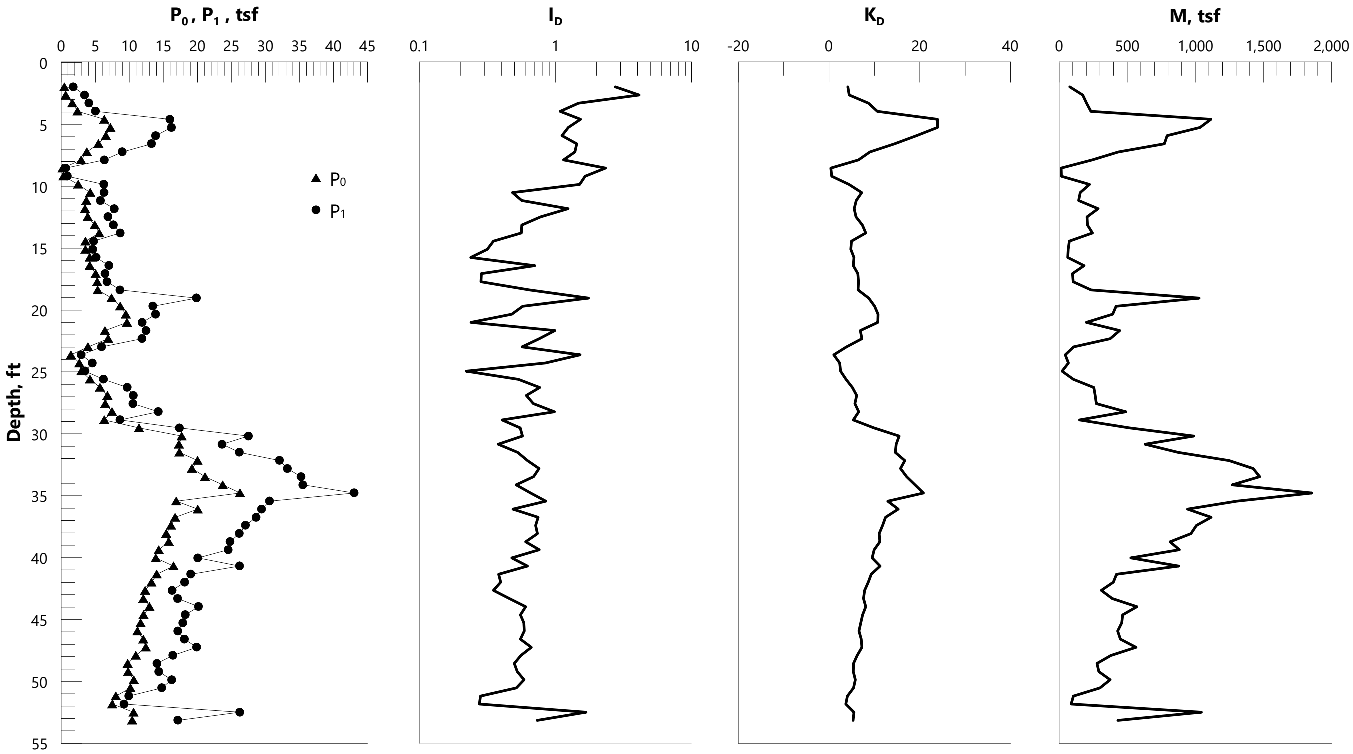
*SBT/SPT CORRELATION: UBC-1983

Operator:	Oregon Geotechnical Explorations, Inc.
Cone ID:	DDG1654
Test Date:	8/23/2024
Total Depth:	62.500 Feet

GRI

CONE PENETRATION TEST
CPT-2
(SEISMIC VELOCITY PROFILE)

NOV. 2024 JOB NO. 6988-A FIG. 7A

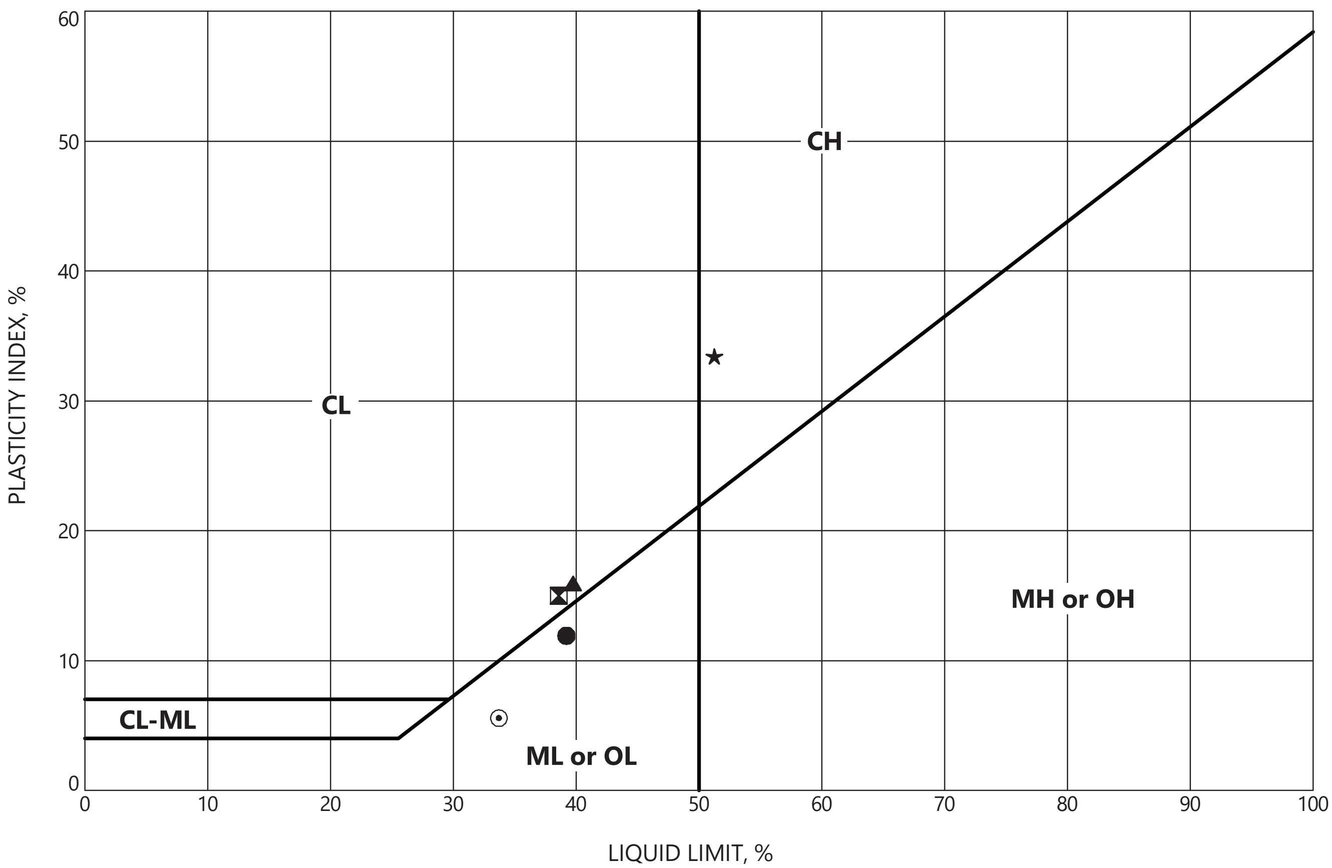


DMT-1

NOV. 2024 JOB NO. 6988-A FIG. 8A

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
ML	INORGANIC CLAYEY SILTS TO VERY FINE SANDS OF SLIGHT PLASTICITY
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
MH	INORGANIC SILTS AND CLAYEY SILT
CH	INORGANIC CLAYS OF HIGH PLASTICITY



	Location	Sample	Depth, ft	Classification	LL	PL	PI	MC, %
●	B-1	S-3	7.5	SILT, trace sand; ML; brown; low plasticity; moist; fine sand (Willamette Silt)	39	27	12	38
⧗	B-1	S-6	14.5	Clayey Silt to Silty CLAY, trace sand; ML/CL; brown; low to medium plasticity; fine to medium sand (Willamette Silt)	39	24	15	41
▲	B-1	S-7	20.0	Clayey Silt to Silty CLAY, trace sand; ML/CL; brown; medium plasticity; fine to medium sand (Willamette Silt)	40	24	16	37
★	B-1	S-11	30.0	CLAY, trace sand; CH; gray mottled yellow; high plasticity; fine to medium sand (Hillsboro Formation)	51	18	33	28
⊙	B-2	S-6	15.0	SILT, trace sand; ML; brown; low plasticity; moist; fine to medium sand (Willamette Silt)	34	28	6	34

ATTERBERG-PLASTICITY 5 PER PAGE GRI DATA TEMPLATE.GDT 11/5/24

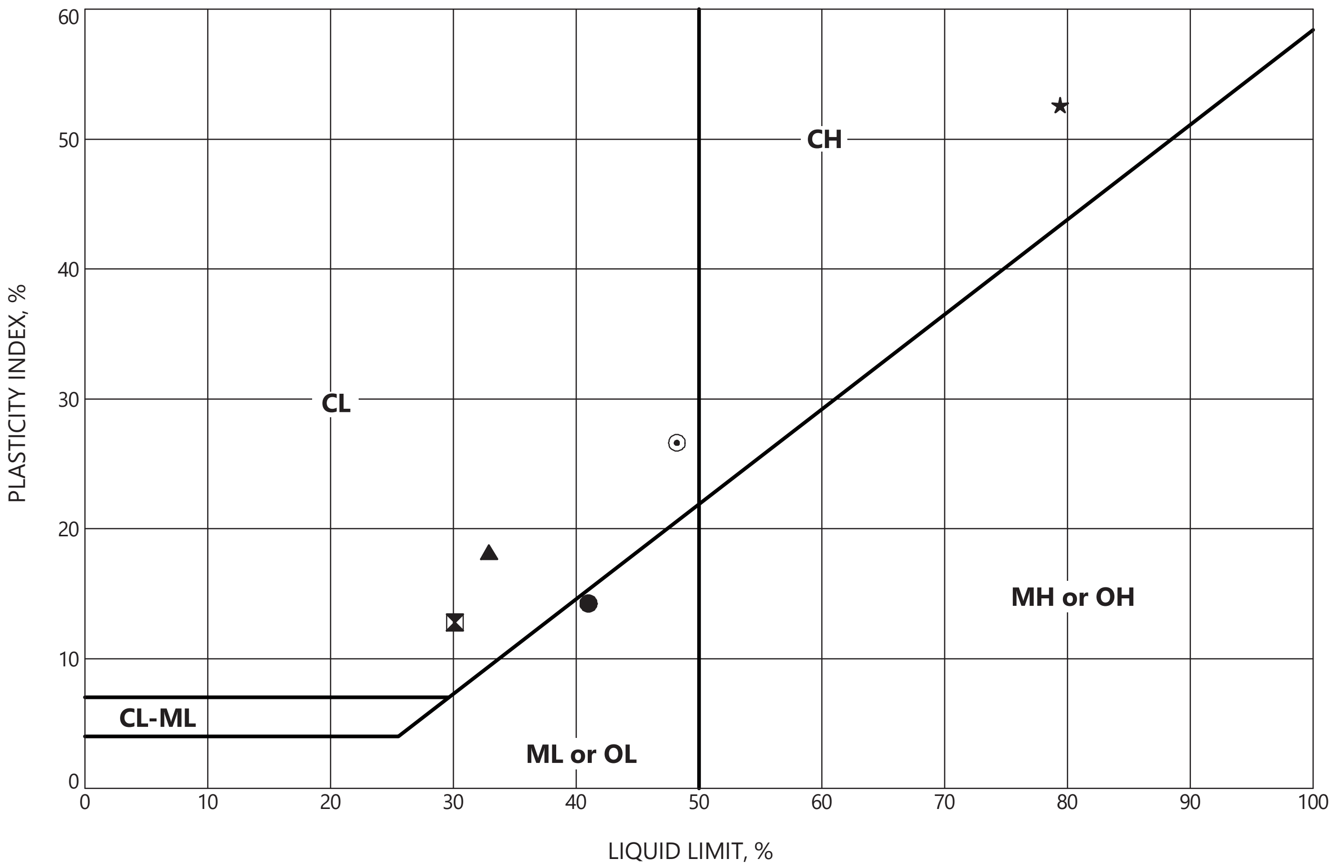
GRI

PLASTICITY CHART

NOV. 2024 | JOB NO. 6988-A | FIG. 9A

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
ML	INORGANIC CLAYEY SILTS TO VERY FINE SANDS OF SLIGHT PLASTICITY
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
MH	INORGANIC SILTS AND CLAYEY SILT
CH	INORGANIC CLAYS OF HIGH PLASTICITY



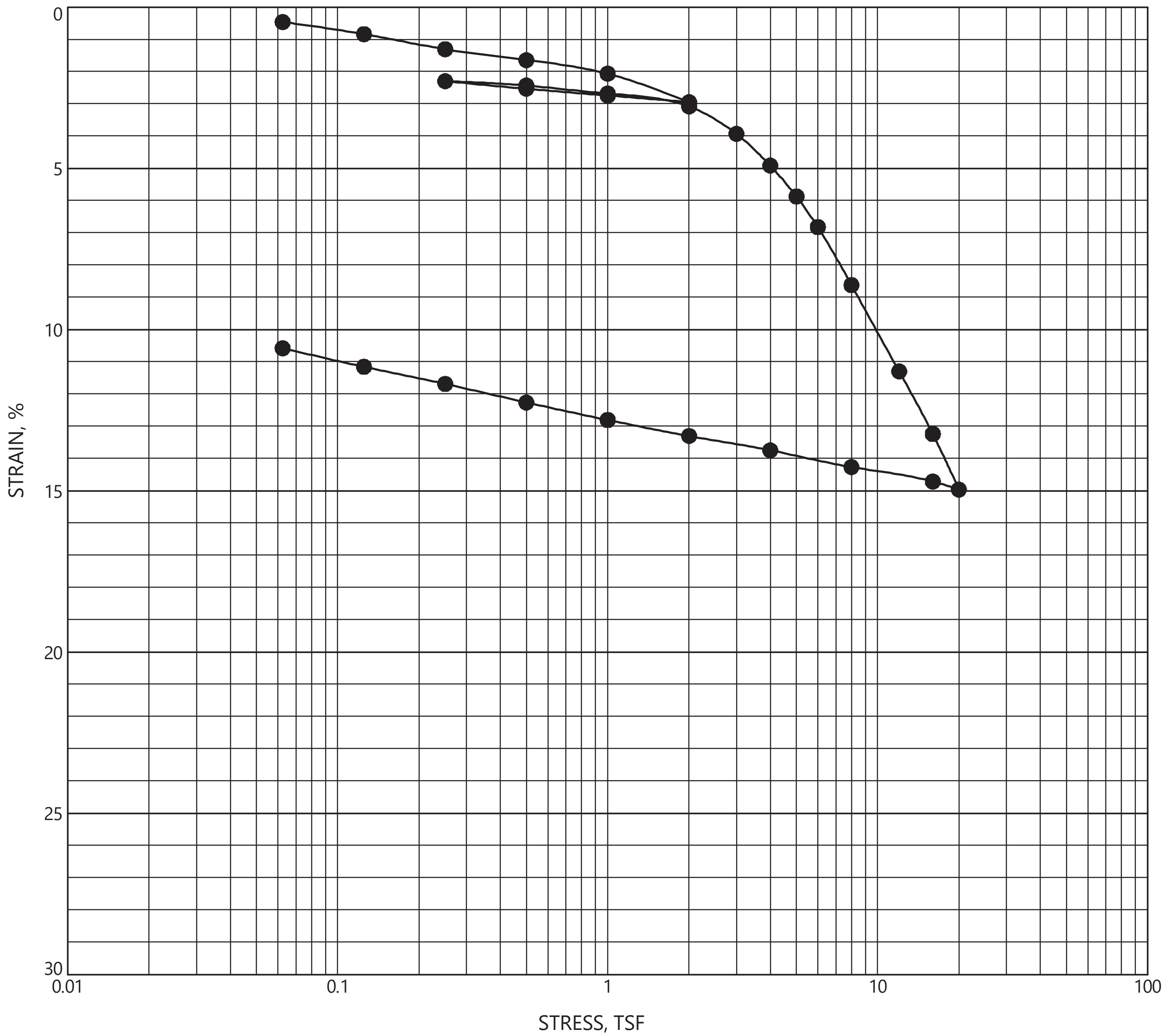
	Location	Sample	Depth, ft	Classification	LL	PL	PI	MC, %
●	B-2	S-8	20.0	SILT, trace sand; ML; brown; low plasticity; moist; fine to medium sand (Willamette Silt)	41	27	14	35
⊠	B-3	S-8	22.5	Clayey SILT to Silty CLAY, trace sand; ML/CL; gray to gray mottled yellow and red brown; low plasticity; moist; fine sand (Willamette Silt)	30	17	13	29
▲	B-3	S-9	24.5	Clayey SILT to Silty CLAY, trace sand; CL; gray to gray mottled yellow and red brown; medium plasticity; moist; fine sand (Willamette Silt)	33	15	18	26
★	B-3	S-12	35.0	CLAY, trace sand; CH; gray to gray mottled yellow and red brown; high plasticity; moist; fine sand (Hillsboro Formation)	79	27	52	34
⊙	B-3	S-15	45.0	Silty CLAY, some sand; CL; gray mottled red; medium plasticity; moist; fine sand (Hillsboro Formation)	48	22	26	35

GRI

PLASTICITY CHART

NOV. 2024 JOB NO. 6988-A FIG. 10A

ATTERBERG-PLASTICITY 5 PER PAGE GRI DATA TEMPLATE.GDT 11/5/24



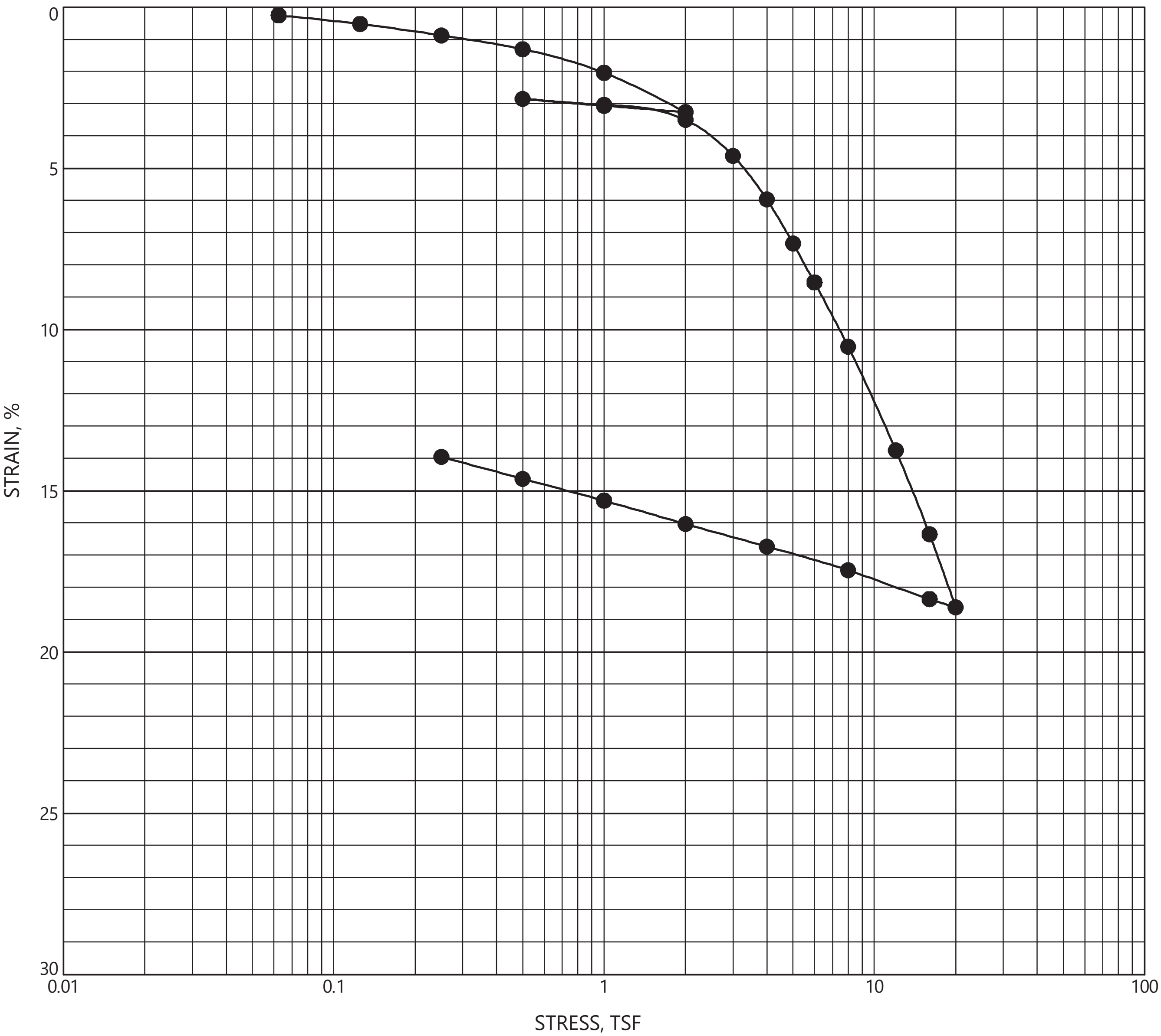
	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-1	S-3	8.0	SILT, trace sand; ML; brown; low plasticity; moist; fine sand (Willamette Silt)	89	29

CONSOL STRAIN GRI - 0 TO 30-1PER PAGE GRI DATA TEMPLATE.GDT 11/5/24

GRI

CONSOLIDATION TEST

NOV. 2024 JOB NO. 6988-A FIG. 11A



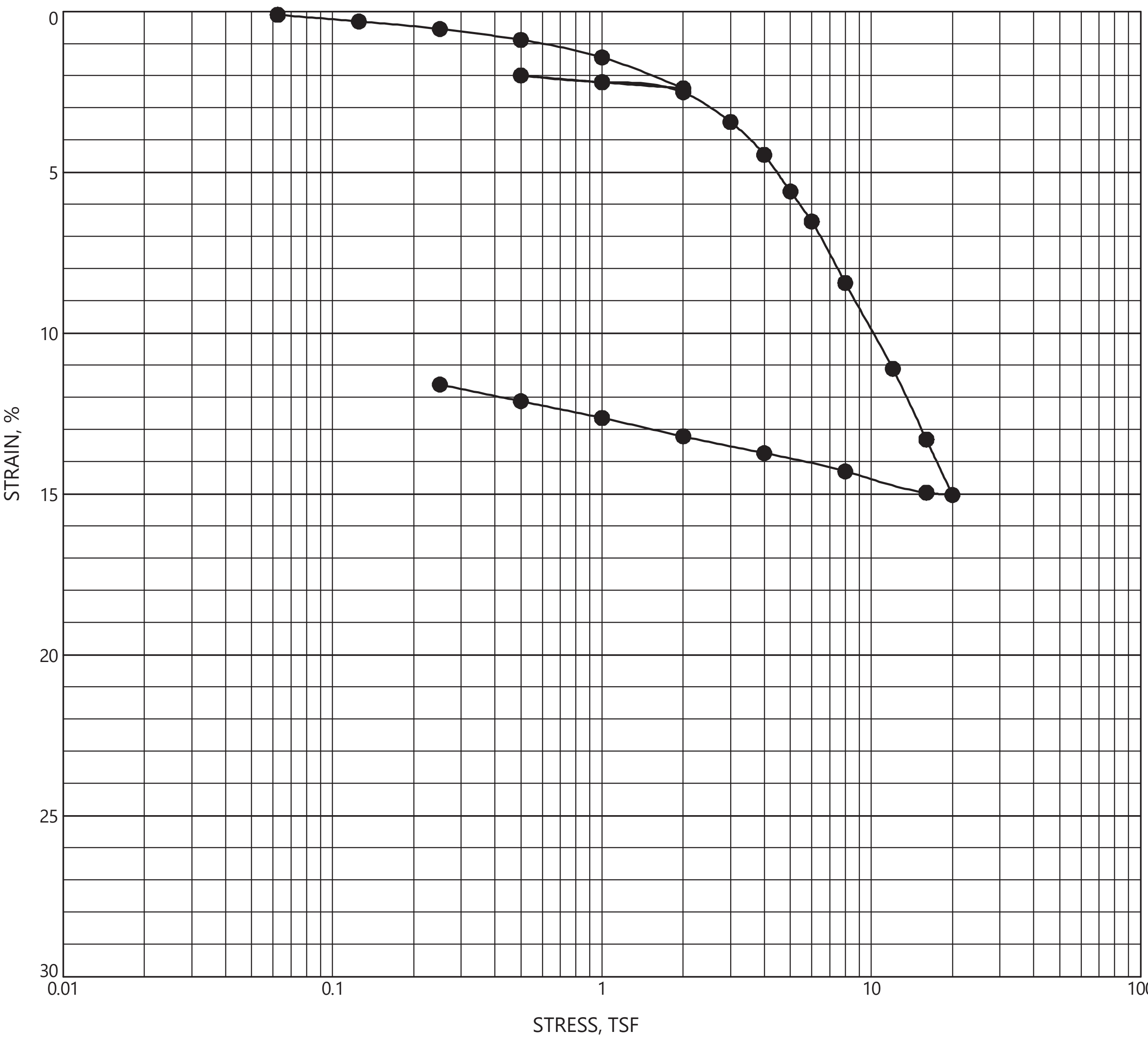
	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-1	S-7	21.2	Clayey SILT to Silty CLAY, trace sand; ML/CL; brown; medium plasticity; fine to medium sand (Willamette Silt)	84	39

GRI

CONSOLIDATION TEST

NOV. 2024 JOB NO. 6988-A FIG. 12A

CONSOL STRAIN GRI - 0 TO 30-1PER PAGE GRI DATA TEMPLATE.GDT 11/5/24



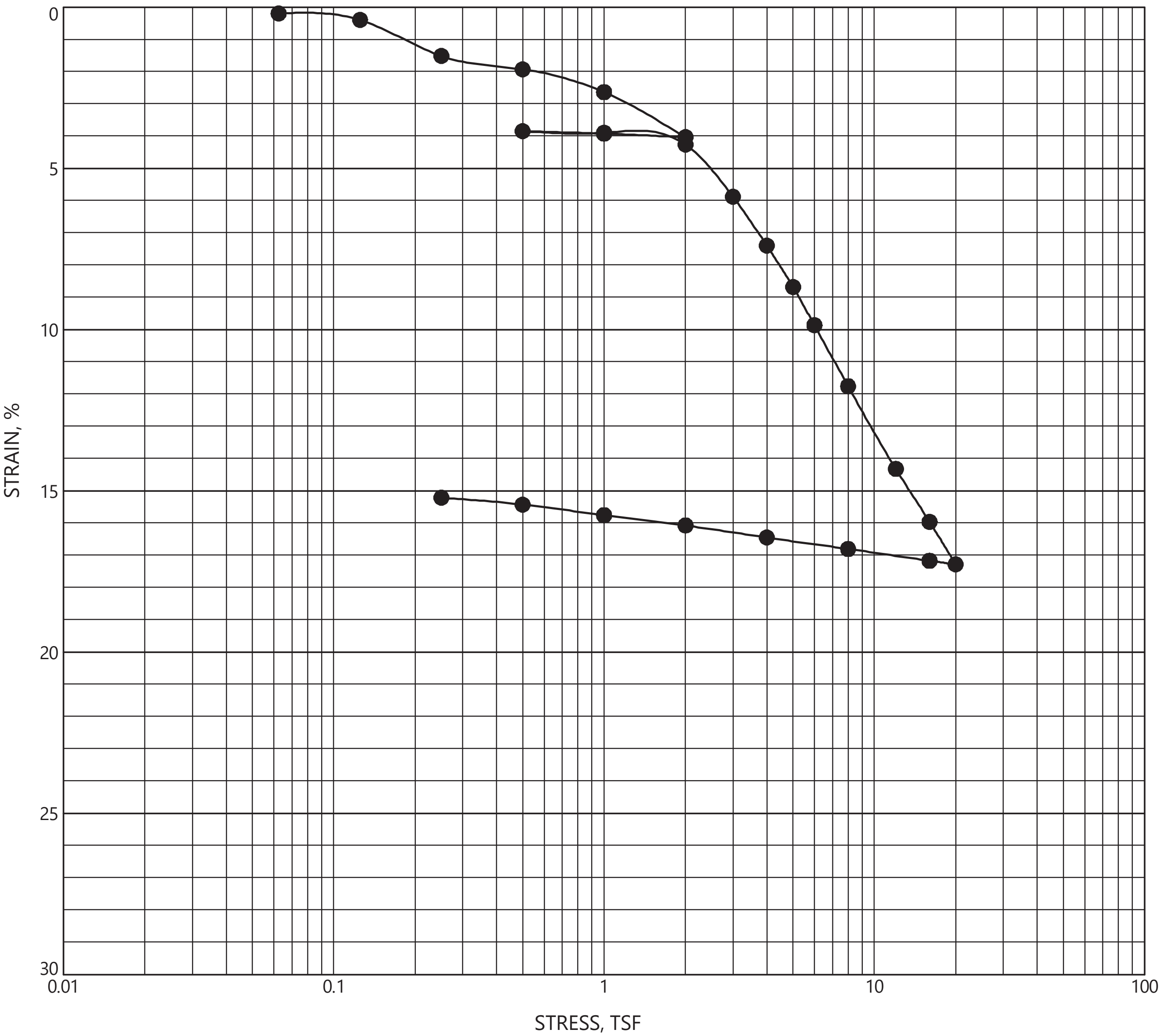
	Location	Sample	Depth, ft	Classification	Initial γd, pcf	Initial MC, %
●	B-2	S-6	15.8	SILT, trace sand; ML; brown; low plasticity; moist; fine to medium sand (Willamette Silt)	83	41

CONSOL STRAIN GRI - 0 TO 30-1PER PAGE GRI DATA TEMPLATE.GDT 11/5/24

GRI

CONSOLIDATION TEST

NOV. 2024 JOB NO. 6988-A FIG. 13A



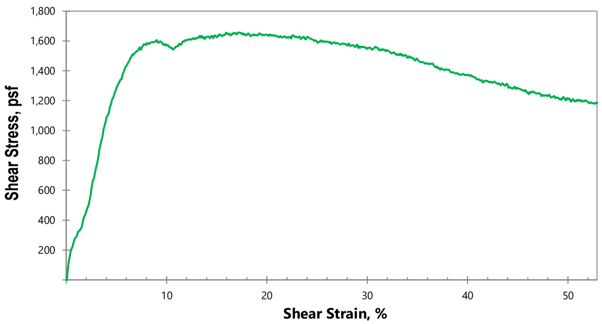
	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-3	S-8	24.0	Clayey SILT to Silty CLAY, trace sand; ML/CL; gray mottled yellow and red brown; medium plasticity; moist; fine sand (Willamette Silt)	92	29

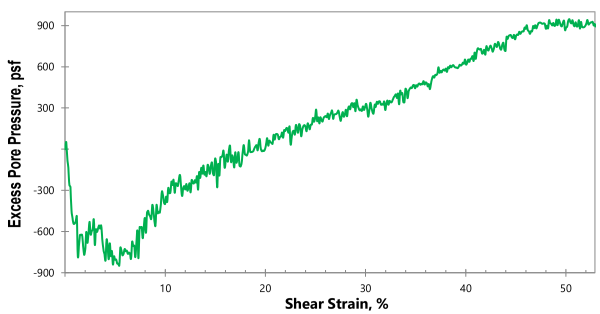
CONSOL STRAIN GRI - 0 TO 30-1PER PAGE GRI DATA TEMPLATE.GDT 11/5/24

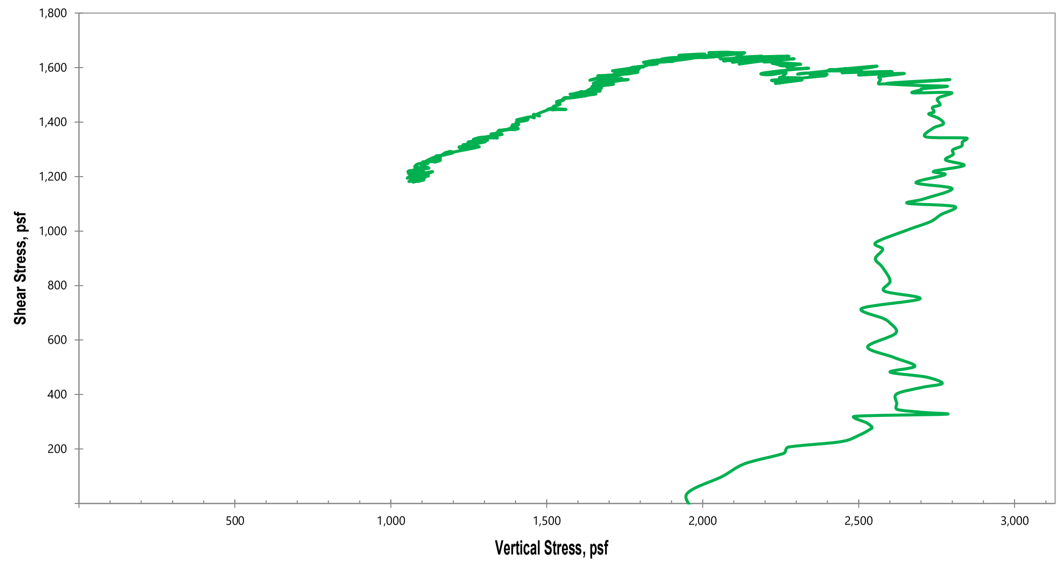
GRI

CONSOLIDATION TEST

NOV. 2024 JOB NO. 6988-A FIG. 14A







	TEST 1
TEST SYMBOL	■
BORING NO.	B-1
SAMPLE NO.	S-7
DEPTH (FT)	20.5 ft
VERTICAL EFFECTIVE CONSOLIDATION STRESS (PSF)	2,000
EST. OVERCONSOLIDATION RATIO	2.5
LIQUID LIMIT (%)	5
PLASTICITY INDEX (%)	24
FINES CONTENT (%)	100
DRY UNIT WEIGHT (PCF)	79
INITIAL WATER CONTENT (%)	42
FINAL WATER CONTENT (%)	41
STRAIN RATE (%/HR)	5

TYPE OF TEST: ■ CU ☐ CD

FAILURE CRITERIA: ■ MAX. SHEAR STRESS ☐ % SHEAR STRAIN

TYPE OF SAMPLE: ■ UNDISTURBED ☐ REMOLDED

SOIL CLASSIFICATION: Silty CLAY; CL; gray; medium plasticity; moist; medium stiff

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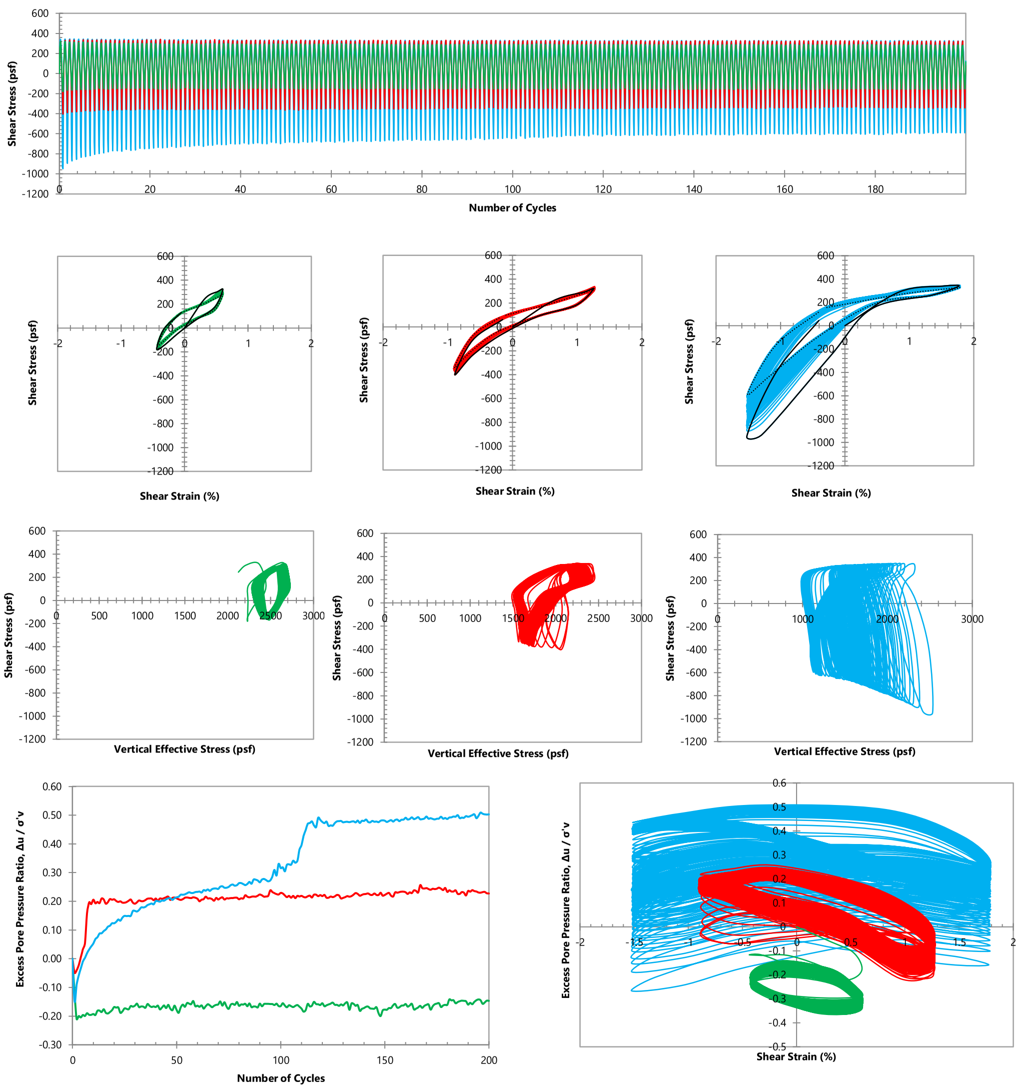
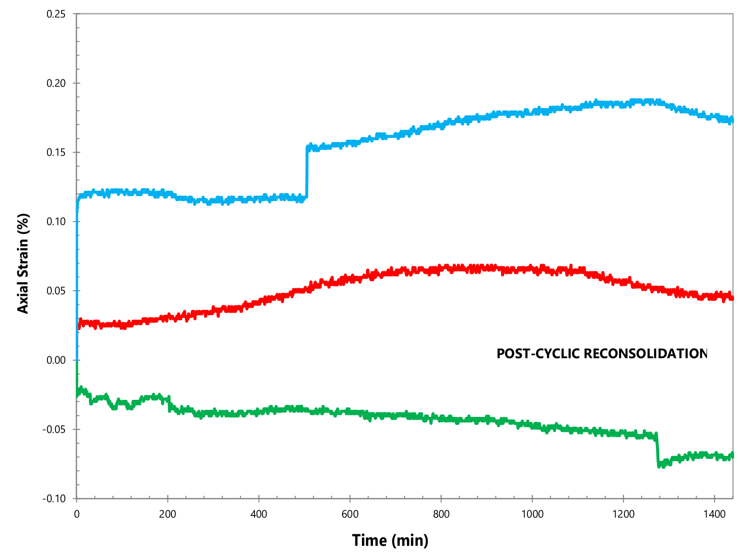
DIRECT SIMPLE SHEAR STRENGTH
(BORING B-1, S-7)

NOV. 2024 JOB NO. 6988-A FIG. 15A





TYPE OF TEST: ☐ STRESS-CONTROLLED ■ STRAIN-CONTROLLED

TYPE OF SAMPLE: ■ UNDISTURBED ☐ REMOLDED

SOIL CLASSIFICATION: Silty CLAY; CL; gray; medium plasticity; moist; medium stiff

										BEFORE			AFTER		
	BORING	DEPTH (FT)	Strain, %	FREQ. (Hz)	σ'_{vo} (psf)	PI	LL	Fines %	OCR	Ht. (in)	γ'_d	w%	Ht. (in)	γ'_d	w%
■ (green)	B-1	21.0	0.5	0.1	2,000	24	40	100	2.5	1.0	78	42	0.95	82	39
■ (red)	B-1	21.8	1.1	0.1	2,000	24	40	100	2.5	1.0	82	37	0.91	90	36
■ (blue)	B-1	21.4	1.7	0.1	2,000	24	40	97	2.5	1.0	88	33	0.91	96	31

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CYCLIC DIRECT SIMPLE SHEAR
(BORING B-1, S-7)

NOV. 2024 JOB NO. 6988-A FIG. 16A

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APPENDIX B

Site-Specific Seismic Hazard Study and Site Response Analysis

APPENDIX B

SITE-SPECIFIC SEISMIC HAZARD STUDY AND SITE RESPONSE ANALYSIS

B.1 GENERAL

GRI completed a site-specific seismic hazard study and site response analysis for the proposed Banks Middle School and High School improvements project in Banks, Oregon. The purpose of the site-specific seismic hazard evaluation was to evaluate the potential seismic hazards associated with regional and local seismicity, and the site response analysis was completed to evaluate the potential for seismic energy amplification at the site and determine site-specific acceleration response spectra for design of the proposed improvements. The site-specific seismic hazard evaluation is intended to fulfill the requirements of amended Section 1803 of the 2022 *Oregon Structural Specialty Code* (OSSC) for special-occupancy structures, which references the 2016 American Society of Civil Engineers (ASCE) 7-16 document, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7-16), for seismic design.

Our site-specific seismic hazard evaluation was based on the potential for regional and local seismic activity, as described in the existing scientific literature, and the subsurface conditions at the site, as disclosed by the geotechnical explorations completed for the project. Specifically, our work included the following tasks:

1. A review of available literature, including published papers, maps, open-file reports, seismic histories and catalogs, and other sources of information regarding the tectonic setting, regional and local geology, and historical seismic activity that might have a significant effect on the site.

2. Compilation and evaluation of subsurface data gathered at and in the vicinity of the site, including classification and laboratory analyses of soil samples. This information was used to prepare a generalized subsurface profile at the site.

3. Identification of potential seismic sources appropriate for the site and characterization of those sources in terms of magnitude, distance, and acceleration response spectra.

4. Office studies based on the generalized subsurface profile and controlling seismic sources resulting in conclusions and recommendations concerning:

    a. Specific seismic events and characteristic earthquakes that might have a significant effect on the project site.

    b. The potential for seismic energy amplification at the site.

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c. Site-specific acceleration response spectra for design of improvements at the site.

This appendix describes the work accomplished and summarizes our conclusions and recommendations.

B.2 GEOLOGIC SETTING

B.2.1 Regional Scale

On a regional scale, the site lies within the Willamette-Puget Sound lowland trough of the Cascadia convergent tectonic system (Blakely et al., 2000). The lowland area consists of a broad, north-south-trending basin in the underlying geologic structure between the Coast Range to the west and the Cascade Mountains to the east. The lowland trough is characterized by alluvial plains with areas of buttes and terraces. The site lies approximately 49 kilometers (km) inland from the down-dip edge of the seismogenic extent of the Cascadia Subduction Zone (CSZ), an active convergent-plate boundary along which remnants of the Farallon Plate (the Gorda, Juan de Fuca, and Explorer plates) are being subducted beneath the western edge of the North American continent. The subduction zone is a broad, eastward-dipping zone of contact between the upper portion of the subducting slabs and the overriding North American Plate, as shown on the Tectonic Setting Summary, Figure 1B.

B.2.2 Local Scale

On a local scale, the site is located within the Portland Basin, a large, well-defined, northwest-trending structural basin in the forearc of the CSZ. Based on numerous wells located within the region, the general consensus is that the Portland Basin is a right-lateral, pull-apart basin. However, recent surveys indicate the basin is actually a more-complex transpressional structure (Evarts et al., 2009). The Portland Basin is bounded by high-angle, northwest-trending, right-lateral strike-slip faults considered to be seismogenic. The geologic units in the area are shown on the Regional Geologic Map, Figure 2B. The distribution of nearby Quaternary faults is shown on the Local Fault Map, Figure 3B. Information regarding the continuity and potential activity of these faults is lacking due largely to the scale at which geologic mapping in the area has been conducted and the presence of thick, relatively young, basin-filling sediments that obscure underlying structural features. Other faults may be present within the basin but clear stratigraphic and/or geophysical evidence regarding their location and extent is not presently available.

B.2.3 Subsurface Conditions

Published geologic mapping indicates the site is mantled with Missoula flood deposits, locally referred to in the project area as the Willamette Silt Formation (Wells et al., 2020). In general, Willamette Silt is composed of beds and lenses of silt and sand. Stratification within this formation commonly consists of 4- to 6-inch-thick beds; although in some areas, the silt and sand are massive, and the bedding is indistinct or nonexistent. The Hillsboro Formation, which typically consists of stratified clay, silt, sand, and gravel, commonly underlies the Willamette Silt at depths between about 35 feet and 45 feet in this area.

B.3 SEISMICITY

B.3.1 General

The available information indicates that the potential seismic sources that may affect the site can be grouped into four independent categories:

1. Subduction zone events related to sudden slip between the upper surface of the Juan de Fuca plate and the lower surface of the North American plate

2. Subcrustal (intraslab) events related to deformation and volume changes within the deeper portion of the subducted Juan de Fuca plate

3. Local crustal events associated with movement on shallow local faults

4. Volcanic earthquakes associated with volcanic eruptions.

Each source is considered capable of producing damaging earthquakes in the Pacific Northwest. However, there are no historical records of significant subcrustal (i.e., moment magnitude [Mw] > 6.0), intraslab earthquakes or volcanic earthquakes in the region. The impact of Cascade Range volcano seismicity on the project site is considered to be low because the project site is located over 200 km from the nearest active volcano and there are no historical records of large earthquakes (i.e., Mw ≥ 6) associated with the volcanic activity. Based on review of historical records and evaluation of U.S. Geological Survey (USGS) National Seismic Hazard Maps (NSHMs), the two primary types of seismic sources at the site are the megathrust CSZ and local crustal faults.

B.3.2 Cascadia Subduction Zone

Written Japanese tsunami records suggest a great CSZ earthquake occurred in January 1700 (Atwater et al., 2015). Geological studies suggest great megathrust earthquakes have occurred repeatedly in the past 7,000 years (Atwater et al., 1995; Clague, 1997; Goldfinger et al., 2003; Kelsey et al., 2005), and geodetic studies (Hyndman and Wang, 1995; Savage et al., 2000) indicate a rate of strain accumulation consistent with the assumption that the CSZ is locked beneath offshore northern California, Oregon, Washington, and southern British Columbia (Fluck et al., 1997; Wang et al., 2001). Numerous geological and geophysical studies suggest the CSZ may be segmented (Hughes and Carr, 1980; Weaver and Michaelson, 1985; Guffanti and Weaver, 1988; Goldfinger, 1994; Kelsey and Bockheim, 1994; Mitchell et al., 1994; Personius, 1995; Nelson and Personius, 1996; Witter, 1999), but the most recent studies suggest that, for the last great earthquake in 1700, most of the subduction zone ruptured in a single M_W 9 earthquake (Satake et al., 1996; Atwater and Hemphill-Haley, 1997; Clague et al., 2000). Published estimates of the probable maximum size of subduction-zone events range from M_W 8.3 to >M_W 9. Numerous detailed studies of coastal subsidence, tsunamis, and turbidites yield a wide range of recurrence intervals, but the most complete records (>4,000 years) indicate intervals of about 350 years to 600 years between great earthquakes on the CSZ (Adams, 1990; Atwater and Hemphill-Haley, 1997; Witter, 1999; Clague et al., 2000; Kelsey et al., 2002; Kelsey et al., 2005; Witter et al., 2003). Tsunami inundation in buried marshes along the Washington and Oregon coasts and stratigraphic evidence from the Cascadia margin support these recurrence intervals (Kelsey et al., 2005; Goldfinger et al., 2003). Goldfinger et al. (2003, 2012, and 2017) evaluated turbidite evidence for 20 earthquakes that ruptured the entire CSZ over the past 10,000 years and about 20 M_W 8 earthquakes that only ruptured along the southern portion of the CSZ and developed a model for recurrence of CSZ M_W 8 to M_W 9 earthquakes.

The USGS probabilistic analysis assumes four potential locations (three alternative down-dip edge options and one up-dip edge option) for the eastern edge of the earthquake rupture zone for the CSZ, as shown on the Variation of Earthquake Rates Cascadia Subduction Zone, Figure 4B. As discussed in Petersen et al. (2014), the 2014 USGS mapping effort represents the 2014 CSZ source model with the full CSZ ruptures with moment magnitudes from M_W 8.6 to M_W 9.3 supplemented by partial ruptures with smaller magnitudes from M_W 8.0 to M_W 9.1. The partial ruptures were accounted for using a segmented model and an unsegmented model. The magnitude-frequency distribution showing the contributions to the earthquake rates from each model and how the rates vary along the fault are presented on the Location of Surface Traces for Up-Dip Edge & Three Down-Dip Edge Options Used in 2014 NSHMs, Figure 5B. In general, the earthquake rates along the CSZ are dominated by the full-characteristic CSZ ruptures, with one event in 526 years (M_W 8.6 to M_W 9.3 earthquakes likely occur more often than the smaller, segmented ruptures). Therefore, in our opinion, the CSZ event should be represented by an earthquake of M_W 9.0 at a focal depth of 30 km and rupture distance of about 49 km.

B.3.3 Local Crustal Event

The locations and general information regarding Quaternary faults (i.e., those that have experienced movement during the last 2.6 million years and are considered to be potentially active) are available through the USGS Earthquake Hazards Program. The precise relationship between specific earthquakes and individual faults is not well understood because few of the faults in the area are expressed at the ground surface, and the foci of the observed earthquakes have not been located with precision. The history of local seismic activity is commonly used as a basis for determining the size and frequency

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to be expected of local crustal events. Although the historical record of local earthquakes is relatively short (the earliest reported seismic event in the area occurred in 1920), it can serve as a guide for estimating the potential for seismic activity in the area.

The USGS Quaternary Fault and Fold Database of the United States indicates there are four faults within 25 km of the site: the Gales Creek fault zone at about 9.8 km, the Helvetia Fault at about 12 km, the Newberg Fault at about 14.8 km, and the Portland Hills Fault at about 18.6 km. However, USGS only considers the Gales Creek fault zone and Portland Hills Fault to be active, contributing sources in their Probabilistic Seismic Hazard Analysis (PSHA). The Gales Creek fault zone is considered to be a strike-slip fault with a total length of 27 km and a characteristic earthquake magnitude of M_W 6.8; the Portland Hills Fault is considered to be a reverse fault that dips to the southwest with a total fault length of approximately 50 km and a characteristic earthquake magnitude of M_W 7.0. Based on our review of the USGS Quaternary Fault and Fold Database of the United States, it is our opinion that a seismic event occurring on the Gales Creek fault zone should be represented by a source-to-site distance of approximately 9.8 km and a corresponding characteristic earthquake magnitude of M_W 6.8; the Portland Hills Fault should be represented by a source-to-site distance of approximately 18.6 km and a corresponding characteristic earthquake magnitude of M_W 7.1.

B.4 ONE-DIMENSIONAL SITE-RESPONSE ANALYSIS

B.4.1 General

A nonlinear, one-dimensional (1D) Total Stress Analysis (TSA) was completed to evaluate the site-specific influence of subsurface conditions on the resulting ground-surface response spectra. The nonlinear, dynamic site-response modeling generally consisted of determination of target response spectrum at the base of the soil column and applying one-dimensional numerical model to analyze the site-specific behavior of the soils using horizontal ground-motion acceleration time histories scaled to the approximate level of the target response spectrum over the periods of interest. The site-response analysis was completed in accordance with Chapter 21 of ASCE 7-16. The following sections discuss the steps in additional detail.

B.4.2 Target Spectra Development

The site-response analysis requires developing target spectra at the base of the soil column prior to selecting and scaling the input earthquake-acceleration time histories. The target spectra were developed based on the site-specific PSHA values in accordance with the requirements of ASCE 7-16. The site-specific PSHA values were derived based on the recently released NSHM Hazard Tool (https://earthquake.usgs.gov/nshmp/) at the base of the soil column. The base motions for site response are commonly developed for Site Class B/C boundary conditions, which correspond to a shear-wave velocity of 2,500 feet per second (ft/s). Table 1B summarizes the uniform hazard spectral (UHS) values at the project site for Site Class B/C boundary conditions. These UHS values correspond to 2,475-year (2% in 50 years) return period and represent the "geomean" spectral response accelerations.

Table 1B: SITE-SPECIFIC 2,475-YEAR UHS VALUES (B/C BOUNDARY CONDITION)

Spectral Acceleration, g	
Period, seconds	**2,475-year**
0.01	0.47
0.10	0.97
0.20	1.01
0.30	0.86
0.50	0.64
0.75	0.50
1.00	0.41
2.00	0.23
3.00	0.15
4.00	0.11
5.00	0.08

Abbreviation: UHS = uniform hazard spectral

The 2018 USGS deaggregation of probabilistic ground motions at the site indicates the CSZ interface and crustal sources are the primary contributors to the potential seismicity of the site. In general, the local crustal sources control the seismic hazard at shorter time-period ranges, while the CSZ interface sources control the hazard at longer periods. Therefore, individual target spectra were developed for both CSZ interface and local crustal sources to characterize the contribution of each primary source more appropriately. These individual target spectra were developed using the same ground-motion models and corresponding weights employed in developing the 2018 NSHMs. The Target Spectra for B/C Boundary Condition, Figure 6B, shows a comparison of the 2018 NSHMs PSHA values and the individual CSZ and crustal target bedrock spectra developed for this analysis.

B.4.3 Ground-Motion Development

For the site-response analysis, a suite of 10 recorded horizontal ground-motion acceleration time histories were selected from a dataset of subduction zone and crustal earthquakes having spectral shapes consistent with those that control the target spectra. The subduction-zone records were obtained from the Next Generation Attenuation Subduction (NGA-sub) project database, and the crustal ground motions were obtained from the Pacific Earthquake Engineering Research Center project database. A summary of the selected time histories for site response modeling is provided in Table 2B, below. Most of the subduction-zone time histories were selected from the two most recent large subduction-zone earthquakes (i.e., 2010 Maule and 2011 Tohoku) with magnitudes between M_W 8 to M_W 9, and additional time histories were selected from the 2015 Illapel and 2018 Hokkaido earthquakes. The crustal time histories were selected from the 1989 Loma Prieta, 1994 Northridge, 2003 San Simeon, and 2008 Iwate earthquakes with magnitudes ranging from M_W 6 to M_W 7.

Table 2B: SUMMARY OF GROUND-MOTION RECORDS SELECTED FOR SITE-RESPONSE MODELING

No.	Earthquake/Year	Magnitude, M_W	Station Name	Record ID	Unscaled PGA, g	Sampling Frequency, Hz	Record Length, seconds
1	Tohoku/2011(a)	9.1	GN4	GN4-EW	0.22	100	300
2	Tohoku/2011(a)	9.1	CHB004	CHB004-NS	0.29	100	300
3	Maule/2010(a)	8.8	SantiagoLa Florida	SLaFlorida-NS	0.19	200	208
4	Maule/2010(a)	8.8	SLUC	SLUC090	0.34	100	160
5	Illapel/2015(a)	8.3	BO01H	BO01HNE	0.09	100	400
6	Hokkaido/2018(a)	6.6	HKD094	HKD094EW	0.14	100	188
7	San Simeon/2003(b)	6.5	Templeton	SANSIMEO-090	0.44	200	101
8	Iwate/2008(b)	6.9	AKT023	IWATE-AKT023EW	0.37	100	300
9	Loma Prieta/1989(b)	6.9	Gilroy Array #6	LOMAP_G06 000	0.13	200	40
10	Northridge/1994(b)	6.7	Vasquez Rocks	NORTHR_VA S090	0.15	50	40

Abbreviations: ID = identification; PGA = peak ground acceleration; Hz = Hertz

Following the selection of the time histories, the input bedrock motions were linearly modified using amplitude scaling, so the mean response spectra of the recordings reasonably matched the crustal and CSZ target spectra. The amplitude-scaling process involves selecting a single scaling factor for each time history and multiplying the entire acceleration time history by this factor, so its response spectrum approximates the input target spectra. Time histories were scaled to reasonably approximate the target spectra at the fundamental period of the site. Figures 7B and 8B show the comparison of the amplitude-scaled motions and the target spectra for CSZ interface and crustal motions, respectively, for the 2,475-year hazard level.

B.4.4 Modeling Approach

The non-linear site-response modeling was performed using a 1D, non-linear, effective-stress site-response analysis in the DEEPSOIL program (Hashash et al., 2016) developed by the University of Illinois. The program employs a time-domain site-response analysis capable of incorporating the non-linear hysteretic soil behavior that is observed during cyclic loading and unloading. The program computes the dynamic response of a layered soil profile to vertically propagating shear waves using a built-in total stress or effective stress analysis option. The program uses the pressure-dependent modified hyperbolic constitutive model initially developed by Kondner and Zelasko (1963; Modified Kondner and Zelasko model) and the General Quadratic/Hyperbolic (GQ/H) strength-controlled constitutive model recently introduced by Groholski et al. (2015). The GQ/H model allows the shear strength at failure to be defined while still providing the flexibility to represent small-strain soil behavior. Therefore, the GQ/H material model was utilized since it provides a better approximation of modulus reduction and damping and higher levels of shear strain approaching the ultimate shear strength while still maintaining small-strain nonlinearity.

The GQ/H parameters are generally obtained by fitting the hyperbolic model to published empirical modulus reduction and damping curves such as (Vucetic and Dobry, 1991; EPRI, 1993; and Darendeli, 2001). The conventional approach for defining unloading-reloading criteria and behavior under general cyclic-loading conditions (hysteretic damping) is based on the Masing criteria (Masing, 1926) and extended Masing criteria (Pyke, 1979; and Vucetic, 1990). An exact match of the target modulus reduction and damping curves is not concurrently possible using the Masing or extended Masing rules (i.e., one must match the target modulus reduction curve as accurately as possible and accept the misfit of damping or optimize the fit of both simultaneously). Phillips and Hashash (2009) developed an alternative non-Masing model by introducing a reduction factor that effectively alters the Masing rules and allows for both modulus reduction and damping curves to be fitted simultaneously. This model is implemented in DEEPSOIL as the Modulus Reduction and Damping Factor Pressure-Dependent Hyperbolic model (Phillips and Hashash, 2009).

In general, DEEPSOIL allows the user to create a discretized soil profile and input a variety of soil-modeling parameters derived from field and laboratory testing and established correlations in the geotechnical literature. A suite of scaled earthquake records is input to the program and propagated up through the soil column to the ground surface. From the modeled ground-surface response for a particular soil profile, a spectral amplification ratio (SAR) can be determined for each earthquake record as the ratio of ground surface to input target or bedrock spectral acceleration at selected periods.

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B.4.5 Input Simplified Soil Profile and Dynamic Properties

A generalized (simplified) subsurface profile was developed at the project site based on the existing subsurface information and recently completed explorations. The thickness and material properties of the site's soils were characterized based on the borings and cone penetration test (CPT) probes. The V_S profile has a large influence on the amplitude and frequency content of predicted ground-surface motions derived from site-response simulations. Therefore, it is crucial to develop appropriate V_S profiles for use in site-specific site-response analyses. The shear-wave velocity profile was developed based on the CPT probes (CPT-1 and CPT-2) that was completed in the project area. Figure 9B presents the results of the shear-wave velocity survey measurements and also shows the profile used for our site-response analysis.

The CPT measurements extend to a depth of up to about 87.6 feet below the existing ground surface. In general, the shear wave measurements show an increasing trend of shear wave velocity with depth. The elastic half-space condition at the base of the model was assumed to be at a depth of about 120 feet. The half-space boundary condition at the base of the model was represented by a visco-elastic boundary with a unit weight of 120 pounds per cubic foot (pcf) and shear-wave velocity of 2,500 ft/s. The generalized input parameter is summarized in Table 3B.

Table 3B: DEEPSOIL INPUT PROFILE (GENERALIZED SOIL PROFILE)

Layer	Soil Type	Depth, feet	Thickness, feet	Unit weight, pcf	Vs, ft/s	Modulus Reduction and Damping Curves[a]
1	Silt	4	4	115	600	Darendeli 2001 (PI = 15, OCR=5)
2	Silt	8	4	115	460	Darendeli 2001 (PI = 15, OCR=5)
3	Silt	12	4	115	500	Darendeli 2001 (PI = 15, OCR=4.5)
4	Silt	16	4	115	550	Darendeli 2001 (PI = 15, OCR=3.5)
5	Silt	20	4	115	646	Darendeli 2001 (PI = 15, OCR=2.5)
6	Silt Clay	25	5	115	672	Darendeli 2001 (PI = 15, OCR=2)
7	Silt Clay	30	5	115	646	Darendeli 2001 (PI =15, OCR=2)
8	Clay	35	5	115	670	Darendeli 2001 (PI = 25, OCR=2)

Layer	Soil Type	Depth, feet	Thickness, feet	Unit weight, pcf	Vs, ft/s	Modulus Reduction and Damping Curves[a]
9	Clay	40	5	115	684	Darendeli 2001 (PI = 25, OCR=1.5)
10	Clay	45	5	115	800	Darendeli 2001 (PI = 25, OCR=1.5)
11	Clay	50	5	115	810	Darendeli 2001 (PI = 25, OCR=1.3)
12	Clay	55	5	115	826	Darendeli 2001 (PI = 25, OCR=1)
13	Silty Sand	60	5	120	1169	Darendeli 2001
14	Silty Sand	65	5	120	1169	Darendeli 2001
15	Silty Sand	70	5	120	1399	Darendeli 2001
16	Silty Sand	75	5	120	980	Darendeli 2001
17	Silty Sand	80	5	120	998	Darendeli 2001
18	Silty Sand	90	10	120	1299	Darendeli 2001
19	Silty Sand	100	10	120	1713	Darendeli 2001
20	Clay	110	10	120	2142	Darendeli 2001 (PI = 25, OCR=1)
21	Clay	120	10	120	2500	Darendeli 2001 (PI = 25, OCR=1)

Abbreviations: pcf = pounds per cubic foot; ft/s = feet per second; PI = Plasticity Index; OCR = overconsolidation ratio

Note:

[a]Darendeli (2001) PI, OCR, and σ'm-dependent soil modulus reduction and material damping curves.

The dynamic properties of each soil layer were estimated using published relationships on similar materials and local experience. The empirical soil modulus and damping curves developed by Darendeli (2001) were assigned to both cohesive and cohesionless materials encountered at the project site. These curves depend on the plasticity index (PI), mean effective confining pressure (σ'_m), and overconsolidation ratio (OCR).

B.4.6 Site-Response Results

B.4.6.1 Ground-Surface Acceleration Response Spectra

Using the scaled ground-motion records listed in the preceding tables and the generalized soil profile, pseudo-acceleration response spectra were developed using total stress analyses and site-response analysis. The ground-surface response spectra for individual earthquake motions were developed at 5% of critical damping. The resulting response spectra were compared with the input target spectrum at the base of the soil column to quantify amplification and/or attenuation through the soil column at the site. In general, the ground-surface response spectra are defined as the base-target response spectrum multiplied by the SAR estimated from the site-response modeling. These response spectra represent the mean ground-surface response spectra at 5% damping using the suite of spectrum-compatible time histories previously discussed. Then, the site-specific spectra were developed for the site to represent the ground-surface MCE_R response spectra developed in accordance with ASCE 7-16, as discussed below. Figure 10B presents the mean SARs obtained from the TSA analyses, and the resulting mean ground-surface MCE_R spectra are summarized on Figure 11B.

ASCE 7-16 defines ground motions as the spectral response acceleration in the maximum direction of ground motions represented by a 5%-damped acceleration response spectrum expected to achieve a 1% probability of collapse within a 50-year period (i.e., MCE_R). Therefore, the ground-surface MCE_R spectra were obtained by applying directivity factors and risk coefficients to the ground-surface response acceleration values. The directivity factors adjust the spectral values from geometric mean to direction of maximum horizontal response and the risk coefficients incorporate the uniform collapse risk objective of 1% in 50 years. The geometric-mean ground motions are converted to the corresponding direction of maximum horizontal response values by factoring average response by period-dependent factors of 1.2 at periods less than or equal to 0.2 seconds, by 1.25 for a period of 1.0 seconds, and by 1.3 for periods greater than or equal to 10 seconds. The risk coefficients obtained from USGS maps indicate the short- and long-period risk coefficients (C_{RS}) and (C_{R1}) at the site are approximately 0.88 and 0.87, respectively. For spectral periods between 0.2 seconds and 1.0 seconds, the risk coefficients were estimated using linear interpolation.

B.4.6.2 Code-Based Spectra Comparisons and Recommended Design Spectra

Typically, the recommended response spectra for structural design can be developed by comparing the site-specific spectra based on site-response modeling with the code-based spectra based on site class and generic site-amplification factors. ASCE 7-16 requires the site-specific spectral accelerations at the ground surface not be taken as less than 80% of the spectral values (i.e., code-minimum) determined using site-specific classification procedures outlined in ASCE 7-16, Chapter 20. At the project site, the site is designated Site Class D based on the V_S profile for the upper 100 feet (i.e., V_{S30}=830 ft/s) obtained from the CPT probes.

Comparisons of the site-specific ground-surface spectra and the code-based ground-surface spectra are shown on Figure 12B. The code-based Site Class D spectrum was derived based on the 0.2- and 1.0-second spectral-acceleration values (S_S and S_1) at the bedrock and corresponding site coefficients, F_a and F_v, in accordance with Chapter 21 of ASCE 7-16, with amendment in subsection 1613.4.13 of 2022 OSSC. The modification typically applies to the value of F_v, suggested to be determined using straight-line interpolation between the value determined from ASCE 7-16 Section 21.3 (i.e., associated with 0% CSZ interface contribution) and the value from the 2022 OSSC Table 1613.2.3(2) (i.e., associated with 100% CSZ interface contribution) based on the relative hazard contribution from the CSZ interface sources at a period of 1.0 seconds. The USGS hazard tool shows about 95% contribution from the CSZ interface source at the site. The 0.2- and 1.0-second spectral (S_S and S_1) values for the site at bedrock are 0.92 g and 0.46 g, respectively. The short-period site coefficient, F_a, which equals 1.13, was determined using Table 11.4-1 of ASCE 7-16. The long-period site coefficient, F_v, which equals 1.88, was determined using straight-line interpolation between the ASCE 7-16-recommended value of 2.5 and the 2022 OSSC value of 1.85 based on the relative CSZ interface hazard contribution. These site coefficients were applied in developing the Site Class D spectrum. ASCE 7-16 requires the site-specific spectral accelerations at the ground surface not to be less than 80% of the spectral values determined for Site Class D.

Figure 12B shows the site-specific response spectrum (i.e., weighted average of mean crustal and CSZ) obtained from site-response modeling was generally higher than the code-based Site Class D spectral values at periods between about 0.15 seconds and 0.85 seconds. If a site-response analysis is completed, ASCE 7-16 allows the maximum spectral acceleration to be limited to 90% of the peak acceleration of the site-specific spectrum at any period within the range from 0.2 seconds to 5 seconds, or 80% of the spectral values determined for Site Class D, whichever is greater. Therefore, the recommended MCE_R spectral response parameters are based on 90% of the peak site-specific spectra for the short periods (S_{MS}), and the site-specific spectra and 80% of Site Class D spectral values for the 1-second period (S_{M1}).

The recommended MCE_R- and design-level spectral response parameters based on site response modeling for Site Class D conditions are provided below, in Table 4B. These values assume dynamic seismic design of the structure will be completed using the Equivalent Lateral Force (ELF) design procedure that meets the intent of ASCE 7-16 with the proposed 2022 OSSC modifications. We should be contacted if seismic design will be completed using modal response-spectrum analysis or nonlinear response history analysis methodologies.

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Table 4B: RECOMMENDED SEISMIC DESIGN PARAMETERS FOR ELF, 5% DAMPING

Seismic Parameter	Recommended Value
Site Class	D
MCE_R Spectral Response Acceleration Parameter at Short Periods, S_{MS}	1.10 g
MCE_R Spectral Response Acceleration Parameter at 1.0-Second Period, S_{M1}	0.79 g
Design Spectral Response Acceleration Parameter at Short Periods, S_{DS}	0.74 g
Design Spectral Response Acceleration Parameter at 1.0-Second Period, S_{D1}	0.53 g

Abbreviations: ELF = Equivalent Lateral Force; MCE_R = Risk-Targeted Maximum Considered Earthquake

B.5 CONCLUSIONS

The site-specific seismic hazard evaluation was completed to fulfill the requirements of amended Section 1803 of the 2022 OSSC for special-occupancy structures. As part of our evaluation, site-specific site response modeling was completed to evaluate the site-specific influence of subsurface conditions on the resulting ground-surface response spectra. The site response modeling was based on the generalized subsurface profiles developed for the site from the results of field explorations and laboratory testing. Based on the result of our site-specific site response analysis, we recommend using the spectral response parameters provided in Table 4B assuming dynamic seismic design of the structure will be completed using the ELF design procedure.

B.6 REFERENCES

Adams, J., 1990, Paleoseismicity of the Cascadia Subduction Zone: Evidence from turbidites off the Oregon-Washington margin, *Tectonics*, vol. 9, no. 4, pp. 569–583.

American Society of Civil Engineers (ASCE), 2016b, ASCE 7-16, Minimum design loads for buildings and other structures, draft copy: ASCE, Reston, Virginia.

Atwater, B. F., and Hemphill-Haley, E., 1997, Recurrence intervals for great earthquakes of the past 3,500 years at northeastern Willapa Bay, Washington: U.S. Geological Survey, Professional Paper 1576, 108 pp.

Atwater, B. F., Musumi-Rokkaku, S., Satake, K., Tsuji, Y., Ueda, K., and Yamaguchi, D. K., 2015, The orphan tsunami of 1700—Japanese clues to a parent earthquake in North America, 2nd ed.: U.S. Geological Survey, Professional Paper 1707, Seattle, University of Washington Press, 135 pp.

Atwater, B. F., Nelson, A. R., Clague, J. J., Carver, G. A., Yamaguchi, D. K., Bobrowsky, P. T., Bourgeois, J., Darienzo, M. E., Grant, W. C., Hemphill-Haley, E., et al., 1995, Summary of coastal geologic evidence for past great earthquakes at the Cascadia Subduction Zone, *Earthquake Spectra*, vol. 11, no. 1, pp. 1–18.

Blakely, R. J., Wells, R. E., Tolan, T. L., Beeson, M. H., Trehu, A. M., and Liberty, L. M., 2000, New aeromagnetic data reveal large strike-slip (?) faults in the northern Willamette Valley, Oregon, *GSA Bulletin*, vol. 112, no. 7, pp. 1,225–1,233.

Clague, J. J., 1997, Evidence for large earthquakes at the Cascadia Subduction Zone, *Reviews of Geophysics*, vol. 35, no. 4, pp. 439–460.

Clague, J. J., Atwater, B. F., Wang, K., Wang, Y., and Wong, I., 2000, Penrose conference report—Great Cascadia earthquake tricentennial, *GSA Today*, vol. 10, no. 11, pp. 14–15.

Darendeli M., 2001, Development of a new family of normalized modulus reduction and material damping curves: University of Texas at Austin, PhD dissertation.

Electric Power Research Institute (EPRI), 1993, Guidelines for site-specific ground motions, Palo Alto, California, TR-102293.

Evarts, R. C., O'Connor, J. E., Wells, Ray E., and Madin, I. P., 2009, The Portland Basin: A (big) river runs through it, *GSA Today*, vol. 19, no. 9, doi:10.1130.

Fluck, P., Hyndman, R. D., and Wang, K., 1997, Three-dimensional dislocation model for great earthquakes of the Cascadia Subduction Zone, *Journal of Geophysical Research*, vol. 102, no. B9, pp. 20,539–20,550.

Goldfinger, C., 1994, Active deformation of the Cascadia Forearc—Implications for great earthquake potential in Oregon and Washington: Oregon State University, unpublished dissertation.

Goldfinger, C., Galer, S., Beeson, J., Hamilton, T., Black, B., Romsos, C., Patton, J., Nelson, C. H., Hausmann, R., and Morey, A., 2017, The importance of site selection, sediment supply, and hydrodynamics: A case study of submarine paleoseismology on the northern Cascadia margin, Washington USA, *Marine Geology In Press*, DOI: 10.1016/j.margeo.2016.06.008.

Goldfinger, C., Nelson, C. H., and Johnson, J. E., 2003, Holocene earthquake records from the Cascadia Subduction Zone and northern San Andreas fault based on precise dating of offshore turbidites, *Annual Review of Earth and Planetary Sciences*, vol. 31, pp. 555–577.

Goldfinger, C., Nelson, C. H., Morey, A., Johnson, J. E., Gutierrez-Pastor, J., Eriksson, A. T., Karabanov, E., Patton, J., Gracia, E., Enkin, R., Dallimore, A., Dunhill, G., and Vallier, T., 2012, Turbidite event history: Methods and implications for Holocene Paleoseismicity of the Cascadia Subduction Zone: U.S. Geological Survey, Professional Paper 1661.

Groholski, D. R., Hashash, Y. M. A., Musgrove, M., Harmon, J., and Kim, B., 2015, Evaluation of 1-D non-linear site response analysis using a general quadratic/hyperbolic strength-controlled constitutive model, 6ICEGE: 6th International Conference on Earthquake Geotechnical Engineering, Christchurch, New Zealand.

Guffanti, M., and Weaver, C. S., 1988, Distribution of Late Cenozoic volcanic vents in the Cascade Range—Volcanic arc segmentation and regional tectonic considerations, *Journal of Geophysical Research*, vol. 93, no. B6, pp. 6,513–6,529.

Hashash, Y. M. A., Musgrove, M. I., Harmon, J. A., Groholski, D. R., Phillips, C. A., and Park, D., 2016, DEEPSOIL 6.1, User manual, 125 pp.

Hughes, J. M., and Carr, M. J., 1980, Segmentation of the Cascade volcanic chain, *Geology*, vol. 8, pp. 15–17.

Hyndman, R. D., and Wang, K., 1995, The rupture zone of Cascadia great earthquakes from current deformation and the thermal regime, *Journal of Geophysical Research*, vol. 100, no. B11, pp. 22,133–22,154.

Kelsey, H. M., and Bockheim, J. G., 1994, Coastal landscape evolution as a function of eustasy and surface uplift rate, Cascadia margin, Southern Oregon, *Geological Society of America Bulletin*, vol. 106, pp. 840–854.

Kelsey, H. M., Nelson, A. R., Hemphill-Haley, E., and Witter, R. C., 2005, Tsunami history of an Oregon coastal lake reveals a 4600 yr record of great earthquakes on the Cascadia Subduction Zone, *Geological Society of America Bulletin*, vol. 117, no. 7-8, pp. 1,009–1,032.

Kelsey, H. M., Witter, R. C., and Hemphill-Haley, E., 2002, Pl.-boundary earthquakes and tsunamis of the past 5500 yr, Sixes River estuary, Southern Oregon, *Geological Society of America Bulletin*, vol. 114, no. 3, pp. 298–314.

Kondner, R. L., and Zelasko, J. S., 1963, Hyperbolic stress-strain formulation of sands, Second pan American Conference on Soil Mechanics and Foundation Engineering, Sao Paulo, Brazil, pp. 289-324.

Masing, G., 1926, Eigenspannungen and verfertigung beim messing: Procedures of the 2nd International Congress on Applied Mechanics, Zurich, Switzerland.

Mitchell, C. E., Vincent, P., Weldon, R. J., III, and Richards, M. A., 1994, Present-day vertical deformation of the Cascadia margin, Pacific Northwest, United States, *Journal of Geophysical Research*, vol. 99, no. B6, pp. 12,257–12,277.

Nelson, A. R., and Personius, S. F., 1996, Great-earthquake potential in Oregon and Washington—An overview of recent coastal geologic studies and their bearing on segmentation of Holocene ruptures, central Cascadia Subduction Zone, in Rogers, A.M., Walsh, T.J., Kockelman, W.J., and Priest, G.R., eds., Assessing earthquake hazards and reducing risk in the Pacific Northwest: U.S. Geological Survey, Professional Paper 1560, vol. 1, pp. 91–114.

Personius, S. F., 1995, Late Quaternary stream incision and uplift in the forearc of the Cascadia Subduction Zone, Western Oregon, *Journal of Geophysical Research*, vol. 100, no. B10, pp. 20,193–20,210.

Petersen, M. D., Moschetti, M. P., Powers, P. M., Mueller, C. S., Haller, K. M., Frankel, A. D., Zeng, Y., Rezaeian, S., Harmsen, S. C., Boyd, O. S., et al., 2014, Documentation for the 2014 update of the United States National Seismic Hazard Maps: U.S. Geological Survey, Open-File Report 2014–1091, 243 pp., obtained at: http://dx.doi.org/10.3133/ofr20141091.

Phillips, C., and Hashash, Y. M. A., 2009, Damping formulation for non-linear 1D site response analyses, *Soil Dynamics and Earthquake Engineering*, vol. 29, no. 7, pp. 1,143–1,158.

Pyke, R., 1979, Nonlinear soil model for irregular cyclic loadings, *Journal of Geotechnical Engineering Division*, vol. 105, pp. 715–726.

Satake, K., Shimazaki, K., Tsuji, Y., and Ueda, K., 1996, Time and size of a giant earthquake in Cascadia inferred from Japanese tsunami records of January 1700, *Nature*, vol. 379, pp. 246–249.

Savage, J. C., Svarc, J. L., Prescott, W. H., and Murray, M. H., 2000, Deformation across the forearc of the Cascadia Subduction Zone at Cape Blanco, Oregon, *Journal of Geophysical Research*, vol. 105, no. B2, pp. 3,095–3,102.

U.S. Geological Survey (USGS), 2014, Unified hazard tool lookup by latitude, longitude, accessed October 1, 2021, from USGS website: https://earthquake.usgs.gov/hazards/interactive/.

Vucetic, M., 1990, Normalized behavior of clay under irregular cyclic loading, *Canadian Geotechnical Journal*, vol. 27, pp. 29–46.

Vucetic, M., and Dobry, R., 1991, Effect of soil plasticity on cyclic response, *Journal of Geotechnical Engineering*, vol. 117, no. 1, pp. 89–107.

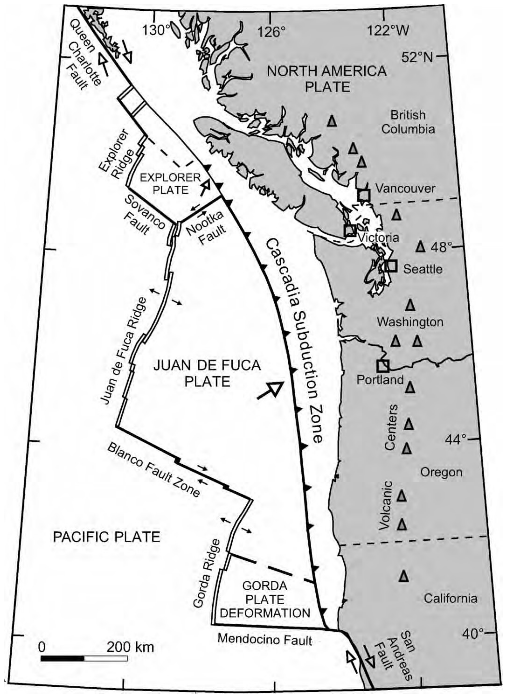
Wang, Y., He, J., Dragert, H., and James, T. S., 2001, Three-dimensional viscoelastic interseismic deformation model for the Cascadia Subduction Zone, *Earth, Planets, and Space*, vol. 53, pp. 295–306.

Weaver, C. S., and Michaelson, C. A., 1985, Seismicity and volcanism in the Pacific Northwest—Evidence for the segmentation of the Juan de Fuca Pl., *Geophysical Research Letters*, vol. 12, no. 4, pp. 215–218.

Wells, R. E., Haugerud, R. A., Niem, A. R., Niem, W. A., Ma, L., Evarts, R. C., O'Connor, J. E., Madin, I. P., Sherrod, D. R., Beeson, M. H., et al., 2020, Geologic map of the greater Portland metropolitan area and surrounding region, Oregon and Washington: U.S. Geological Survey Scientific Investigations Map 3443, pamphlet 55 pp., 2 sheets, scale 1:63,360, https://doi.org/10.3133/sim3443.

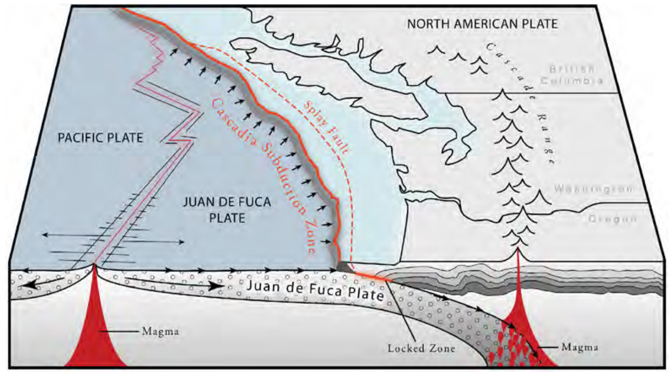
Witter, R. C., 1999, Late Holocene paleoseismicity, tsunamis and relative sea-level changes along the south-central Cascadia Subduction Zone, Southern Oregon: University of Oregon, unpublished PhD dissertation, 178 pp.

Witter, R. C., Kelsey, H. M., and Hemphill-Haley, E., 2003, Great Cascadia earthquakes and tsunamis of the past 6700 years, Coquille River estuary, southern coastal Oregon, *Geological Society of America Bulletin*, vol. 115, no. 10, pp. 1,289–1,306.



A) TECTONIC MAP OF PACIFIC NORTHWEST, SHOWING ORIENTATION AND EXTENT OF CASCADIA SUBDUCTION ZONE (MODIFIED FROM DRAGERT ET AL., 1994)

Cascadia Subduction Zone Setting



CASCADIA SUBDUCTION ZONE SETTING, TSUNAMI INUNDATION MAPS, OREGON DEPARTMENT OF GEOLOGY AND MINERAL INDUSTRY, 2013

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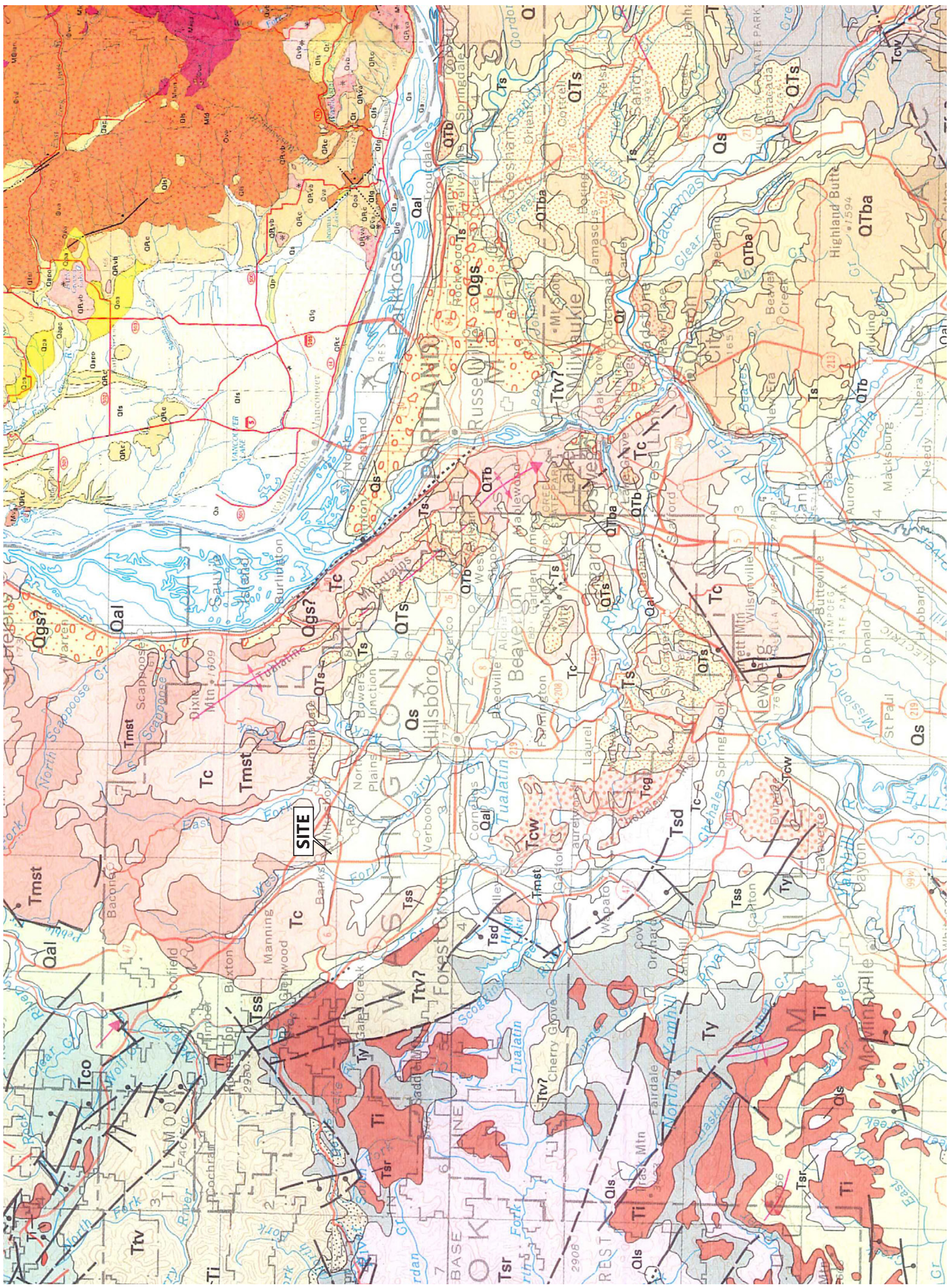
TECTONIC SETTING SUMMARY

NOV. 2024 | JOB NO. 6988-A | FIG. 1B

FROM:

WALSH, T. J., KOROSEC, M. A., PHILLIPS, W. M., LOGAN, R. L., AND SCHASSE, H. W., 1987, GEOLOGIC MAP OF WASHINGTON-SOUTHWEST QUADRANT; 1:250,000: WASHINGTON DIVISION OF GEOLOGY AND EARTH RESOURCES, 6M-34

WALKER, G. W., AND MACLEOD, N. S., 1991, GEOLOGIC MAP OF OREGON: U.S. GEOLOGICAL SURVEY



N

0 10 20 MILES

0 10 20 KILOMETERS

Contact — Approximately located

Fault — Dashed where inferred; dotted where concealed; queried where doubtful; ball and bar on downthrown side

Thrust fault — Dashed where inferred; dotted where concealed; queried where doubtful; sawteeth on upper plate

Strike and dip of bed

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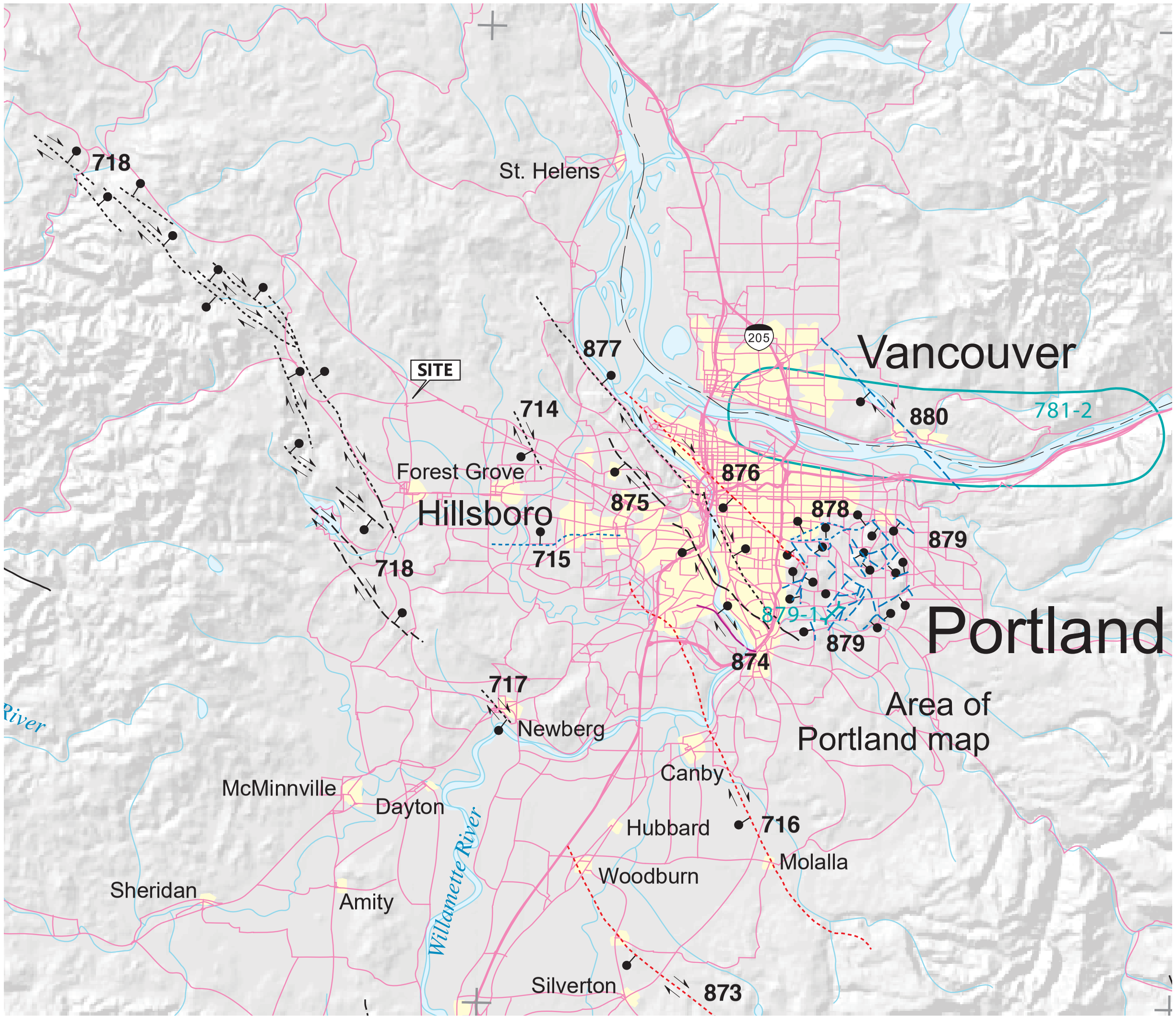
REGIONAL GEOLOGIC MAP

NOV. 2024 | JOB NO. 6988-A | FIG. 2B



MAP EXPLANATION

TIME OF MOST RECENT SURFACE RUPTURE
- Holocene (<10,000 years) or post last glaciation (<15,000 years; 15 ka); no historic ruptures in Oregon to date
- Late Quaternary (<130,000; post penultimate glaciation)
- Late and middle Quaternary (<750,000 years; 750 ka)
- Quaternary, undifferentiated (<1,600,000 years; <1.6 Ma)
- Class B structure (age or origin uncertain)

SLIP RATE
- >5 mm/year
- 1.0-5.0 mm/year
- 0.2-1.0 mm/year
- <0.2 mm/year

TRACE
- Mostly continuous at map scale
- Mostly discontinuous at map scale
- Inferred or concealed

STRUCTURE TYPE AND RELATED FEATURES
- Normal or high-angle reverse fault
- Strike-slip fault
- Thrust fault
- Anticlinal fold
- Synclinal fold
- Monoclinal fold
- Plunge direction of fold
- Fault section marker

DETAILED STUDY SITES
- 731-2 Trench site
- 781-2 Subduction zone study site

CULTURAL AND GEOGRAPHIC FEATURES
- Divided highway
- Primary or secondary road
- Permanent river or stream
- Intermittent river or stream
- Permanent or intermittent lake

FAULT NUMBER	NAME OF STRUCTURE
714	HELVETIA FAULT
715	BEAVERTON FAULT
716	CANBY-MOLALLA FAULT
717	NEWBERG FAULT
718	GALES CREEK FAULT ZONE
873	MOUNT ANGEL FAULT
874	BOLTON FAULT
875	OATFIELD FAULT
876	EAST BANK FAULT
877	PORTLAND HILLS FAULT
878	GRANT BUTTE FAULT
879	DAMASCUS-TICKLE CREEK FAULT ZONE
880	HAPPY CAMP FAULT

FROM: PERSONIUS, S. F., ET AL., 2003, MAP OF QUATERNARY FAULTS AND FOLDS IN OREGON, USGS OPEN FILE REPORT OFR-03-095.

N

0 12 24 MILES

0 12 24 KILOMETERS

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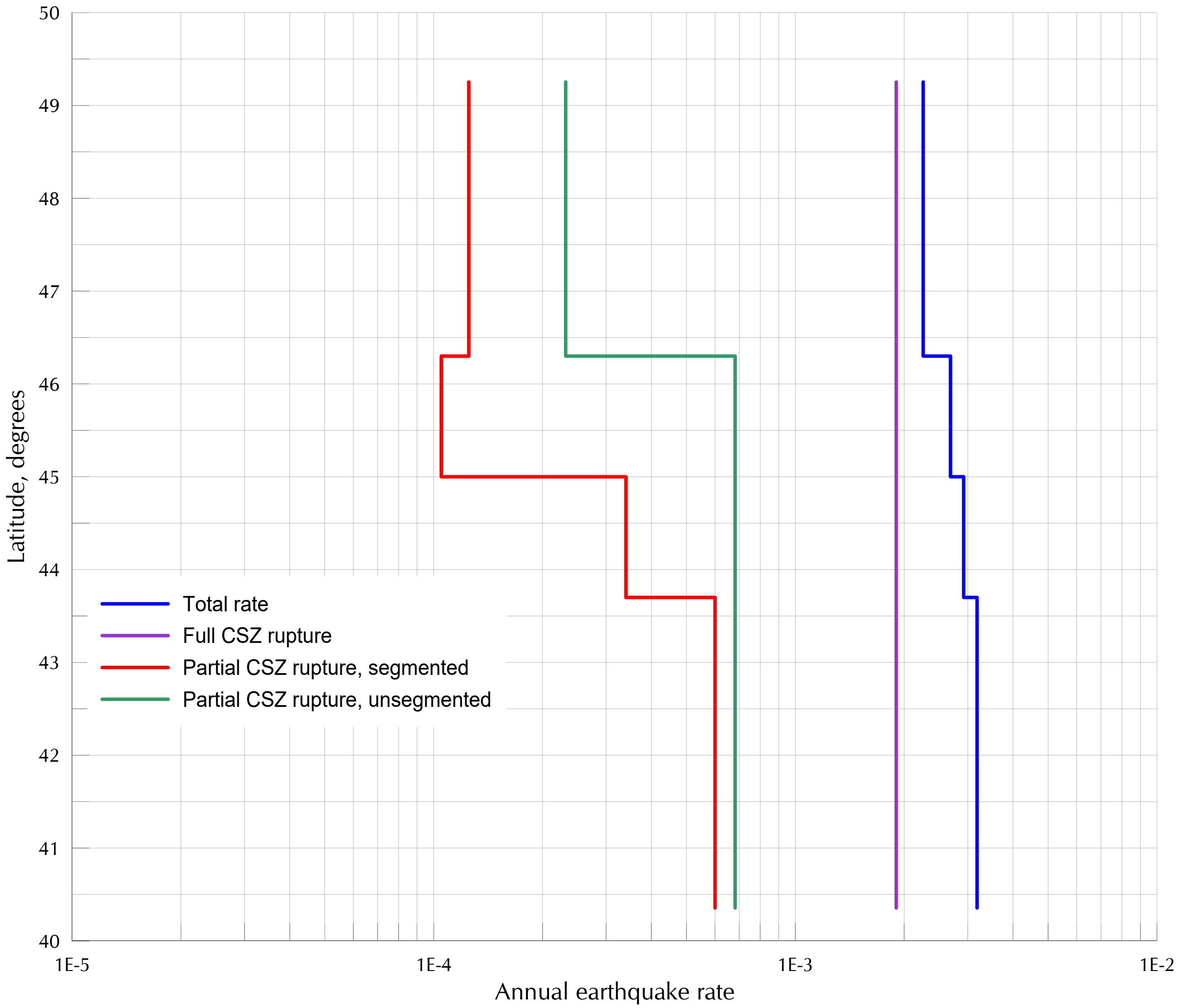
LOCAL FAULT MAP

NOV. 2024 JOB NO. 6988-A FIG. 3B



REFERENCE:
PETERSEN, M. D., MOSCHETTI, M. P., POWERS, P. M., MUELLER, C. S., HALLER, K. M., FRANKEL, A. D., ZENG, Y., REZAEIAN, S., HARMSEN, S. C., BOYD, O. S., FIELD, N., CHEN, R., RUKSTALES, K. S., NICO, L., WHEELER, R. L., WILLIAMS, R. A., AND OLSEN, A. H., 2014, DOCUMENTATION FOR THE 2014 UPDATE OF THE UNITED STATES NATIONAL SEISMIC HAZARD MAPS: U.S. GEOLOGICAL SURVEY OPEN-FILE REPORT 2014–1091, 243 pp.

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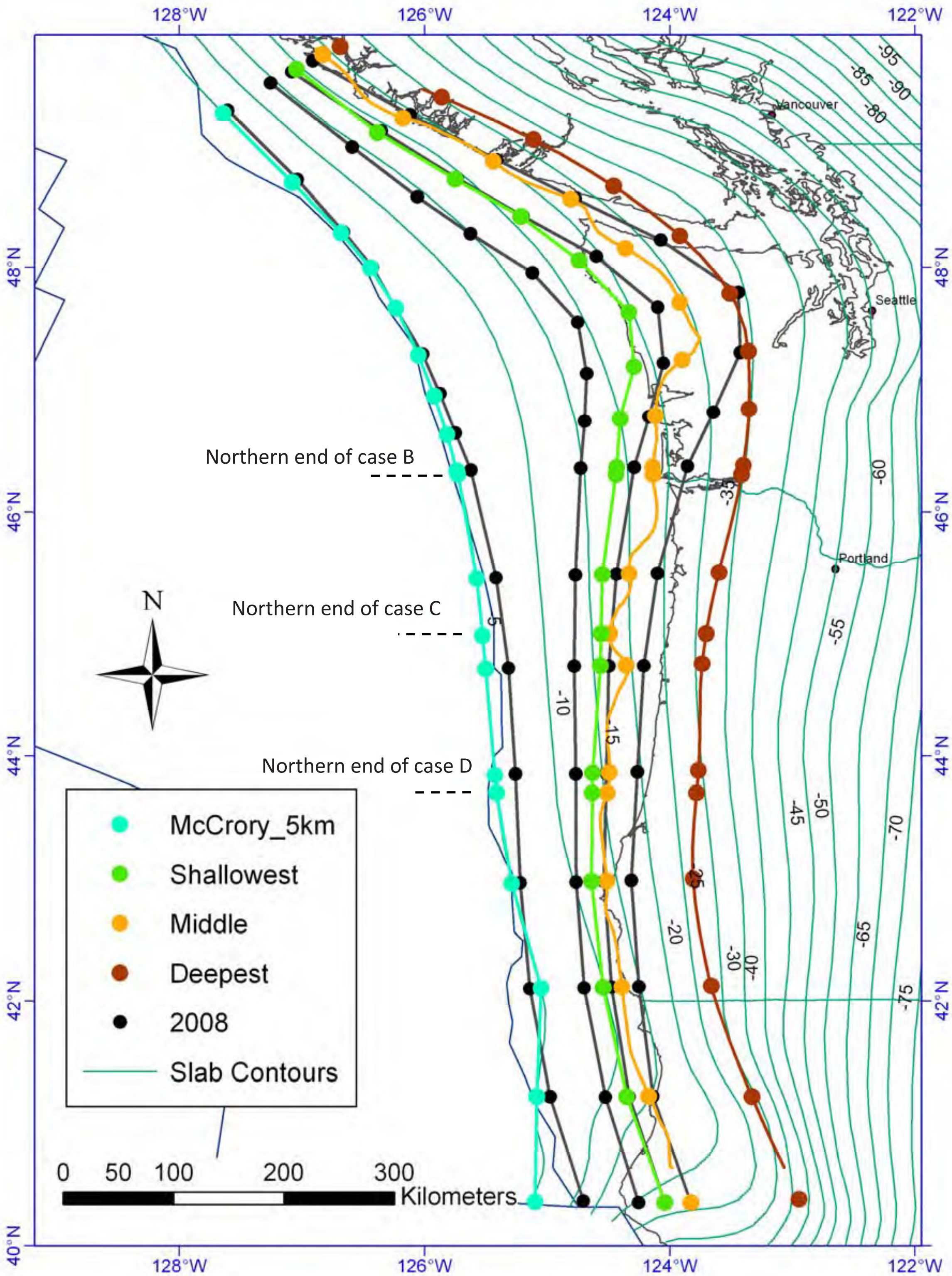
VARIATION OF EARTHQUAKE RATES
CASCADIA SUBDUCTION ZONE (CSZ)

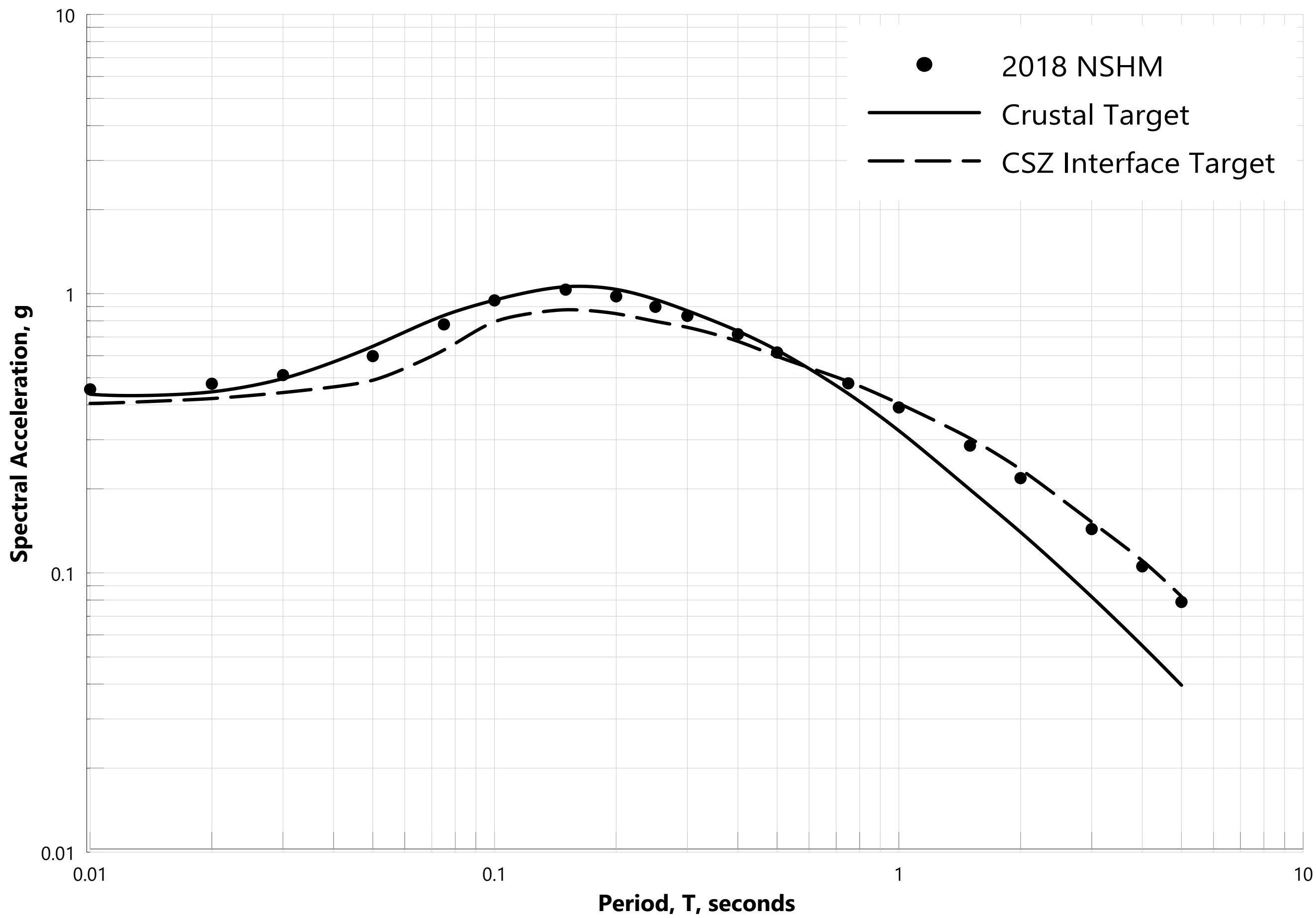
NOV. 2024 JOB NO. 6988-A FIG. 4B



GRI

LOCATION OF SURFACE TRACES FOR
UP-DIP EDGE & THREE DOWN-DIP EDGE
OPTIONS USED IN 2014 NSHMS
(CHEN ET AL., 2014)

NOV. 2024 | JOB NO. 6988-A | FIG. 5B

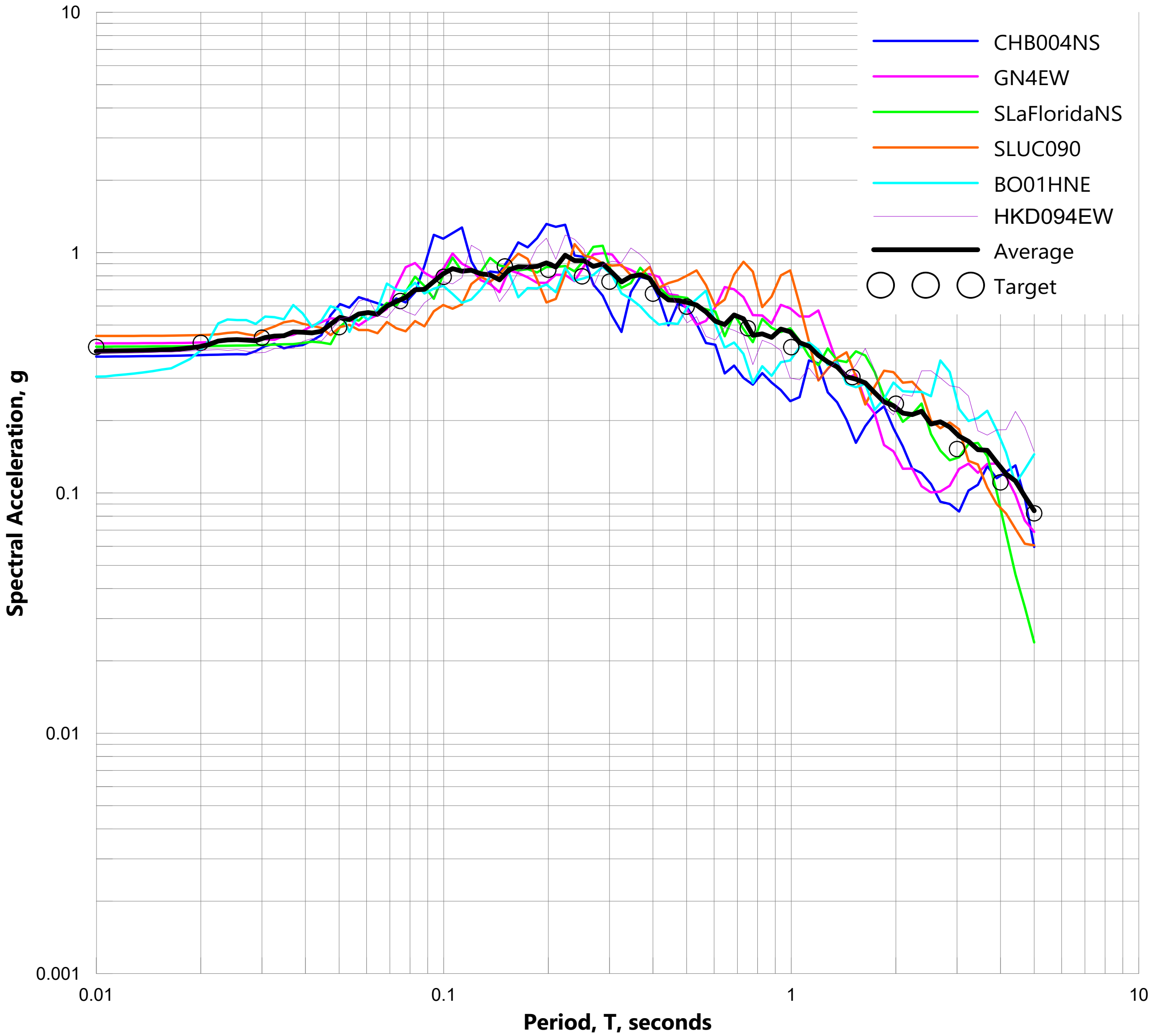


ABBREVIATIONS:
NSHM = NATIONAL SEISMIC HAZARD MAP
CSZ = CASCADIA SUBDUCTION ZONE

GRI

TARGET SPECTRA FOR B/C BOUNDARY CONDITION (5% DAMPING)

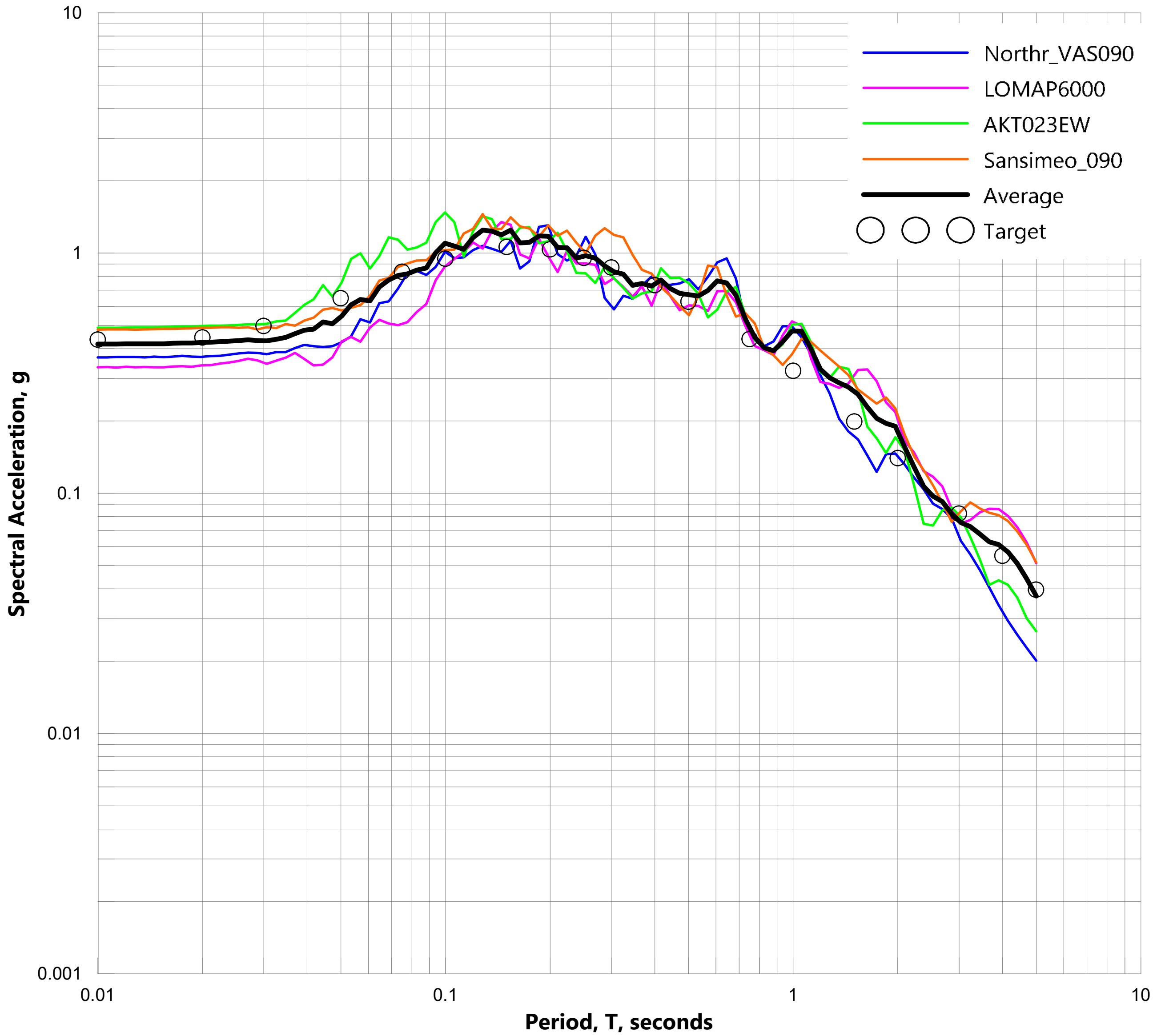
NOV. 2024 JOB NO. 6988-A FIG. 6B



GRI

AMPLITUDE-SCALED SUBDUCTION INTERFACE MOTIONS AND TARGET SPECTRA COMPARISON (5% DAMPING)

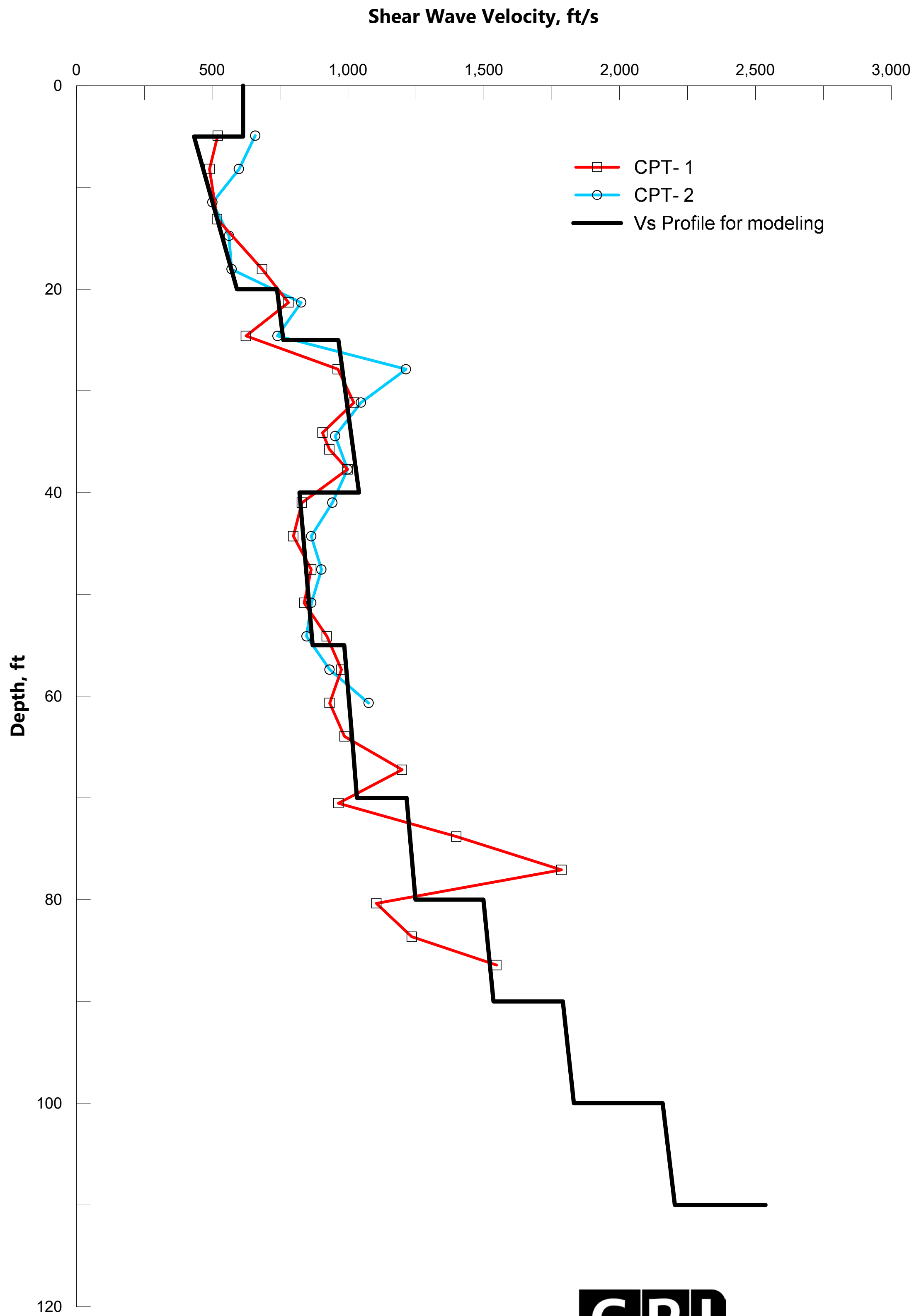
NOV. 2024 JOB NO. 6988-A FIG. 7B



GRI

AMPLITUDE-SCALED CRUSTAL MOTIONS
AND TARGET SPECTRA COMPARISON
(5% DAMPING)

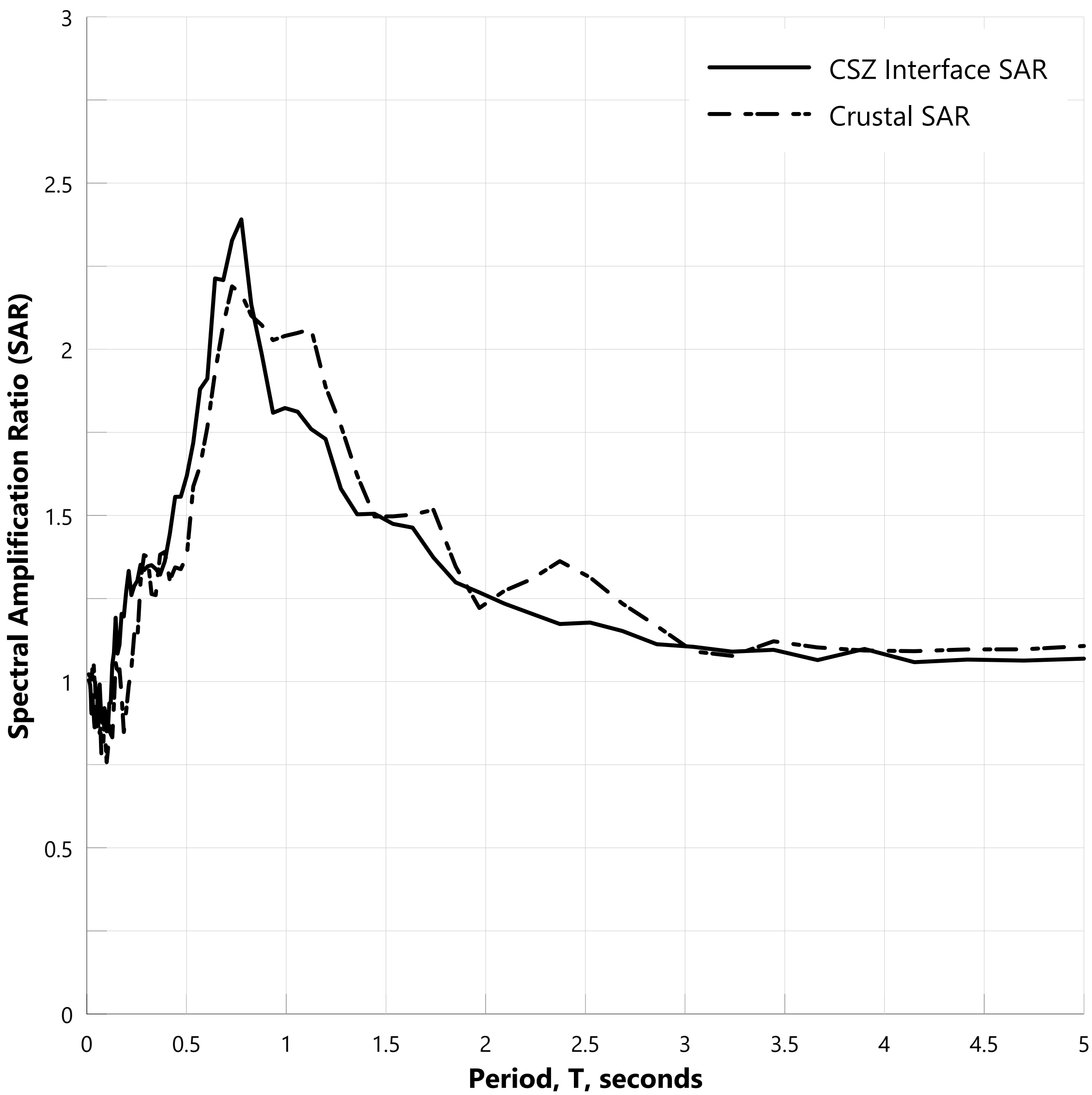
NOV. 2024 JOB NO. 6988-A FIG. 8B



GRI

SHEAR WAVE VELOCITY DATA

NOV. 2024 JOB NO. 6988-A FIG. 9B

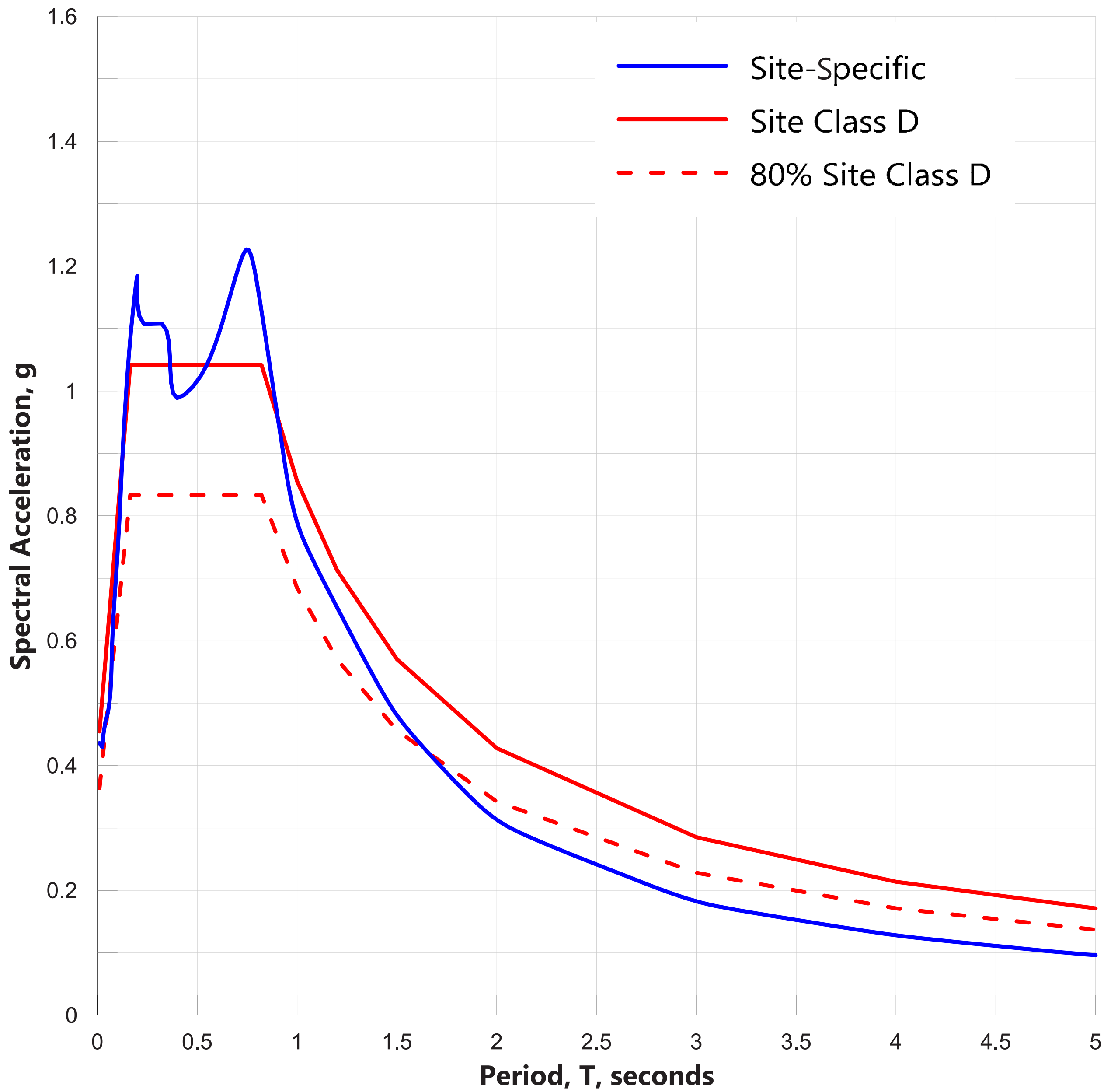


ABBREVIATIONS:
CSZ = CASCADIA SUBDUCTION ZONE
SAR = SPECTRAL AMPLIFICATION RATIO

GRI

MEAN SPECTRAL AMPLIFICATION RATIOS (5% DAMPING)

NOV. 2024 | JOB NO. 6988-A | FIG. 10B



GRI

GROUND SURFACE
SPECTRA COMPARISON
(5% DAMPING)

NOV. 2024 JOB NO. 6988-A FIG. 12B

GRI

APPENDIX C

Geoprofessional Business Association Guidance Document

Important Information about This Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

Appendix

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NW MAIN ST
NW MAIN ST
ELEMENTARY SCHOOL
MIDDLE SCHOOL
AUX
DISTRICT
CTE
HIGH SCHOOL
Cafeteria
Gym
Band/ Choir
NW OAK WAY
NW TRELLIS WAY
NW WILKES ST
NW PARK ST
Storm Retention Pond
WETLANDS BUFFER
WETLANDS BUFFER
GRANDSTANDS
WETLANDS
WETLANDS
NW PARMLEY ST
NW OAK WAY

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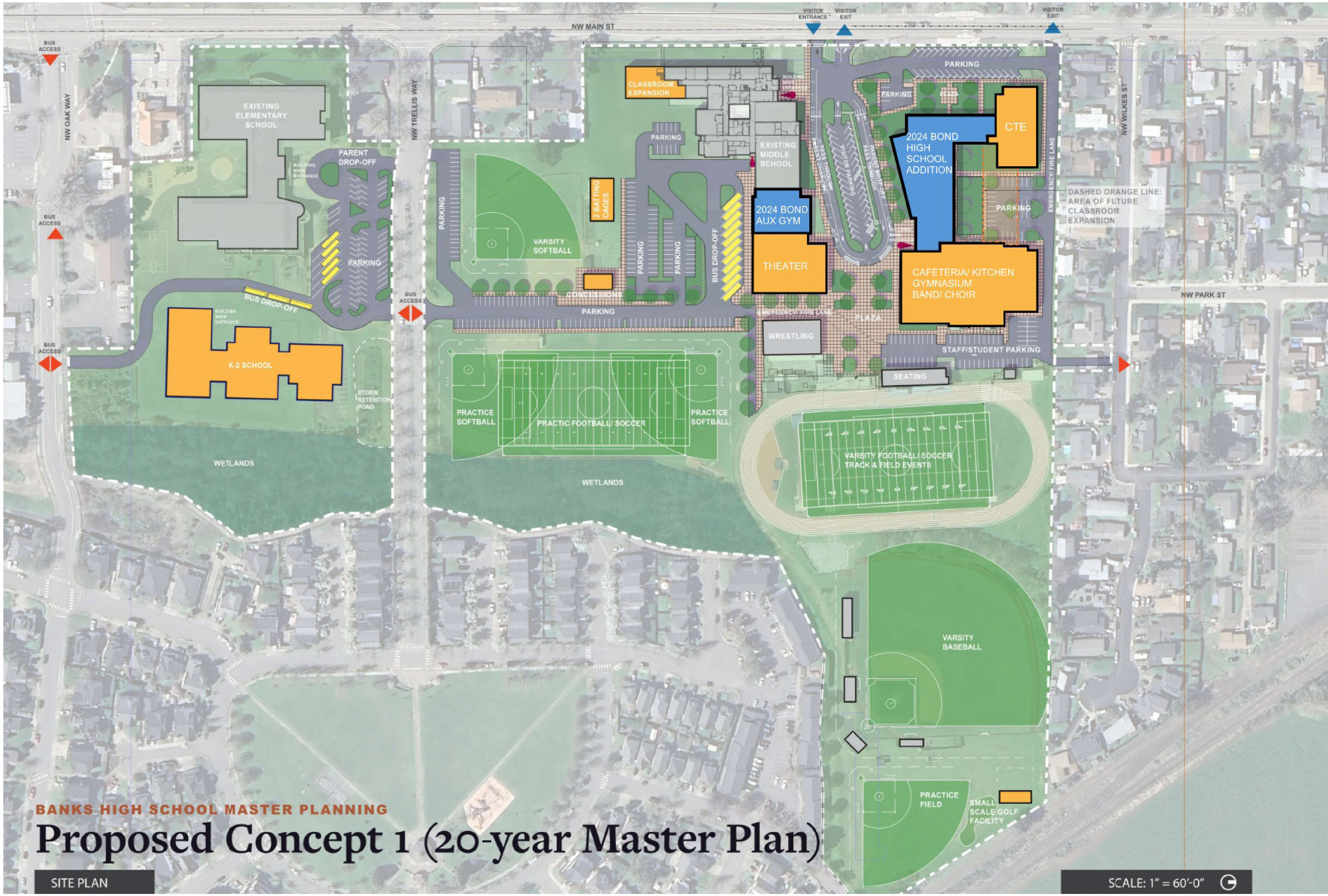
BANKS HIGH SCHOOL MASTER PLANNING
Proposed Concept 1 (20-year Master Plan)
SITE PLAN
SCALE: 1" = 60'-0"
NW MAIN ST
NW OAK WAY
NW TRELLIS WAY
NW WILKES ST
NW PARK ST
BUS ACCESS
VISITOR ENTRANCE
VISITOR EXIT
EXISTING ELEMENTARY SCHOOL
PARENT DROP-OFF
PARKING
BUS DROP-OFF
K-2 SCHOOL
STORM RETENTION POND
WETLANDS
CLASSROOM EXPANSION
EXISTING MIDDLE SCHOOL
VARSITY SOFTBALL
2 BATTING CAGES
CONCESSIONS
BUS DROP-OFF
2024 BOND AUX GYM
THEATER
2024 BOND HIGH SCHOOL ADDITION
CTE
CAFETERIA/ KITCHEN
GYMNASIUM
BAND/ CHOIR
EMERGENCY/ FIRE LANE
DASHED ORANGE LINE: AREA OF FUTURE CLASSROOM EXPANSION
PLAZA
WRESTLING
STAFF/STUDENT PARKING
SEATING
PRACTICE SOFTBALL
PRACTIC FOOTBALL/ SOCCER
VARSITY FOOTBALL/ SOCCER TRACK & FIELD EVENTS
VARSITY BASEBALL
PRACTICE FIELD
SMALL SCALE GOLF FACILITY

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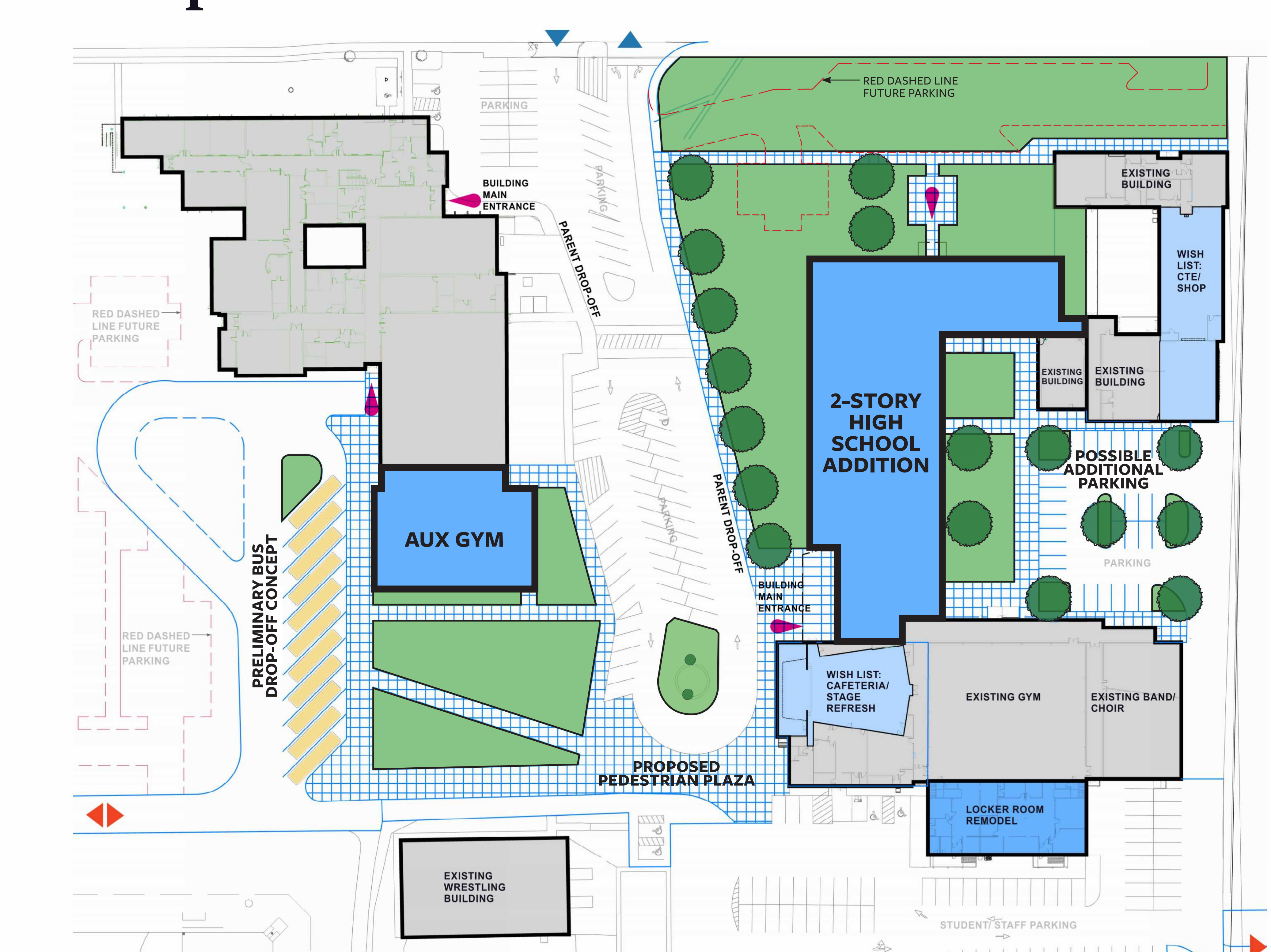
BANKS HIGH SCHOOL MASTER PLANNING

Concept 1



SITE PLAN

NOT TO SCALE

NEW ADDITION

43,000 SF OF PROGRAM IN NEW ADDITION	
43,000 SF of Program x $740/SF = $31,820,000	
TOTAL CONSTRUCTION BUDGET FOR CONCEPT 1 = $31,820,000	

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NW MAIN ST
VISITOR ENTRANCE
VISITOR EXIT
VISITOR EXIT
BUS ACCESS
NW OAK WAY
NW TRELLIS WAY
NW WILKES ST
NW PARK ST
EXISTING ELEMENTARY SCHOOL
PARENT DROP-OFF
PARKING
BUS DROP-OFF
K-2 SCHOOL
STORM RETENTION POND
WETLANDS
CLASSROOM EXPANSION
EXISTING MIDDLE SCHOOL
PARKING
2 BATTING CAGES
VARSITY SOFTBALL
CONCESSIONS
BUS DROP-OFF
2024 BOND AUX GYM
THEATER
2024 BOND HIGH SCHOOL REMODEL
CTE
2024 BOND HIGH SCHOOL ADDITION
CAFETERIA/ KITCHEN
GYMNASIUM
BAND/ CHOIR
EMERGENCY FIRE LANE
PLAZA
WRESTLING
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SEATING
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PRACTICE SOFTBALL
PRACTIC FOOTBALL/ SOCCER
PRACTICE SOFTBALL
VARSITY FOOTBALL/ SOCCER TRACK & FIELD EVENTS
WETLANDS
VARSITY BASEBALL
PRACTICE FIELD
SMALL SCALE GOLF FACILITY
BANKS HIGH SCHOOL MASTER PLANNING
Proposed Concept 2 (20-year Master Plan)
SITE PLAN
SCALE: 1" = 60'-0"

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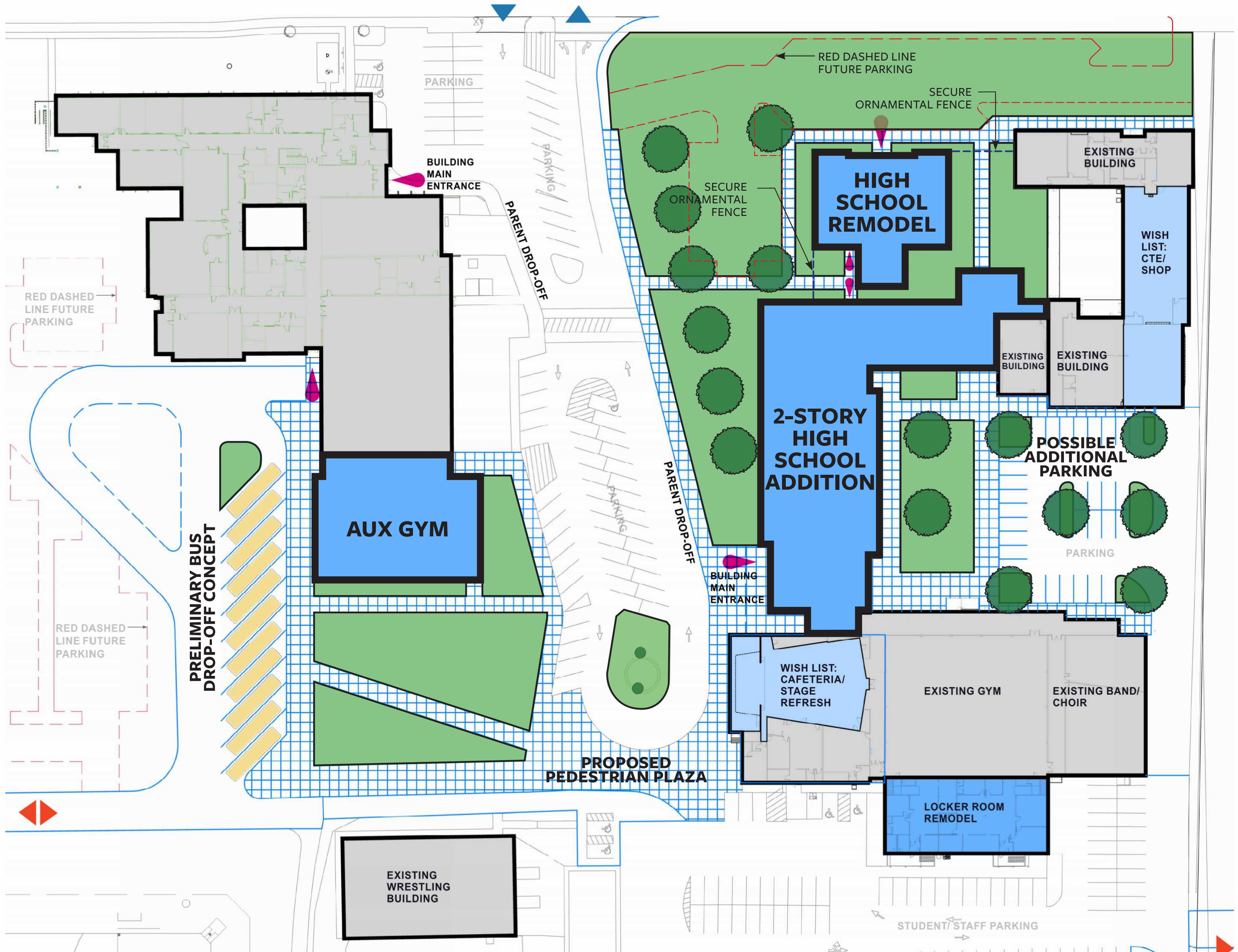
BANKS HIGH SCHOOL MASTER PLANNING

Concept 2



SITE PLAN

NOT TO SCALE

REMODEL OF DISTRICT OFFICE

6,000 SF (APPROX.) OF PROGRAM BUT REQUIRES FULL REMODEL OF ENTIRE 10,500 SF BUILDING

10,500 SF of Program x $800/SF = $8,400,000

NEW ADDITION

37,000 SF OF PROGRAM IN NEW ADDITION

37,000 SF of Program x $740/SF = $27,380,000

TOTAL CONSTRUCTION BUDGET FOR CONCEPT 2 = $35,780,000

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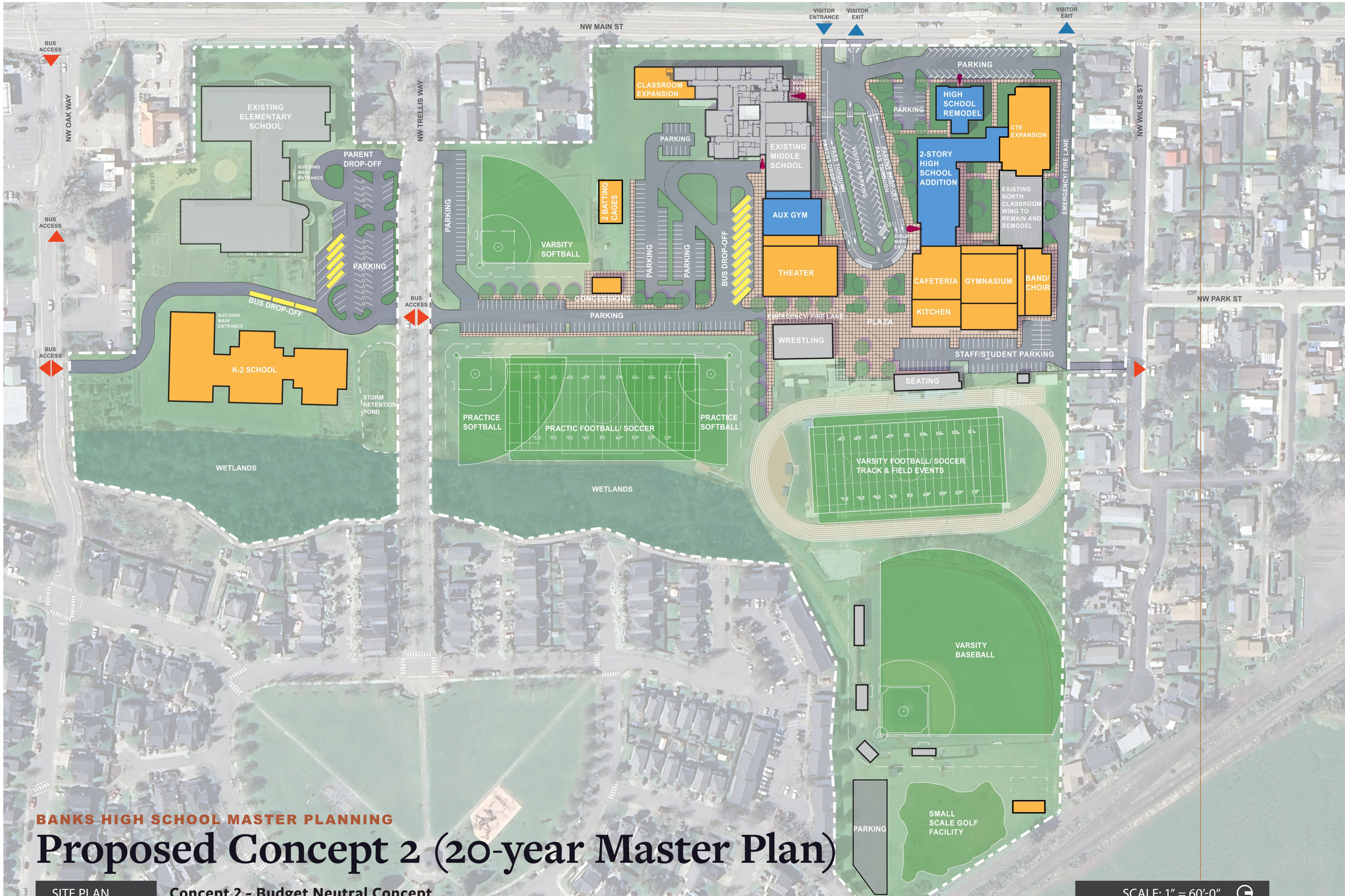
NW MAIN ST
VISITOR ENTRANCE
VISITOR EXIT
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BUS ACCESS
NW OAK WAY
NW TRELLIS WAY
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NW PARK ST
EXISTING ELEMENTARY SCHOOL
PARENT DROP-OFF
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STORM RETENTION POND
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EXISTING MIDDLE SCHOOL
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VARSITY SOFTBALL
CONCESSIONS
AUX GYM
THEATER
WRESTLING
EMERGENCY/ FIRE LANE
PLAZA
HIGH SCHOOL REMODEL
CTE EXPANSION
2-STORY HIGH SCHOOL ADDITION
EXISTING NORTH CLASSROOM WING TO REMAIN AND REMODEL
CAFETERIA
GYMNASIUM
BAND/ CHOIR
KITCHEN
STAFF/STUDENT PARKING
SEATING
PRACTICE SOFTBALL
PRACTIC FOOTBALL/ SOCCER
PRACTICE SOFTBALL
VARSITY FOOTBALL/ SOCCER TRACK & FIELD EVENTS
VARSITY BASEBALL
SMALL SCALE GOLF FACILITY
BANKS HIGH SCHOOL MASTER PLANNING
Proposed Concept 2 (20-year Master Plan)
SITE PLAN
Concept 2 - Budget Neutral Concept
SCALE: 1" = 60'-0"

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