Seismic Evaluation For:

MEDFORD SCHOOL DISTRICT 549C

Jackson County, Oregon

Ron Havniear, Executive Director Of Facilities And Leadership Development
680 Biddle Rd, Medford, OR 97504

STRUCTURAL REGISTERED PROFESSIONAL ENGINEER 70881 OREGON July 08, 2014 MATTHEW R. SMITH

EXPIRES: 06-30-24

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Medford SCHOOL DISTRICT
ALL ARE LEARNING & LEARNING IS FOR ALL

April, 2022

ZCS ENGINEERING ARCHITECTURE

Contents

1 Executive Summary

1.1 Background

The Medford School District 549C (District) is centrally located in Medford, Oregon in Jackson County and is the largest school district in the county. Excluding charter schools, the District operates out of 14 elementary schools, 3 middle schools, 3 high schools, and supporting District buildings.

The purpose of this report is to provide a planning level seismic evaluation of existing educational facilities. The facilities are used for classrooms, administrative offices, and assembly areas. To complete this report, ZCS has reviewed available construction documents and/or performed visual observations at each of the facilities listed below. The team identified general structural deficiencies and performed a review of the expected seismic performance of the structural system. The analysis provided is schematic and is meant to aid in planning purposes, rather than provide a comprehensive evaluation of each structure. The report includes a brief description of observed structural deficiencies and corresponding recommendations for each school and support facility. This report may be used by the District to prioritize structural improvements and determine interest in seeking grant funding through the seismic rehabilitation grant programs. Additionally, this report may be used to aid in planning seismic improvements as directed by ORS 455.400. This statute mandates that school districts address structures (either through rehabilitation or other action to reduce risk) that pose undue risk to life safety in the event of an earthquake. These improvements shall be made by January 1, 2032, subject to available funding. To date, the District has undertaken a number of seismic improvement projects and this report also seeks to record and track these improvements.

The schools and support facilities included in this study are listed below. The structures vary in style, age, condition, and use, and each campus has undergone multiple additions and remodels.

- Abraham Lincoln Elementary School
- Griffin Creek Elementary School
- Hoover Elementary School
- Howard Elementary School
- Jackson Elementary School
- Jacksonville Elementary School
- Jefferson Elementary School
- Kennedy Elementary School
- Lone Pine Elementary School
- Oak Grove Elementary School
- Roosevelt Elementary School
- Ruch Community School
- Washington Elementary School
- Wilson Elementary School

- Hedrick Middle School
- Mcloughlin Middle School
- Oakdale Middle School (Old Central High School)

- North Medford High School
- South Medford High School

- District Admin/Maintenance Building
- District Distribution Center Building

1.2 Definitions and Terms

The following terms are meant to support the reader in understanding the intent and results of the assessment.

- **ASCE 41 Seismic Evaluation and Retrofit of Existing Buildings**. A standard produced by the American Society of Civil Engineers (ASCE) that is the commonly adopted code for the seismic evaluation of existing structures in the United States. All section references and excerpts in this report reference the 2017 edition, ASCE 41-17.
- **Tier 1 Evaluation**. A process and checklist provided by ASCE 41 that is utilized to determine if a structure can be expected to meet the Life Safety performance level.
- **Tier 2 Evaluation**. An additional level of evaluation used to confirm or resolve the deficiencies screened by the Tier 1.
- **Life Safety Structural Performance Level**. *Structural Performance Level S-3, Life Safety, is defined as the post-earthquake damage state in which a structure has damaged components but retains a margin of safety against the onset of partial or total collapse. A structure in compliance with the acceptance criteria specified in this standard (ACSE 41-17) for this Structural Performance Level is expected to achieve this state* (ASCE 41-17, Section 2.3.1.3). Attaining Life Safety requires compliance with both structural and non-structural provisions and can be determined by completion of the Tier 1 and associated field work, completion of the Tier 2 evaluation and associated field work, or by determining the structure classifies as a benchmark building and performing the associated fieldwork. Attaining a Life Safety designation also requires addressing non-structural elements such as suspended ceilings, chimneys, MEP, and similar elements that may pose a risk during a seismic event. Conformance to a *Life Safety* determination for each campus (or portion thereof) may be evidenced by a letter from a registered design professional stating such, or by the original construction documents for modern benchmark construction. This letter or similar documentation can be provided by the original Engineer of Record, or by a different registered design professional after completion of the ASCE 41 evaluation procedures.

- **Immediate Occupancy Structural Performance Level**. *Structural Performance Level, S-1, Immediate Occupancy, is defined as the postearthquake damage state in which a structural remains safe to occupy and essentially retains its preearthquake strength and stiffness. A structure in compliance with the acceptance criteria of this standard for Immediate Occupancy is expected to achieve this postearthquake state.* (ASCE 41-17, Section 2.3.1.3). As with Life Safety, attaining full designation requires compliance with both structural and non-structural provisions. Structures compliant with Immediate Occupancy criteria would be expected to meet or exceed the performance of structures meeting the Life Safety criteria.
- **Relative Hazard Severity**. A metric used by Federal Emergency Management Agency (FEMA) to quantify the probability of catastrophic damage to a structure during a seismic event. This is typically used to help identify structures that are potentially hazardous during a seismic event. The method used is called a Rapid Visual Assessment, and the results help determine the hazard value. These scores were provided for District facilities during the Statewide Seismic Needs Assessment (~2005) by the State of Oregon Department of Geology and Mineral Industries (DOGAMI). Where buildings have changed significantly or additional information was made available, ZCS has modified the severity rating based upon our observations of the structure and past evaluation experience. In our opinion, the scores approximately conform to the following behaviors. **Very High** indicates buildings or portions thereof that have a very high potential for collapse when exposed to a code level seismic event. **High** indicates buildings or portions thereof that have a high potential for collapse when exposed to a code level seismic event. **Moderate** corresponds to a structure that will experience damage during code events but with a reduced likelihood of collapse. **Low** indicates a structure which will experience damage, but collapse is unlikely**. L.S. Compliant** indicates the structure meets the structural Life Safety requirements, either because it was designed with a benchmark code, has been previously seismically rehabilitated, or has passed the ASCE 41 analyses. Performance of a L.S. compliant structure is as described above.
- **Benchmark Building.** A building that was designed using a code that meets or exceeds the provisions for Life Safety performance. The year of Benchmark codes varies per building type and is determined by ASCE 41. See Appendix C for more information. Per ASCE 41, fieldwork is required to fully confirm compliance with a Life Safety designation when evaluating a benchmark building.
- **Building Type**. This is a shorthand method to describe the type of building system being evaluated. Definitions of building types have been included in Appendix B.
- **Soil Liquefaction.** This is a condition where soils lose substantial strength and stiffness in response to shaking. This typically occurs in sandy soil saturated with water. Areas with potentially liquefiable soils have been cataloged by DOGAMI (which serves as the basis for Appendix A soil notes). Liquefaction can be confirmed or disconfirmed by a site-specific geotechnical report. Resolution of liquification potential is a requirement of the Tier 1 analysis.

- **Soil Landslide Susceptibility.** This condition occurs where a soil or slope has the potential to move laterally during a seismic event. Areas that have topography or soil types susceptible to this condition have been cataloged by DOGAMI (which serves as the basis for Appendix A soil notes). Landslide stability can be confirmed or disconfirmed by a site-specific geotechnical report. Resolution of landslide susceptibility is a requirement of the Tier 1 analysis.

1.3 Summary of Evaluation Results

The following table summarizes the results of our evaluations and ranks each campus based on the relative hazard severity of the observed deficiencies at the present time. This is a campus level aggregation based on the campus buildings, and building specific information is available in the campus specific report sections and Appendix A.

School	Structural Life Safety Compliant	Relative Hazard Severity[1]
Abraham Lincoln Elementary School	Noncompliant	Low
Griffin Creek Elementary School[2]	Noncompliant	Moderate
Hoover Elementary School[4]	Compliant	L.S. Compliant[4]
Howard Elementary School[2,5]	Noncompliant	Moderate
Jackson Elementary School[2]	Noncompliant	Low
Jacksonville Elementary School[5]	Noncompliant	High
Jefferson Elementary School	Noncompliant	Low
Kennedy Elementary School[5]	Noncompliant	Moderate
Lone Pine Elementary School[2]	Noncompliant	Low
Oak Grove Elementary School[2]	Noncompliant	Moderate
Roosevelt Elementary School	Noncompliant	Low
Ruch Community School[2]	Noncompliant	Moderate
Washington Elementary School[2]	Noncompliant	Moderate
Wilson Elementary School	Noncompliant	Moderate
Hedrick Middle School[5]	Noncompliant	Moderate
Mcloughlin Middle School	Noncompliant	Low
Oakdale Middle School (Old Central High School)[6]	Noncompliant[6]	Moderate[6]
North Medford High School	Noncompliant	Moderate
South Medford High School[2]	Compliant	L.S. Compliant
District Admin/Maintenance Building[3]	Noncompliant	High
Distribution Center Building[3]	Noncompliant	Moderate

1. Relative Hazard Severity level has been provided based upon an aggregation of the hazard risks across the campus. Portions of the campus may not share this hazard severity, and building-specific scores have been provided in the school specific sections.
2. These campuses have some buildings or structures that meet the compliance criteria as defined in this report.

3. The Admin/Maintenance Facility and Distribution Center are not eligible for a Seismic Retrofit Grant Application (SRG). At this time, SRG applications are limited to educational and emergency services buildings and do not include ancillary or supporting service buildings.
4. At time of writing this campus is undergoing a seismic retrofit project that has attained substantial completion. The anticipated effects of this project are represented in the current assessment summary, reflected in Section 3.0, and in the recommended prioritization list.
5. This campus or a portion thereof has applied for a Seismic Rehabilitation Grant from the IFA.
6. At time of writing this campus is undergoing a renovation and seismic retrofit project. The anticipated effects of this project are represented in the current assessment summary, reflected in Section 3.0, and in the recommended prioritization list.

1.4 Recommendations

Our recommendation is to prioritize campuses (or portions thereof) possessing High or Very High hazard severity ratings. Below is a list of schools we recommend prioritizing with respect to seismic upgrades. These recommendations take into account ongoing (2021-2022) Seismic Retrofit projects. Section 3.0 covers the specific structural deficiencies and subsequent recommendations observed in the schools and support facility structures.

- Jacksonville Elementary
- Medford School District Administration Building

Beyond these campuses, we do also recommend pursuing seismic hazard mitigation for all campuses that have a campus-wide Moderate rating, or for structures that have a High to Moderate rating. These include:

- Griffin Creek Elementary
- Howard Elementary School
- Kennedy Elementary School
- Hedrick Middle School
- McLoughlin Middle School
- Ruch Community School
- North Medford High School
- Distribution Center Building

In conjunction with the efforts above, ZCS also recommends taking steps to establish or confirm Low hazard buildings as meeting the Life Safety designation. These efforts are often lower cost than mitigation of severe hazards and in some cases may only require additional documentation, calculation, or field confirmation/destructive investigation. More detailed descriptions of the recommended steps are discussed in Sections 3 and 4.

A number of campus buildings were retrofit during the bond campaign around 2008. Depending on project funds available at time of construction, these retrofits may have been limited to a hazard reduction approach which resulted in a Low designation, or designed to more stringent standards that meet or exceed the current Life Safety designation. Given the documentation currently available, additional information from the Architect or Engineer of Record such as a confirmation letter or project calculation package would be needed to confirm the Life Safety designation. For the initial effort, we recommend pursuing additional retrofit documentation for the following campuses:

- Jackson Elementary School
- Lone Pine Elementary School
- Roosevelt Elementary School
- Washington Elementary School
- Wilson Elementary School
- North Medford High School

Additionally, there are campuses that contain Low hazard designated structures (such as wood frame single story buildings) that may meet the requirements for Life Safety pending completion of the ASCE 41 required field verification, soil hazard review, non-structural element review, and destructive investigation. To begin this effort, campuses with structures that we believe are suitable for the additional investigation and review are:

- Kennedy Elementary
- Roosevelt Elementary

ZCS performed site investigations and obtained additional seismic retrofit documentation where available to determine the extent of seismic retrofit installed during the bond campaign around 2008. The intent of this phase is to identify remaining seismic deficiencies to assist the District in planning next steps. The additional investigations include the following campuses:

- Jackson Elementary School
- Lone Pine Elementary School
- Roosevelt Elementary School
- Washington Elementary School
- Wilson Elementary School
- North Medford High School

1.5 Conclusions

The following is a brief summary of the major report conclusions.

- The campus buildings are in generally good condition. The team did not identify a substantial immediate risk to the safety of occupants outside of a code lateral event.
- The buildings are at varying levels of seismic risk. For large seismic rehabilitation efforts, we recommend prioritizing these in order of their relative hazard severity.
- For Low or Life Safety designated buildings, we recommend the District proceed with the lower level efforts to complete the Life Safety designation.

Generally speaking, the condition of the District's schools and support facilities are good based on their respective ages. The recommended improvements listed above reflect items that do not pose a substantial immediate risk to the safety of occupants (unless noted otherwise) outside of code lateral events. It should be noted that structural deficiencies in schools of this age group are fully expected, and the severity of the deficiencies noted above is not uncommon.

To ensure that the District continues to get the most out of their schools and support facilities and to provide a safe learning environment for the students, we recommend generating a priority list for capital improvement projects to systematically address deficiencies as funds become available. This report may be used to aid in that effort. Additionally, incremental improvements should be identified and considered during projects that may make performing the work easier. For example, a roof replacement project is a good opportunity to install connections from the roof diaphragm to the walls, or a window replacement project is a good opportunity to install shearwalls in place of windows.

In addition to the District's self-directed efforts, ORS 455.400 mandates that school districts address structures (either through rehabilitation or other action to reduce risk) that pose undue risk to student safety in the event of an earthquake. This statute directs these actions to be taken prior to January 1, 2032 subject to available funding.

Senate Bills 4 and 5 in Oregon authorized the state to issue bonds as the funding mechanism for a program to fund the seismic rehabilitation of schools and emergency services buildings. Upcoming seismic retrofit grant programs may be a good opportunity for the District. Several of the schools noted above are good candidates, and successful grant applications can fund some or all of the expenses related to seismic retrofit of school buildings. Schools that have had a grant application submitted to the Infrastructure Finance Authority have been noted in the summary table in Section 1.3 with a more detailed list in Section 4.

The seismic hazard determinations made in this report are intended to help the District understand the overall seismic risk status of each campus. Designation and prioritization rank are subject to change based upon field verification, additional information becoming available, or the completion capital improvement or seismic retrofit projects.

The balance of the report provides specific details regarding the construction of each school, observed deficiencies, and recommended repairs. Given the ongoing and potential future rehabilitation projects, we recommend this report be updated periodically in order to keep campus tracking and project prioritization up to date.

2 Project Overview

The Medford School District 549C is located in a high seismicity zone and contains multiple schools; nineteen schools and two district support facilities are the focus of this evaluation. The objective of this effort is to perform visual observations and/or review available construction documents at each of the above-mentioned schools to identify general structural deficiencies and perform a review of the expected seismic performance of the structural system. The intent is to screen for and prioritize potentially hazardous structures. The study provides a brief discussion of observed structural deficiencies and corresponding recommendations. This evaluation may be used by the District to prioritize structural improvements and determine interest in seeking grant funding through the seismic rehabilitation grant programs.

The facilities covered by this evaluation total over 1,800,000 square feet and are used as elementary, middle, and high schools, as well as District support facilities. The age of each school and their additions are included and reflect the best information available at time of writing. The facilities serve as classrooms, administrative space, and assembly areas. The evaluations were developed through site observations and review of available record drawings.

While each school was constructed differently, access to their structural systems was limited to observation only. No destructive investigation was performed. Observed construction type for each school and a summary of each facility's additions and their respective construction types are located in Section 3.

2.1 Inspection and Evaluation Process

The following sections detail the inspection process, the individuals who participated in the inspections, and our methodology for review of deficiencies.

2.1.1 Inspection Process

An initial facility assessment was provided for the District in August of 2017. That report addressed twelve schools and two support facilities. This report builds upon those findings, including the addition of seven more schools and updating sections for work that has been completed. Investigation for the additional schools was performed utilizing existing record drawings when available, with site visits providing supplemental information.

2.1.2 Participants

For the initial report, a detailed inspection effort was performed utilizing several individuals offering different perspectives and areas of expertise. Inspections were performed on August 24th, October 14th, October 26th, November 4th, and November 10th, 2016, December 2018, and January 2019. For this update, additional document review and site visits were performed September through December 2021 and January 2022 through February 2022. In addition, geotechnical evaluations were provided by the Galli Group at various sites to verify/determine the extent of site seismic hazards. These site hazards are reflected in Appendix A and Appendix D.

A list of those who participated from ZCS Engineering in the inspection process is provided in the table below:

Name	Role
Stephen Chase	Drawing Review, Site Observations & Evaluations
Jacob Coppola	Drawing Review, Site Observations & Evaluations
Sylas Allen, PE	Site Observations & Evaluations
Matthew Smith, PE SE	Review
Russell Carter, PE SE	Review

2.1.3 Evaluation & Deficiency Lists

Evaluation of the structures was provided utilizing the Tier 1 checklist (ASCE 41), RVS score, and referencing benchmark codes. When applicable, RVS scores from DOGAMI were utilized. **Note:** This evaluation is intended as a planning and prioritization tool. The evaluations were limited to structural components and record drawing information available at time of writing and did not include destructive investigations. Some structures may have their designation improved as more information becomes available. To achieve full Life Safety designation for existing structures, ASCE 41 requires certain actions such as field verification (investigative demolition), condition assessment, geologic site hazard assessment, and non-structural hazard assessment.

The report provides a brief description of the deficiencies observed during our on-site investigation and/or record drawing review for each school. The deficiencies correspond to items outlined in ASCE 41: Seismic Evaluation and Retrofit of Existing Buildings where applicable and include areas where additional information is required. They are not intended to be an exhaustive list. A summary of each building's structural systems and observed deficiencies is provided in Section 3. Some deficiencies noted in Section 3 may be resolved upon further exploratory inspection and analysis. Conversely, there are potentially additional deficiencies that may be present upon further inspection and analysis. Additional deficiencies can be identified and further discussed in a detailed campus or building-specific seismic evaluation. Deficiencies vary in severity depending on construction type and lateral force resisting system. The seismic hazard level attempts to summarize the severity and potential risk of the deficiency.

3 Structure Summaries, Observed Deficiencies, and General Recommendations

The information obtained through review of the record drawings and the on-site observations is summarized below. A general summary of each portion of the campus is provided followed by a table summarizing the deficiencies observed. Lastly, a list of repair recommendations is provided. For clarity and visualization purposes, **the reader may find it useful to print Appendix A separately to view the campus maps corresponding with the campus information.**

3.1 Abraham Lincoln Elementary School

Figure 1: Abraham Lincoln Elementary School

3.1.1 Seismic Evaluation Summary

The following table summarizes the building type, age, and construction for structures on this campus. It also includes a summary of the seismic assessment findings. Please see Appendix A for a campus map summarizing the campus construction and evaluation results.

Building Label (RVS/ZCS)	Building Type	Year Built, Remodel, and/or Retrofit	Structural Life Safety Assessment	Relative Hazard Severity
School Campus [A]	PC2/RM1	1996	Noncompliant	Low

3.1.2 Structure Summary

The following summarizes the building timeline and structural systems for each portion of Abraham Lincoln Elementary School:

- **Original 1996**: The original building is one story and consists of reinforced concrete and Concrete Masonry Unit (CMU) walls with a metal roof diaphragm. The roof consists of metal deck with steel framing bearing on interior and exterior concrete and CMU walls. The foundation consists of slab-on-grade with concrete and CMU stem walls. The school includes classrooms, an administrative area, media center, music room, stage, gymnasium, and cafeteria. The approximate footprint of this building is 63,500 square feet.

3.1.3 Observed Deficiencies:

Relative to the other school structures within the Medford School District, Abraham Lincoln Elementary is of newer construction and possesses many details consistent with current practices. The reinforcing in concrete and CMU walls appears adequate for performance under seismic loadings. The relative seismic hazard for this facility is low due to the age of construction. The following list summarizes the structural seismic deficiencies observed during our visual inspections and/or original construction documents:

Building Label (RVS/ZCS)	Deficiency
1996 Campus Buildings [A]	• WALL ANCHORAGE: A discreet load path for wall out-of-plane forces at parallel roof framing conditions could not be determined.

3.1.4 Recommendations

The following are rehabilitation recommendations to address the observed deficiencies. In addition, for pursuit of Life Safety compliance we recommend the additional evaluation, field investigation, and non-structural assessment as noted in Section 2.

Building Label (RVS/ZCS)	Recommendations
1996 Campus Buildings [A]	• Provide refined analysis of structure to fully determine deficiencies. • Provide retrofit solutions as needed, including potential out-of-plane connections at the top of existing walls.

3.2 Griffin Creek Elementary School

Figure 2: Griffin Creek Elementary School

3.2.1 Seismic Evaluation Summary

The following table summarizes the building type, age, and construction for structures on this campus. It also includes a summary of the seismic assessment findings. Please see Appendix A for a campus map summarizing the campus construction and evaluation results.

Building Label (RVS/ZCS)	Building Type	Year Built, Remodel, and/or Retrofit	Structural Life Safety Assessment	Relative Hazard Severity
Classrooms and Library [A]	RM1	1966, 1970, 2018	Compliant	L.S. Compliant
Classrooms [B]	RM1/W2	1969, 2018	Compliant	L.S. Compliant
Cafeteria [C]	W2	1982	Noncompliant	Low
Gymnasium [D]	W2	1950	Noncompliant	Low
Classrooms [E]	C2/RM1/ W2	1953, 1955, 1982	Noncompliant	High
Classrooms [F]	RM1	1996	Noncompliant	Low
Classrooms [G]	RM1	2018	Compliant	L.S. Compliant

3.2.2 Structure Summary

The following summarizes the building timeline and structural systems for each portion of Griffin Creek Elementary School:

- **1900's Original**: The original building was constructed in the early 1900s and was removed for a new classroom addition in the 1970's
- **1950 Cafeteria Addition**: This addition was originally constructed as a gymnasium. It consists of wood framed walls with a flexible wood roof diaphragm. The roof is composed of plywood sheathing over wood purlins bearing on laminated wood arches. The foundation consists of slab-on-grade with concrete stem walls. The footprint of this building in approximately 4,500 square feet.
- **1953 Addition**: This classroom addition consists of concrete walls with a flexible wood roof diaphragm. The roof consists of plywood sheathing over rigid insulation bearing on glulam beams. The foundation consists of slab-on-grade with concrete stem walls. The footprint of this addition is approximately 2,900 square feet.
- **1955 Addition**: This classroom addition consists of concrete walls with a flexible wood roof diaphragm. The roof consists of plywood sheathing over wood framing. The foundation consists of slab-on-grade with concrete stem walls. The footprint of this addition is approximately 2,300 square feet.
- **1966 Addition**: This classroom addition consists of CMU walls with flexible wood roof diaphragms. The roof consists of plywood sheathing over trussed joists bearing on exterior walls. The foundation consists of slab-on-grade with concrete stem walls. The footprint of these additions is approximately 3,500 square feet.
- **1969 Addition**: This classroom addition consists of CMU walls with flexible wood roof diaphragms. The roof consists of plywood sheathing over trussed joists bearing on exterior walls. The foundation consists of slab-on-grade with concrete stem walls. The footprint of these additions is approximately 4,500 square feet.
- **1970 Main Building Addition**: This addition replaced the original classroom building and consists of exterior CMU walls and interior wood framed walls with a flexible wood roof diaphragm. The roof consists of plywood sheathing over trusses bearing on beams, interior wood framed walls, and perimeter CMU walls. The foundation consists of slab-on-grade with concrete stem walls. This building houses a library, classrooms, a workroom, and has a footprint of approximately 16,700 square feet.
- **1982 Gym Addition**: This addition consists of wood framed walls with a flexible roof and floor diaphragm. The roof consists of plywood sheathing over roof trusses bearing on beams and walls. The floor consists of wood joists bearing on beams and perimeter stem walls. The foundation consists of cast-in-place concrete stem walls and footings. The approximate footprint of this building is 8,000 square feet.

- **1982 Classroom Addition**: The classroom addition consists of wood framed walls with a flexible wood roof diaphragm. The roof consists of plywood sheathing over roof trusses bearing on exterior walls. The foundation consists of slab-on-grade with concrete stem walls. The footprint of this addition is approximately 2,800 square feet. A restroom was added to the south end of the 1961 addition; this addition consists of CMU walls with a flexible wood roof diaphragm. The roof consists of plywood sheathing over wood joists bearing on exterior CMU walls. The foundation consists of slab-on-grade with concrete stem walls. The footprint of this addition is approximately 600 square feet.
- **1996 Addition**: This classroom addition consists of CMU walls with a flexible wood roof diaphragm. The roof consists of plywood sheathing over wood trusses bearing on exterior CMU walls. The foundation consists of slab-on-grade with CMU stem walls. The footprint of this addition is approximately 6,000 square feet.
- **Seismic Study 2017**: Tier 1 Seismic evaluations were provided for Buildings C, D, and E. (Cafeteria, Gym, and Classroom respectively.) These reports describe the seismic deficiencies and recommended rehabilitation steps in greater detail and are on file with the District.
- **Seismic Retrofit 2018**: In the summer of 2018 the 1966 classroom addition, 1969 classroom addition, and 1970 main building classroom received a seismic upgrade bringing the structures up to Life Safety standards as defined in the ASCE 41. The areas brought up to Life Safety are illustrated in the Appendix A campus map. Letters stating compliance with Life Safety for the affected areas are on file with the District.
- **2018 Addition**: In the summer of 2018 a new classroom building was installed. This structure consists of reinforced masonry with a flexible wood roof diaphragm. The roof consists of plywood sheathing over manufactured timber trusses bearing on wood beams and exterior masonry walls. The foundation consists of slab-on-grade with cast-in-place concrete footings. The approximate footprint of this structure is 3,200 square feet. Given the age of construction, this structure is considered to meet the benchmark criteria and deemed to be complain with Life Safety provisions.
- **2019 SRG Application**: An SRG application was filed with the state in January 2019. The scope of that application covered Buildings C, D, E, and F. The application package includes additional information regarding deficiencies and recommended rehabilitations and is on file with the District. Funds for this application were not awarded for the January 2019 season.

3.2.3 Observed Deficiencies:

The following list summarizes the structural seismic deficiencies observed during our visual inspections and/or original construction documents:

Building Label (RVS/ZCS)	Deficiency
1951 Gym Addition [D]	• LOAD PATH: The laminated wood arches are not adequate for seismic forces.
1953 Addition [E]	• WALL ANCHORAGE: Out-of-plane connections at the top of wall are not present. • WALL ANCHORAGE: Ties at top of wall to glulam beams are present.
1955 Addition [E]	• WALL ANCHORAGE: Out-of-plane connections at the top of wall are not present. • WALL ANCHORAGE: Ties at top of wall to glulam beams are not present.
1982 Gymnasium [C]	• WALL ANCHORAGE: Out-of-plane connections at the top of wall are not detailed.
1996 Classrooms [F]	• SPAN: Plywood diaphragm span exceeds the allowable 40 feet.

3.2.4 Recommendations

The following are rehabilitation recommendations to address the observed deficiencies. In addition, for pursuit of Life Safety compliance we recommend the additional evaluation, field investigation, and non-structural assessment as noted in Section 2.

Building Label (RVS/ZCS)	Recommendations
1951 Gym Addition [D]	• Strengthen existing wood framed walls and provide adequate connections to transfer seismic forces from the roof diaphragm to the foundation.
1953 Addition [E]	• Provide new out-of-plane connections at the top of concrete walls. • Provide new out-of-plane connections at glulam beams to the top of concrete walls.
1955 Addition [E]	• Provide new out-of-plane connections at the top of concrete walls. • Provide new out-of-plane connections at glulam beams to the top of concrete walls.
1982 Gymnasium [C]	• Provide new out-of-plane connections at the top of existing walls.
1996 Classrooms [F]	• Provide refined analysis and/or diaphragm blocking.

3.3 Hoover Elementary School

Figure 3: Hoover Elementary School

3.3.1 Seismic Evaluation Summary

The following table summarizes the building type, age, and construction for structures on this campus. It also includes a summary of the seismic assessment findings. Please see Appendix A for a campus map summarizing the campus construction and evaluation results.

Building Label (RVS/ZCS)	Building Type	Year Built, Remodel, and/or Retrofit	Structural Life Safety Assessment	Relative Hazard Severity
Administration [A]	W1	1958, 1992, 2019	Compliant	L.S. Compliant
Classrooms [B]	RM1/W2	1958, 2019	Compliant	L.S. Compliant
Classrooms [C]	RM1/W2	1958, 2019	Compliant	L.S. Compliant
Classrooms [D]	RM1/W2	1958, 2019	Compliant	L.S. Compliant
Classrooms [E]	RM1/W2	1960, 2019	Compliant	L.S. Compliant
Classrooms [F]	RM1/W2	1982, 2019	Compliant	L.S. Compliant

Classrooms [G]	W1	1996, 2019	Compliant	L.S. Compliant
Classrooms [H]	W2	1975, 1992, 2019	Compliant	L.S. Compliant
Media Center [I]	RM1	1982, 1992, 2019	Compliant	L.S. Compliant
Gymnasium & Cafeteria[1] [J]	RM1	1958, 2019	Compliant	I.O. Compliant[1]
Classrooms [K]	W1/MOD	2015, 2019	Compliant	L.S. Compliant
Restrooms [L]	RM1	1996, 2019	Compliant	L.S. Compliant

1. *Structure was retrofit to meet ASCE 41 Life Safety provisions and structural Immediate Occupancy (IO) provisions. The retrofit was not expanded to satisfy non-structural IO requirements and therefore does not meet the full IO designation per ASCE 41.*

3.3.2 Structure Summary

The following summarizes the building timeline and structural systems for each portion of Hoover Elementary School:

- **1958 Original Buildings**: The original construction of Hoover Elementary consists of three classroom buildings, a separate office building, and one large building containing a cafeteria, gymnasium, stage, and boiler room. The original classrooms consist of CMU walls with flexible wood roof diaphragms. The classroom roofs consist of plywood sheathing on wood joists supported by glulam beams. The foundations consist of slab-on-grade with concrete stem walls. The classroom buildings have a combined footprint of approximately 14,700 square feet. The original gym consists of partial height CMU walls with wood framed walls above and a flexible wood roof diaphragm. The roof consists of plywood sheathing over wood joists bearing on wood trusses which bear on steel columns. The wood trusses were replaced with steel beams at an unknown later date. The main floor consists of wood joists bearing on beams and perimeter concrete stem walls and footings. The foundation consists of cast-in-place concrete stem walls and footings. The original cafeteria and boiler room building consists of full height CMU walls with flexible wood roof diaphragms. The roof consists of plywood sheathing over wood joists bearing on glulam beams. The cafeteria and gym floors consist of wood joists over glulam beams bearing on concrete stem walls and footings. At the boiler room the roof consists of plywood sheathing over 3x decking bearing on glulam beams. The boiler room foundation consists of cast-in-place slab-on-grade with concrete stem walls and footings. The original gym, cafeteria, and boiler room building have a footprint of approximately 12,800 square feet. The original office consists of wood framed walls with a flexible wood roof diaphragm. The roof system consists of plywood sheathing over wood joists bearing on the exterior walls. The foundation consists of slab-on-grade with concrete stem walls. The footprint of the office is approximately 1,280 square feet.

- **1960 Classroom Addition**: The classroom addition consists of CMU walls with a flexible wood roof diaphragm. The addition roof consists of plywood sheathing over wood joists bearing on glulam beams. The glulam beams bear on steel posts with concrete footings at one end and CMU walls at the other. The foundation consists of slab-on-grade with concrete stem walls. This addition has a footprint of approximately 4,200 square feet.
- **1975 Classroom Addition**: This addition is a pre-manufactured modular building consisting of wood framed walls with flexible wood diaphragms. The roof consists of plywood sheathing over wood joists bearing on wood beams. The floor consists of wood joists on wood beams and concrete stem walls. The foundation consists of cast-in-place concrete stem walls and footings. The building houses classrooms and has a footprint of approximately 4,900 square feet. An interior remodel of this building was completed in 1992.
- **1982 Additions**: These classroom additions consist of CMU walls with flexible wood roof diaphragms. The roof diaphragm consists of plywood sheathing bearing on wood trusses. The foundation consists of slab-on-grade with concrete stem walls. This addition has a footprint of approximately 5,200 square feet. The library addition consists of CMU walls with a flexible wood roof diaphragm. The roof consists of plywood sheathing over TJ joists bearing on CMU walls. The foundation consists of slab-on-grade with concrete stem walls. The library addition has a footprint of approximately 2,000 square feet.
- **1992 Addition and Remodel**: The work completed during this effort included a remodel of the 1975 classroom pod and an addition to the media center. The media center addition consists of CMU walls with a flexible wood roof diaphragm. The roof consists of plywood sheathing over wood truss joists bearing on exterior CMU walls. The foundation consists of slab-on-grade with concrete stem walls. The footprint of this addition is approximately 3,200 square feet.
- **1996 Additions**: A new classroom building was added to the northeast part of campus. A new restroom was added between the 1982 North addition and the 1960 North addition. The office received an addition of a classroom and new space for the principal's office. The classroom addition consists of wood framed walls with a flexible wood roof diaphragm. The classroom roof consists of plywood sheathing over trusses bearing on exterior walls. The foundation consists of slab-on-grade with concrete stem walls. The footprint of the classroom addition is approximately 2,100 square feet. The restroom addition consists of CMU walls with a flexible wood roof diaphragm. The roof consists of plywood sheathing over wood joists bearing on exterior CMU walls. The foundation consists of slab-on-grade with concrete stem walls. The footprint of the restroom is approximately 500 square feet. The office classroom addition consists of wood framed walls with a flexible wood roof diaphragm. The roof system consists of plywood sheathing over wood roof joists bearing on exterior walls. The foundation consists of slab-on-grade with concrete stem walls. The footprint of this addition is approximately 1,400 square feet.
- **2015 Addition**: A new classroom modular was added to the north side of the campus consisting of two classroom spaces totaling approximately 1,700 square feet. The modular consists of light timber roof framing, wood stud walls, and a concrete foundation.

- **2019 Seismic Retrofit**: All structures on this campus underwent a seismic retrofit during the summer and fall of 2019. Buildings B through E and J were funded by the state SRG program to bring them into Life Safety compliance, and the project was expanded to include all campus buildings. The structural systems of the Gymnasium and cafeteria were retrofitted to Immediate Occupancy performance standards as defined in the ASCE 41.

3.3.3 Observed Deficiencies:

All structures on this campus have undergone a seismic retrofit. See Section 3.3.2. Therefore, we did not observe any structural seismic deficiencies during our review of the campus.

3.3.4 Recommendations

This campus underwent a seismic retrofit designed to meet or exceed ASCE 41-13 provisions for Life Safety. Therefore, our evaluation is that the structures meet Life Safety requirements and do not require any additional steps by the District at this time.

3.4 Howard Elementary School

Figure 4: Howard Elementary School

3.4.1 Seismic Evaluation Summary

The following table summarizes the building type, age, and construction for structures on this campus. It also includes a summary of the seismic assessment findings. Please see Appendix A for a campus map summarizing the campus construction and evaluation results.

Building Label (RVS/ZCS)	Building Type	Year Built, Remodel, and/or Retrofit	Structural Life Safety Assessment	Relative Hazard Severity
Classrooms[2] [A]	W2	1980	Noncompliant	Moderate
Classrooms [B]	RM1	1972, 2018	Compliant	L.S. Compliant
Classrooms [C]	RM1	1970, 2018	Compliant	L.S. Compliant
Gymnasium[1] [D]	RM1	1970, 2018	Compliant	I.O. Compliant
Stage & Open Play Structure [E]	RM1	1980, 2018	Compliant	L.S. Compliant
Multi-Purpose [F]	RM1	1985, 2018	Compliant	L.S. Compliant

1*. Structure was retrofit to meet ASCE 41 Life Safety provisions and* *structural* *Immediate Occupancy (IO) provisions. The retrofit was not expanded to satisfy non-structural IO requirements and therefore does not meet the full IO designation per ASCE 41.*
2. An SRG grant application was submitted for all or part of this structure. See Structure Summary below for additional information.

3.4.2 Structure Summary

The following summarizes the building timeline and structural systems for each portion of Howard Elementary School:

- **1970's Original Buildings**: The original school classrooms and gymnasium consist of CMU walls with flexible wood roof diaphragms. The classroom roof consists of 2x decking over glulam beams. The original gymnasium is constructed with plywood sheathing over roof rafters bearing on glulam beams. The foundations consist of slab-on-grade with concrete stem walls. The structures house classrooms and a gymnasium, with a footprint of approximately 8,800 square feet.
- **1972 Addition**: Construction of the Northwest classroom building consists of CMU walls and wood framed walls with a flexible wood roof diaphragm. The roof system consists of 2x decking over glulam beams and plywood sheathing over wood trusses at the interior roof. The foundation consists of slab-on-grade with concrete stem walls. The structure houses classrooms and has a footprint of approximately 7,400 square feet.
- **1982 Additions**: The 1982 Main Classroom Building to the North consists of timber framed walls with brick veneer and a flexible wood roof diaphragm consisting of plywood sheathing on trusses over glulam beams. The foundation is slab-on-grade with concrete stem walls. This structure houses several classrooms, an administrative room, and has a footprint of approximately 34,500 square feet. A new stage building and cafeteria building were added to the south. The stage building consists of CMU walls with a flexible wood roof diaphragm. The roof system consists of plywood sheathing over truss joists. The foundation consists of slab-on-grade with concrete stem walls. This structure houses a stage and 2nd story classroom and has a footprint of approximately 7,400 square feet. The cafeteria building consists of a CMU West wall and wood framed walls with brick veneer and a flexible wood roof diaphragm. The roof system consists of plywood sheathing over truss joists. The foundation consists of slab-on-grade with concrete stem walls. This structure houses classrooms and has a footprint of approximately 4,800 square feet.
- **Seismic Retrofit 2018**: In the summer of 2018 the original 1970's classrooms, 1972 classroom addition, and the 1982 Stage/Classroom, Covered Play Area, and Cafeteria additions received a seismic upgrade bringing the structures up to Life Safety performance standards as defined in the ASCE 41. The structural systems of the Gymnasium were retrofitted to Immediate Occupancy performance standards as defined in the ASCE 41. Letters stating compliance with Life Safety for the affected areas are on file with the District.

- **SRG Applications**: An SRG application was filed with the state in November 2018 and resubmitted in January 2020. The scope of that application covered Building A. The application package includes additional information regarding deficiencies and recommended rehabilitations and is on file with the District. Funds for this application were not awarded for the November 2018 and January 2020 seasons.

3.4.3 Observed Deficiencies:

The following list summarizes the deficiencies observed during our visual inspections and/or original construction documents:

Building Label (RVS/ZCS)	Deficiency
1980 Main Building Addition [A]	• LOAD PATH: Exterior wood framed walls are not continuous to roof framing. • SHEAR CAPACITY: Interior shear walls are not present.

3.4.4 Recommendations:

The following are rehabilitation recommendations to address the observed deficiencies. In addition, for pursuit of Life Safety compliance we recommend the additional evaluation, field investigation, and non-structural assessment as noted in Section 2.

Building Label (RVS/ZCS)	Recommendations
1980 Main Building Addition [A]	• Provide new shear panels at top of exterior walls to roof structure above. • Add new interior wood framed shear walls at strategic locations.

3.5 Jackson Elementary School

Figure 5: Jackson Elementary School

3.5.1 Seismic Evaluation Summary

The following table summarizes the building type, age, and construction for structures on this campus. It also includes a summary of the seismic assessment findings. Please see Appendix A for a campus map summarizing the campus construction and evaluation results.

Building Label (RVS/ZCS)	Building Type	Year Built, Remodel, and/or Retrofit	Structural Life Safety Assessment	Relative Hazard Severity
Classroom & Administration [A]	S2a	2008	Compliant	L.S. Compliant
Multi-Purpose [B]	C2a	1949, 2008	Noncompliant	Low
Media Center [C]	RM1/W2	1995	Noncompliant	Low
Cafeteria [D]	W2	1995	Noncompliant	Low
Gymnasium [E]	W2	2008	Compliant	L.S. Compliant
Modular [F]	W1/MOD	2005	Noncompliant	Low
Modular [G]	W1/MOD	2015	Noncompliant	Low

3.5.2 Structure Summary

The following summarizes the building timeline and structural systems for each portion of the school still in use:

- **1995 Additions**: This project consisted of two additions. One addition was built to serve as a media center for the school. It was RM1/W2 building type, with portions of the building constructed from reinforced masonry and other portions utilizing wood framing. The roof consisted of timber trusses with plywood roof sheathing. The second addition was built to serve as the cafeteria. It was constructed from light timber framing including light timber roof trusses, plywood roof sheathing, and wood stud shear walls.
- **2008 Addition and Structural Improvements**: In 2008 a significant portion of the existing school was demolished for construction of a new classroom and administration building. The new building was construction type S2a. consisting primarily of open web roof joists supported by steel framing. The roof deck is comprised of concrete over metal deck, and the lateral system is comprised of Ordinary Concentric Braced Frames. The 2008 work also included seismic improvements for the Building B, including seismic in-plane and out-of-plane connections.
- **Modular units**: Two light timber modular units have been added to the campus in 2005 and 2015, respectively.
- **2021 Seismic Investigation**: ZCS performed additional investigations to confirm the extent of seismic retrofit installed within Building 'B' during the District Bond Campaign around 2008. These investigations were limited to accessible framing cavities and attics, no destructive investigations were performed as part of these investigations. In addition, geotechnical investigations were performed on this site to evaluate site specific hazards such as Liquefaction. The results of the geotechnical investigations are summarized in Appendix A and Appendix D.

  ZCS confirmed install of out-of-plane connection hardware between the existing second floor framing and roof framing to existing concrete walls noted in the Jackson Elementary School construction drawings dated August 20, 2008. Seismic isolation between existing Building 'B' and the 2008 building addition was confirmed. Confirmation of new in-plane connection hardware was confirmed at existing steel angles supporting existing floor and roof framing. It should be noted upon further review of the existing steel angle construction, an adequate connection for transfer of seismic forces between diaphragms and existing steel angle could not be identified. Install of plywood sheathing at second floor and roof diaphragms of Building 'B' was confirmed as noted in the 2008 construction drawings. During the site investigations it was confirmed that existing roof and floor diaphragms are unblocked and exceed span limitations per ASCE 41 Teir 1 checklists. This additional deficiency is reflected in section 3.5.3.

These findings provide confirmation that seismic hazard mitigation at Building 'B' has taken place. It should be noted a basis of design for the seismic retrofit scope was not confirmed with the Architect/Engineer of Record, all retrofit scope items could not be verified, and additional deficiencies were identified. The seismic retrofit scope of this structure appears to have been focused on a seismic hazard reduction and not an all-encompassing seismic retrofit. Therefore, the structural life safety designation of noncompliant and relative seismic hazard severity designation of low remains unchanged for this building. The installed seismic retrofit items significantly improve the performance of this building during a seismic event but do not meet life safety standards per ASCE 41-17.

3.5.3 Observed Deficiencies

The following list summarizes the deficiencies observed during our visual inspections and/or original construction documents:

Building Label (RVS/ZCS)	Deficiency
1949/2008 Multi-Purpose [B]	• INFORMATION: Insufficient information to confirm level of retrofit design. • LOAD PATH: In-plane connection hardware not confirmed. • SPANS: Unblocked plywood roof and floor diaphragms exceed span limits.
1995 Media Center [C]	• SPAN: diaphragm spans farther than allowable 40 feet. • ANCHORAGE: Adequacy of in-plane and out-of-plane anchorage should be verified.
1995 Cafeteria [D]	• SPAN: diaphragm spans farther than allowable 40 feet. • INFORMATION: Verification of construction required.
2005/2015 Modular Units [F, G]	• ANCHORAGE: Insufficient information to confirm foundation anchorage.

3.5.4 Recommendations:

The following are rehabilitation recommendations to address the observed deficiencies. In addition, for pursuit of Life Safety compliance we recommend the additional evaluation, field investigation, and non-structural assessment as noted in Section 2.

Building Label (RVS/ZCS)	Recommendations
1949/2008 Multi-Purpose [B]	• INFORMATION: Obtain additional/supporting design information for the completed retrofit.

	• LOAD PATH: Provide new in-plane connection hardware between existing concrete walls and diaphragms. • SPANS: Provide blocking at floor and roof diaphragms or provide new plywood sheathing at interior wood framed walls to reduce diaphragm spans.
1995 Media Center [C]	• Provide refined analysis of building (Tier 2). • Pursue diaphragm strengthening or installation of additional points of lateral support.
1995 Cafeteria [D]	• Provide field verification of construction and non-structural elements. Provide refined analysis of building (Tier 2) if required.
2005/2015 Modular Units [F, G]	• Provide field verification of foundation anchorage.

3.6 Jacksonville Elementary School

Figure 6: Jacksonville Elementary School

3.6.1 Seismic Evaluation Summary

The following table summarizes the building type, age, and construction for structures on this campus. It also includes a summary of the seismic assessment findings. Please see Appendix A for a campus map summarizing the campus construction and evaluation results.

Building Label (RVS/ZCS)	Building Type	Year Built, Remodel, and/or Retrofit	Structural Life Safety Assessment	Relative Hazard Severity
Media Center, Classrooms & Administration[1] [A]	RM1	1982	Noncompliant	High
Gymnasium[1] [B]	RM1	1982	Noncompliant	High
Classrooms [C]	RM1	1990	Noncompliant	Moderate
Classrooms & Cafeteria [D][1]	RM1	1954, 1982	Noncompliant	High

1. An SRG grant application was submitted for all or part of this structure. See Structure Summary below for additional information.

3.6.2 Structure Summary

The following summarizes the building timeline and structural systems for each portion of Jacksonville Elementary School:

- **1954 Original**: This original school consists of full height CMU walls with a flexible wood roof diaphragm. The roof of the classroom wings consists of 3x decking over glulam beams bearing on exterior and interior CMU walls. The roof of the Multi-Purpose Room consists of 2x decking over steel joists bearing on exterior CMU walls. The roof between the classroom wing and the multi-purpose building consists of plywood sheathing over wood joists bearing on interior and exterior walls. The foundation consists of slab-on-grade with concrete stem walls. This building houses classrooms, a Multi-Purpose room and is approximately 13,000 square feet.
- **1982 Roof Remodel**: The roof system of the original buildings was remodeled during the 1982 addition. New wood framing and joists were added above the existing CMU walls and roof system of the original building, presumably to improve drainage and better match the addition roof lines. The roof consists of plywood sheathing over wood joists.
- **1982 Addition**: This addition consists of full height CMU walls with a flexible wood roof diaphragm. The roof consists of plywood sheathing over TJ joists and wood joists bearing on exterior and interior CMU walls. At the new gym the roof consists of steel deck over steel joists bearing on exterior CMU walls. The foundation consists of slab-on-grade with concrete stem walls. The approximate footprint of this addition is 31,500 square feet.
- **1990 Media Center Addition**: This addition consists of CMU walls with a flexible wood roof diaphragm. The roof consists of plywood sheathing over TJ joists bearing on interior beams and exterior CMU walls. The foundation consists of slab-on-grade with concrete stem walls. This addition has an approximate footprint of 8,600 square feet.
- **SRG Applications**: Two SRG applications were filed with the state in November 2018 and resubmitted in January 2020. The scope of those applications covered Building D, C, B, and A. The application packages include additional information regarding deficiencies and recommended rehabilitations and are on file with the District. Funds for this application were not awarded for the November 2018 or January 2020 season.

  One SRG application was filed with the state in December 2020. The scope of this application covered Buildings A, B, C, and most of D (excluding the cafeteria portion). The application package includes additional information regarding deficiencies and recommended rehabilitations and are on file with the District. Funds for this application were not awarded for the December 2020 season. One SRG application was filed with the state in February 2022. The scope of this application covers all buildings on the Jacksonville Elementary campus utilizing match funds from the District to include all structures. Grant award announcements are expected in the Spring of 2022. In addition, geotechnical investigations were performed on this site to evaluate site specific hazards such as Liquefaction. The results of the geotechnical investigations are summarized in Appendix A and Appendix D.

3.6.3 Observed Deficiencies:

The following list summarizes the structural seismic deficiencies observed during our visual inspections and/or original construction documents:

Building Label (RVS/ZCS)	Deficiency
1954 Original [D]	• WALL ANCHORAGE: Out-of-plane connections at the top of wall are not present. • STRAIGHT SHEATHING: A straight sheathed roof diaphragm is present and does not meet span limitations.
1982 Addition [A, B, D]	• LOAD PATH: The 1982 roof framing over the original 1954 school is not adequately connected to masonry walls. • WALL ANCHORAGE: Out-of-plane connections at top of wall to 1982 roof diaphragm are not present. • WALL ANCHORAGE: In-plane shear connections at top of wall to 1982 roof framing are not present.
1990 Addition [C]	• SPAN: The plywood diaphragm exceeds the allowable span. • LEDGER: There are wood ledgers detailed to experience cross-grain bending in a seismic event.

3.6.4 Recommendations

The following are rehabilitation recommendations to address the observed deficiencies. In addition, for pursuit of Life Safety compliance we recommend the additional evaluation, field investigation, and non-structural assessment as noted in Section 2.

Building Label (RVS/ZCS)	Recommendations
1954 Original [D]	• Provide new out-of-plane connections at the top of CMU walls. • Provide new sheathing over 3x decking of original classroom roof system. • Provide new sheathing over 2x decking at cafeteria roof system.
1982 Addition [A, B, D]	• Provide proper attachment of upper roof system to original roof and CMU walls.
1990 Addition [C]	• Provide refined analysis, add blocking to the diaphragm, or add additional shearwall lines. • Provide out-of-plane connection at ledger framing locations.

3.7 Jefferson Elementary School

Figure 7: Jefferson Elementary School

3.7.1 Seismic Evaluation Summary

The following table summarizes the building type, age, and construction for structures on this campus. It also includes a summary of the seismic assessment findings. Please see Appendix A for a campus map summarizing the campus construction and evaluation results.

Building Label (RVS/ZCS)	Building Type	Year Built, Remodel, and/or Retrofit	Structural Life Safety Assessment	Relative Hazard Severity
Classrooms, Gymnasium & Cafeteria [A]	C2a	1954, 2007	Noncompliant	Low
Classrooms [B]	RM1	1996	Noncompliant	Low
Library & Media Center [C]	RM1	1971, 1996	Noncompliant	Low
‘Classrooms [D]	RM1	1971, 2007	Noncompliant	Low
Classrooms [E]	W1	1977, 2007	Noncompliant	Low
Day Care Center [F]	W1/MOD	1994	Noncompliant	Low

3.7.2 Structure Summary

The following summarizes the building timeline and structural systems for each portion of Jefferson Elementary School:

- **1954 Original**: The original building consists of concrete walls with a brick veneer and a flexible wood roof diaphragm. The roof consists of plywood sheathing over trussed roof joists in the North classrooms bearing on exterior and interior walls. The roof of the southern classroom consists of plywood sheathing over wood joists bearing on exterior and interior walls. The roof of the gymnasium consists of plywood sheathing over wood joists on wood trusses bearing on concrete pilasters. The roof in the Cafeteria consists of plywood sheathing over wood joists on glulam beams. The main floor framing consists of wood joists on beams; the foundation consists of cast-in-place concrete stem walls and footings. Some areas are slab-on-grade with concrete stem walls. This structure houses classrooms and a gym with an approximate footprint of 34,000 square feet.
- **1971 Addition**: This addition includes a library and classrooms which consist of CMU walls with a flexible wood roof diaphragm. The roof consists of plywood sheathing over trussed joists bearing on exterior walls and an interior beam. The floor of the library consists of a flexible wood diaphragm bearing on concrete stem walls and post and beams. The foundation consists of cast-in-place concrete footings. The classroom foundation consists of slab-on-grade with concrete stem walls. The approximate footprint of these additions is 6,800 square feet.
- **1977 Addition**: This addition consists of wood framed walls with a flexible wood roof diaphragm. The roof consists of wood joists bearing on exterior walls. The main floor framing consists of wood joists bearing on CMU stem walls and beams; the foundation consists of cast-in-place concrete footings. This addition houses a classroom with a footprint of approximately 2,500 square feet.
- **1994 Addition**: A new classroom modular building was added to the campus to serve as a day care center. This consists of a light wood framed structure.
- **1996 Addition**: A new classroom and Media Center were added in 1996. The Media Center consists of wood framed walls framed against existing concrete walls with a flexible wood roof diaphragm. The classroom addition consists of new concrete walls and wood framed walls at the existing structure with a flexible wood roof diaphragm. The foundations consist of slab-on-grade with concrete stem walls. The footprint of these additions is approximately 4,500 square feet.
- **2007 Bond Improvements**: In 2007 Jefferson Elementary received upgrades to the existing structure in addition to interior renovations. For structural upgrades, Out-of-plane connections were added to Building A. The roof of Building D and adjacent canopies were completely reframed. The structural upgrades for Building D included new out-of-plane connections at the top of CMU walls to roof framing, new blocking at concrete walls to roof framing, new structural roof sheathing, new interior shear walls, and new footings to transfer shear forces. Seemingly non-seismic structural work was completed on Building E (additional wall openings installed).
- **2021 Seismic Investigation**: Geotechnical investigations were performed on this site to evaluate site specific hazards such as Liquefaction. The results of the geotechnical investigations are summarized in Appendix A and Appendix D.

3.7.3 Observed Deficiencies:

The following list summarizes the structural seismic deficiencies observed during our visual inspections and/or original construction documents:

Building Label (RVS/ZCS)	Deficiency
1954 Original [A]	• SPAN: Unblocked wood diaphragm spans farther than 40 feet. • INFORMATION: Insufficient information to confirm level of retrofit design.
1996 Classrooms [B]	• SPAN: Unblocked wood diaphragm spans farther than 40 feet.
1971,1977 Classrooms & Library [C, E]	• SPAN: Unblocked wood diaphragm spans farther than 40 feet. • LEDGER: There are wood ledgers detailed to experience cross-grain bending in a seismic event.
1971, 2007 Classrooms [D]	• INFORMATION: Insufficient information to confirm level of retrofit design.
1994 Modular [F]	• ANCHORAGE: Insufficient information to confirm foundation anchorage

3.7.4 Recommendations

The following are rehabilitation recommendations to address the observed deficiencies. In addition, for pursuit of Life Safety compliance we recommend the additional evaluation, field investigation, and non-structural assessment as noted in Section 2.

Building Label (RVS/ZCS)	Recommendations
1954 Original [A]	• Provide refined analysis and field verification to determine adequacy of existing systems and partial 2007 seismic retrofit. (Opportunity during any reroof project). Obtain additional/supporting design information for the completed retrofit. • Provide new plywood sheathing at roof systems found lacking plywood.

1996 Classrooms [B]	• Provide refined analysis, add blocking to the diaphragm, or add additional shearwall lines.
1971,1977 Classrooms & Library [C, E]	• SPAN: Unblocked wood diaphragm spans farther than 40 feet. • LEDGER: There are wood ledgers detailed to experience cross-grain bending in a seismic event.
1971, 2007 Classrooms [D]	• Obtain additional/supporting design information for the completed retrofit.
1994 Modular [F]	• Provide field verification of foundation anchorage.

3.8 Kennedy Elementary School

Figure 8: Kennedy Elementary School

3.8.1 Seismic Evaluation Summary

The following table summarizes the building type, age, and construction for structures on this campus. It also includes a summary of the seismic assessment findings. Please see Appendix A for a campus map summarizing the campus construction and evaluation results.

Building Label (RVS/ZCS)	Building Type	Year Built, Remodel, and/or Retrofit	Structural Life Safety Assessment	Relative Hazard Severity
Administration and Classrooms [A]	W2	1977, 1979	Noncompliant	Low
Multi-Purpose[1] [B]	RM1	1981, 1992, 1994	Noncompliant	High
Classrooms [C]	W2	Est. 1978, 1981	Noncompliant	Low
Classrooms [D]	W2	Est. 1978, 1981, 1994	Noncompliant	Low
Classrooms [E]	W1	1996	Noncompliant	Low
Classrooms [F]	W2	Est. 1978	Noncompliant	Low
Classrooms [G]	W2	1979	Noncompliant	Low

Cafeteria [H]	W2	1979, 1981, 1992	Noncompliant	Low
Classrooms [I]	W2	1979	Noncompliant	Low
Classrooms [J]	W2	1977	Noncompliant	Low
Classrooms [K]	W2	1977	Noncompliant	Low

1. An SRG grant application was submitted for all or part of this structure. See Structure Summary below for additional information.

3.8.2 Structure Summary

The following summarizes the building timeline and structural systems for each portion of Jefferson Elementary School:

- **1977 Original Classrooms**: The original buildings consist of wood framed walls with flexible wood roof diaphragms. The roofs consist of plywood sheathing over wood joists bearing on exterior walls and interior beams. The floor consists of wood joists bearing on interior beams and partially grouted CMU perimeter stem walls. The foundation consists of cast-in-place concrete footings. The approximate footprint of these buildings is 9,800 square feet.
- **1977 Classroom Additions**: The three 1977 classroom additions at Kennedy consist of wood framed walls with flexible wood roof diaphragms. The roofs consist of plywood sheathing over wood joists bearing on exterior walls and interior beams. The floor consists of wood joists bearing on interior beams and partially grouted CMU perimeter stem walls. The foundation consists of cast-in-place concrete footings. The approximate footprint of these buildings is 8,200 square feet.
- **1979 Classroom Additions**: The four 1979 additions at Kennedy consist of wood framed walls with flexible wood roof diaphragms. The roofs consist of plywood sheathing over wood joists bearing on exterior walls and interior beams. The floor consists of wood joists bearing on interior beams and partially grouted CMU perimeter stem walls. The foundation consists of cast-in-place concrete footings. The approximate footprint of these buildings is 12,900 square feet.
- **1981 Multi-Purpose (MP) Addition**: This addition consists of partial height CMU walls with wood framed walls above, a flexible wood mezzanine and roof diaphragm; some areas are full height CMU walls. The areas of partial height CMU walls have steel columns at pilaster locations anchored to the top of the CMU walls. The roof consists of plywood sheathing over wood joists bearing on glulam beams and exterior walls. The floor consists of wood framed joists bearing on wood beams and concrete perimeter stem walls. The foundation consists of cast-in-place concrete footings. This building houses a gymnasium, stage, music room, and dressing rooms with an approximate footprint of 10,500 square feet.

- **1981 Classroom Addition**: The three 1981 classroom additions at Kennedy consist of wood framed walls with flexible wood roof diaphragms. The roof consists of plywood sheathing over wood joists bearing on exterior walls and interior beams. The floor consists of wood joists bearing on interior beams and partially grouted CMU stem walls. The foundation consists of cast-in-place concrete footings. The approximate footprint of these buildings is 18,200 square feet.
- **1992 Addition**: A small storage room was added to the Multi-Purpose building in 1992; this addition consists of wood framed walls with a flexible wood roof diaphragm. The roof consists of plywood sheathing over wood joists bearing on exterior walls. The foundation consists of cast-in-place slab-on-grade with concrete stem walls. This addition has an approximate footprint of 260 square feet.
- **1996 Addition**: A new addition was added adjacent to Building F, adding approximately 2,100 square feet. This consists of wood light framed timber construction including light timber trusses, wood stud walls, and reinforced concrete foundations.
- **2007 Bond Improvements**: A New HVAC system and flooring were installed in 2007, new site security fencing was added in 2009, and security upgrades were added to the office entry in 2013.
- **SRG Applications**: An SRG application was filed with the state in November 2018. The scope of that application covered Building B. The application package includes additional information regarding deficiencies and recommended rehabilitations and is on file with the District. Funds for this application were not awarded for the November 2018 season.

  An expanded SRG application was filed with the state in January 2020. The scope of that application covered Building A, B, C, D, E, F, and K. the application package includes additional information regarding deficiencies and recommended rehabilitations and is on file with the District. Funds for this application were not awarded for the January 2020 season. In addition, geotechnical investigations were performed on this site to evaluate site specific hazards such as Liquefaction. The results of the geotechnical investigations are summarized in Appendix A and Appendix D.

3.8.3 Observed Deficiencies:

Based on our observations and available construction documents the timber framed structures at Kennedy Elementary School have a low seismic hazard level.

The following list summarizes the structural seismic deficiencies observed during our visual inspections and/or original construction documents:

Building Label (RVS/ZCS)	Deficiency
1981 Multi-Purpose [B]	• SPAN: Unblocked wood diaphragm spans farther than 40 feet. • WALL ANCHORAGE: Out-of-plane connections at the top of wall are not present.
Various Classroom Additions and Admin [A, B, C, D, E, F, G, H, I, J, K, H]	• SPAN: Unblocked wood diaphragm spans farther than 40 feet.

3.8.4 Recommendations

The following are rehabilitation recommendations to address the observed deficiencies. In addition, for pursuit of Life Safety compliance we recommend the additional evaluation, field investigation, and non-structural assessment as noted in Section 2.

Building Label (RVS/ZCS)	Recommendations
1981 Multi-Purpose [B]	• Provide new out-of-plane connections at the top of CMU walls. • Provide new blocking at underside of roof sheathing and re-nail roof sheathing.
Various Classroom Additions and Admin [A, B, C, D, E, F, G, H, I, J, K, H]	• Provide refined analysis, add blocking to the diaphragm, or add additional shearwall lines.

3.9 Lone Pine Elementary School

Figure 9: Lone Pine Elementary School

3.9.1 Seismic Evaluation Summary

The following table summarizes the building type, age, and construction for structures on this campus. It also includes a summary of the seismic assessment findings.

Building Label (RVS/ZCS)	Building Type	Year Built, Remodel, and/or Retrofit	Structural Life Safety Assessment	Relative Hazard Severity
Administration & Classrooms [A]	S2a	2008	Compliant	L.S. Compliant
Cafeteria & Gymnasium [B]	S2a	2008	Compliant	L.S. Compliant
Classrooms [C]	RM1	1956, Est., 2008	Compliant	L.S. Compliant
Classrooms (Southern Portion) [C]	RM1	1995, 2008	Noncompliant	Low
Classrooms [D]	RM1	1963, 1965, 1966, 2008	Compliant	L.S. Compliant
Library [E]	W2	1982, 2008	Compliant	L.S. Compliant
Classrooms [F]	W1/MOD	2015	Noncompliant	Low

Classrooms [G]	W1/MOD	2015	Noncompliant	Low

3.9.2 Structure Summary

The following summarizes the building timeline and structural systems for each portion of the school:

- **Original Campus**: The original classroom and gymnasium buildings were demolished as part of the bond campaign new construction noted below. Record drawings for the original campus were not available at time of writing, with the original date of construction unable to be verified.
- **1956 Addition**: This classroom wing was constructed in 1956 and consists of masonry bearing walls, timber roof beams with light timber purlins, and continuous concrete footings.
- **1963, 1965, 1966 Additions**: This series of classroom additions were completed for Building D. They consist of masonry bearing walls, timber roof beams with light timber purlins, and continuous concrete footings.
- **1982 Addition**: This structure serves as a library. It was constructed of CMU retaining walls and light timber framing.
- **1995 Classroom Addition**: No record drawings for this building were available at time of writing. It was constructed as an addition to Building C and consists of masonry bearing walls that support light timber roof framing. From the available information, it does not appear additional strengthening of the lateral system was provided during the 2008 bond work below.
- **2008 Bond Improvements**: The campus was largely demolished and rebuilt as part of the 2008 bond work. This involved constructing new classroom, administration, and gym/cafeteria buildings. The project also involved the remodel and structural strengthening (including lateral systems) of Building E and the northern portion of Building C. The new buildings were constructed largely of steel framing, including steel open web joists, steel pan roofing deck, and heavy steel framing. The lateral system consists largely of special steel concentric braced frames.
- **2015 Modular Units**: Two wooden modular units were added to the campus in 2015.
- **2021 Seismic Investigation**: ZCS performed additional investigations to confirm the extent of seismic retrofit installed within Buildings ‘C’, ‘D’, and ‘E’ during the District Bond Campaign around 2008. These investigations were limited to accessible framing cavities and attics, no destructive investigations were performed as part of these investigations. In addition geotechnical investigations were performed on this site to evaluate site specific hazards such as Liquefaction. The results of the geotechnical investigations are summarized in Appendix A and Appendix D.

  ZCS confirmed install of steel frames, and top of shear wall connection hardware of Building ‘C’ and ‘D’ noted in the Lone Pine Elementary School construction drawings dated May 30, 2008. Confirmation of new out-of-plane connection hardware could not be confirmed without removal of existing ceiling and wall finishes. Shear wall install at Building ‘E’ noted in the 2008 construction documents was confirmed. ZCS observed install of steel straps at the northeast basement CMU wall of Building ‘E’.

This installation differs from the approved construction documents and likely a scope substitution during time of construction. Additional documentation for this item is not available. The existing CMU wall was observed to have signs of distress and water intrusion, it is our understanding the District is evaluating and addressing this issue. It should be noted the south portion of Building 'C' was not included in the seismic retrofit scope during the bond.

These findings provide confirmation that seismic hazard mitigation at Buildings 'C', 'D', and 'E' has taken place. A basis of design for the seismic retrofit scope was confirmed with Coughlin-Porter-Lundeen the Engineer of Record. The retrofit scope was evaluated in accordance with ASCE 31-04 and designed to the 2006 IBC utilizing a Life Safety performance objective. With the information provided by Coughlin-Porter-Lundeen and the on-site verifications the structural life safety designation of buildings 'C', (excluding the southern wing), 'D', and 'E' have been updated to compliant. Repairs to the existing northeast retaining wall of Building 'E' are required, it is our understanding the District is currently addressing the issue.

3.9.3 Observed Deficiencies:

The following list summarizes the structural seismic deficiencies observed during our visual inspections and/or review of original construction documents:

Building Label (RVS/ZCS)	Deficiency
1995 Classroom [C, South Portion]	• SPAN: Unblocked wood diaphragm spans further than 40 feet.
1982, 2008 Library [E]	• STRUCTURAL: Existing northeast CMU retaining wall shows signs of distress.
2015 Modular Units [G, F]	• ANCHORAGE: Insufficient information to confirm foundation anchorage.

3.9.4 Recommendations

The following are rehabilitation recommendations to address the observed deficiencies. In addition, for pursuit of Life Safety compliance we recommend the additional evaluation, field investigation, and non-structural assessment as noted in Section 2.

Building Label (RVS/ZCS)	Recommendations
1995 Classroom [C, South Portion]	• Provide refined analysis and corresponding destructive investigation. • Pending verification, provide diaphragm blocking or additional interior shearwall lines.
1982, 2008 Library [E]	• Repair existing northeast CMU retaining wall.
2015 Modular Units [G, F]	• Provide field verification of foundation anchorage.

3.10 Oak Grove Elementary School

Figure 10: Oak Grove Elementary School

3.10.1 Seismic Evaluation Summary

The following table summarizes the building type, age, and construction for structures on this campus. It also includes a summary of the seismic assessment findings.

Building Label (RVS/ZCS)	Building Type	Year Built, Remodel, and/or Retrofit	Structural Life Safety Assessment	Relative Hazard Severity
Offices [A]	C1/URM	1891	Noncompliant	Low
Classrooms [B1]	RM1	1948, 1954, 1960	Noncompliant	Low
Classrooms [B2]	RM1	1948, 1954, 1960	Noncompliant	Moderate
Classrooms [C]	RM1	1983, 2008	Noncompliant	Moderate
Administration & Gymnasium [D]	S2a	2008	Compliant	L.S. Compliant
Classrooms [E]	RM1	1996	Noncompliant	Moderate

Classrooms [F]	RM1	1967	Noncompliant	Moderate
Cafeteria [G]	W2	1967	Noncompliant	Moderate

3.10.2 Structure Summary

The following summarizes the building timeline and structural systems for each portion of the campus. At time of writing, no record drawings were available for the portions of the campus outside of the 2008 improvements. Therefore, the below discussion is based on discussions with staff and other available campus documentation.

- **Original Classroom 1891**: This portion was constructed of concrete and unreinforced masonry with timber roof framing.
- **Classroom Addition 1948**: Record drawings for this addition are not available at time of writing, and the date of construction is estimated. The buildings were constructed of masonry (reinforcement unknown) with heavy and light timber roof framing. Existence of plywood roof sheathing, masonry reinforcement, and nature of construction detailing are unknown.
- **Classroom and Cafeteria Addition 1967**: Record drawings for these additions are not available at time of writing, and the date of construction is estimated. These buildings were constructed of masonry (reinforcement unknown) with timber roof framing. Existence of plywood roof sheathing, masonry reinforcement, and nature of construction detailing are unknown.
- **Classroom Addition 1983**: Record drawings for this addition are not available at time of writing, and the date of construction is estimated. This building was constructed of reinforced masonry (assumed based upon estimated construction period) with timber roof framing. It is unknown whether plywood roof sheathing is present. Reinforcement and construction details are unknown.
- **Classroom Addition 1996**: Record drawings for this addition are not available at time of writing, and the date of construction is estimated. This classroom building was constructed of reinforced masonry with timber roof framing. Reinforcement and construction details are unknown.
- **Gymnasium/Multi-Purpose Building 2008**: This structure was built in 2008. The primary structure consists of steel framing with metal roof decking. The lateral system consists of Ordinary Concentric Braced Frames.
- **2008 Bond Improvements**. Various MEP and structural improvements were made to the campus during the 2008 bond campaign. The MEP improvements consisted primarily of new HVAC units for several of the buildings. The structural improvements were limited to Buildings A, B1, and campus canopies. At Building A, these improvements consisted of new out-of-plane ties at the floor and roof levels, new shotcrete shearwalls, and new diaphragm sheathing. At Building B1, these improvements consisted of roof-to-wall connections and improvement of lateral load path elements such as the addition of sheathing to attic cripple walls.
- **2021 Seismic Investigation**: Geotechnical investigations were performed on this site to evaluate site specific hazards such as Liquefaction. The results of the geotechnical investigations are summarized in Appendix A and Appendix D.

3.10.3 Observed Deficiencies:

The following list summarizes the structural seismic deficiencies observed during our visual inspections and/or review of original construction documents:

Building Label (RVS/ZCS)	Deficiency
Various Classrooms [A, B1]	• INFORMATION: Insufficient information to confirm level of retrofit design.
Various Classroom & Cafeteria Buildings [B2, C, E, F, G]	• SPAN: Unblocked wood diaphragm spans farther than 40 feet. • INFORMATION: Incomplete information to verify load paths and anchorage.

3.10.4 Recommendations

The following are rehabilitation recommendations to address the observed deficiencies. In addition, for pursuit of Life Safety compliance we recommend the additional evaluation, field investigation, and non-structural assessment as noted in Section 2.

Building Label (RVS/ZCS)	Recommendations
Various Classrooms [A, B1]	• Obtain additional/supporting design information for the completed retrofit.
Various Classroom & Cafeteria Buildings [B2, C, E, F, G]	• Provide refined analysis and corresponding destructive investigation. • Provide in-plane of out-of-plane connections where determined to be deficient. • Provide additional lateral force resisting systems where determined to be deficient.

3.11 Roosevelt Elementary School

Figure 11: Roosevelt Elementary School

3.11.1 Seismic Evaluation Summary

The following table summarizes the building type, age, and construction for structures on this campus. It also includes a summary of the seismic assessment findings.

Building Label (RVS/ZCS)	Building Type	Year Built, Remodel, and/or Retrofit	Structural Life Safety Assessment	Relative Hazard Severity
Administration & Classrooms [A]	S2a	2008	Compliant	L.S. Compliant
Classrooms [B]	C2a	1949, 2008	Noncompliant	Low
Gymnasium [C]	W2	2008	Compliant	L.S. Compliant
Cafeteria [D]	W2	1995	Noncompliant	Low

3.11.2 Structure Summary

The following summarizes the building timeline and structural systems for each portion of Jacksonville Elementary School:

- **Classroom 1949**: Original plans for this structure were not available. It is constructed of masonry bearing walls (reinforcement and detailing unknown) with flexible wood framed diaphragms.
- **Cafeteria Addition 1995**: This structure is constructed of light timber framing, included light timber trusses, plywood roof diaphragm, and plywood sheathed shearwalls.
- **2008 Bond Additions and Retrofit**: Two structures were added in 2008. One structure serves as classrooms and multi-purpose space, and the second serves as a gymnasium. The classroom addition was constructed steel framing with a light timber framed roof system. The main lateral system consists of ordinary concentric braced frames. The gym structure is constructed of light timber framing including timber chord roof bar joists, stud walls, and plywood sheathed shearwalls. The 1949 addition was also retrofit as part of the bond campaign including the additional of out-of-plane ties, plywood diaphragms, and seismic isolation joints.
- **2021 Seismic Investigation**: ZCS performed additional investigations to confirm the extent of seismic retrofit installed within Building 'B' during the District Bond Campaign around 2008. These investigations were limited to accessible framing cavities and attics, no destructive investigations were performed as part of these investigations. In addition, geotechnical investigations were performed on this site to evaluate site specific hazards such as Liquefaction. The results of the geotechnical investigations are summarized in Appendix A and Appendix D.

  ZCS confirmed install of out-of-plane connection hardware between the existing second floor framing and roof framing to existing concrete walls noted in the Roosevelt Elementary School construction drawings dated August 20, 2008. Seismic isolation between the existing Building 'B' wing and 2008 Building was confirmed. New in-plane connection hardware was confirmed at existing steel angles supporting existing floor and roof framing. It should be noted upon further review of the existing steel angle construction an adequate connection for transfer of seismic forces between diaphragms and existing steel angle could not be confirmed. Install of plywood sheathing at second floor and roof diaphragms of Building 'B' was confirmed as noted in the 2008 construction drawings. During the site investigations it was confirmed that existing roof and floor diaphragms are unblocked and exceed span limitations per ASCE 41 Teir 1 checklists. This additional deficiency is reflected in section 3.11.3.

These findings provide confirmation that seismic hazard mitigation at Building ‘B’ has taken place. It should be noted a basis of design for the seismic retrofit scope was not confirmed with the Architect/Engineer of Record, all retrofit scope items could not be verified, and additional deficiencies were identified. The seismic retrofit scope of this structure appears to have been focused on a seismic hazard reduction and not an all-encompassing seismic retrofit. Therefore, the structural life safety designation of noncompliant and relative seismic hazard severity designation of low remains unchanged for this building. The installed seismic retrofit items significantly improve the performance of this building during a seismic event but do not meet life safety standards per ASCE 41-17.

3.11.3 Observed Deficiencies:

The following list summarizes the structural seismic deficiencies observed during our visual inspections and/or review of original construction documents:

Building Label (RVS/ZCS)	Deficiency
1949, 2008 Classrooms [B]	• INFORMATION: Insufficient information to confirm level of retrofit design. • LOAD PATH: In-plane connection hardware not confirmed. • SPANS: Unblocked plywood roof and floor diaphragms exceed span limits.
Cafeteria [D]	• SPAN: Unblocked wood diaphragm spans farther than 40 feet.

3.11.4 Recommendations

The following are rehabilitation recommendations to address the observed deficiencies. In addition, for pursuit of Life Safety compliance we recommend the additional evaluation, field investigation, and non-structural assessment as noted in Section 2.

Building Label (RVS/ZCS)	Recommendations
1949, 2008 Classrooms [B]	• Obtain additional/supporting information for the completed retrofit. • LOAD PATH: Provide new in-plane connection hardware between diaphragms and existing steel angles. • SPANS: Provide blocking at floor and roof diaphragms or provide new plywood sheathing at interior wood framed walls to reduce diaphragm spans.
Cafeteria [D]	• Provide refined analysis and corresponding destructive investigation.

3.12 Ruch Community School

Figure 12: Ruch Community School

3.12.1 Seismic Evaluation Summary

The following table summarizes the building type, age, and construction for structures on this campus. It also includes a summary of the seismic assessment findings.

Building Label (RVS/ZCS)	Building Type	Year Built, Remodel, and/or Retrofit	Structural Life Safety Assessment	Relative Hazard Severity
Administration [A]	W1	1950, 1999, 2017	Compliant	L.S. Compliant
Classrooms [B]	RM1	1950, 2017	Compliant	L.S. Compliant
Gymnasium[1] & Stage [C]	RM1	1954, 1960, 1968, 2017	Compliant	I.O. Compliant
Classrooms [D]	RM1/W2	1981	Noncompliant	Moderate
Classrooms [E]	W2	1977, 2017	Compliant	L.S. Compliant
Classrooms [F]	URM	1914, 2017	Compliant	L.S. Compliant
Modular [G]	W1	Est. 2000	Noncompliant	Low
Classrooms [H]	W2	Est. 1970	Noncompliant	Low

Ancillary to Gymnasium [I]	RM1	1968, Est. 1970	Noncompliant	Moderate
Classrooms/ Library [J]	RM1	Est. 1955, 1968, 1996	Noncompliant	Moderate
Bathrooms [K]	RM1	Est. 1960	Noncompliant	Moderate

1. *Structure was retrofit to meet ASCE 41 Life Safety provisions and structural Immediate Occupancy (IO) provisions. The retrofit was not expanded to satisfy non-structural IO requirements and therefore does not meet the full IO designation per ASCE 41.*

3.12.2 Structure Summary

The following summarizes the building timeline and structural systems for each portion of Ruch Community School:

- **Original 1913 Classrooms**: The original building is one story and consists of unreinforced masonry walls with a flexible wood roof diaphragm. The roof consists of plywood sheathing over 1x skip sheathing over wood joists bearing on exterior unreinforced masonry walls. The foundation consists of cast-in-place concrete stem walls and footings. This structure houses multiple classrooms with an approximate footprint of 2,000 square feet.
- **1950's Addition**: This administrative addition consists of light timber construction with a flexible wood framed roof diaphragm. The roof consists of plywood sheathing over timber joists bearing on exterior wood framed walls. The foundation consists of masonry stem walls with cast-in-place concrete footings. The approximate footprint of this structure is 1,300 square feet.
- **1954 Gymnasium Addition**: The gymnasium building addition consists of reinforced masonry with flexible wood roof and floor diaphragms. The roof consists of 2x decking over heavy timber trusses bearing on reinforced masonry pilasters. The foundation consists of cast-in-place concrete stem walls and footings. The approximate footprint of this structure is 4,200 square feet.
- **1954 Classroom Addition**: This addition consists of reinforced masonry with a flexible wood roof diaphragm. The roof consists of diagonal sheathing over timber joists bearing on exterior masonry walls. The foundation consists of slab-on-grade with cast-in-place concrete footings. The approximate footprint of this building is 3,400 square feet.
- **1960 Stage/Cafeteria Addition**: This addition consists of reinforced masonry with flexible wood roof and floor diaphragms. The roof consists of 2x decking over glulam beams. Two storage lofts are located at the North and South of this addition. The storage loft floors consist of plywood sheathing over timber joists bearing on masonry walls. The first floor consists of 2x decking over large glulam beams bearing on reinforced masonry pilasters. The foundation consists of slab-on-grade with cast-in-place concrete footings. The approximate floor area of this addition is 6,300 square feet.

- **1968 Classroom Addition**: This addition consists of reinforced masonry with a flexible wood roof diaphragm. The roof consists of plywood sheathing over open bar web joists bearing on exterior masonry walls. The foundation consists of slab-on-grade with cast-in-place concrete footings. The approximate footprint of this structure is 2,400 square feet.
- **1970 Classroom Addition**: This addition consists of light timber framed walls with a flexible wood roof diaphragm. The roof consists of plywood sheathing over timber joists bearing on exterior wood framed walls. The foundation consists of cast-in-place concrete stem walls and footings. The approximate footprint of this addition is 2,400 square feet.
- **1977 Classroom Addition**: This addition consists of light timber framed walls with a flexible wood roof diaphragm. The roof consists of plywood sheathing over I-joists bearing on exterior wood framed walls. The foundation consists of cast-in-place concrete stem walls and footings. The approximate footprint of this addition is 2,500 square feet.
- **1981 Classroom Addition**: This addition consists of reinforced masonry with a flexible wood roof diaphragm. The roof consists of blocked plywood panels over open bar web joists bearing on exterior masonry walls. The foundation consists of slab-on-grade with cast-in-place concrete footings. The approximate footprint of this structure is 4,000 square feet.
- **1996 Library Addition**: This addition consists of reinforced masonry with a flexible wood roof diaphragm. The roof consists of plywood sheathing over manufactured trusses bearing on exterior masonry walls and an interior glulam beam. The foundation consists of slab-on-grade with cast-in-place concrete footings. This addition house the current library and media center with an approximate footprint of 2,600 square feet.
- **2000's Modular Addition**: In the early 2000's a new modular was installed replacing existing deteriorated structures.
- **2006 Bond Improvements**: In 2006 the gym trusses were strengthened.
- **Seismic Retrofit 2017**: In the summer of 2017 the 1913 classrooms, 1950 admin addition, 1954 classroom additions, 1960 stage/cafeteria addition, and the 1977 classroom addition received a seismic upgrade bringing the structures up to Life Safety standards as defined in the ASCE 41. The 1954 gymnasium's structural systems were upgraded to meet the Immediate Occupancy structural performance provisions of ASCE 41. Letters stating compliance with Life Safety for the affected areas are on file with the District.

3.12.3 Observed Deficiencies:

The following list summarizes the structural seismic deficiencies observed during our visual inspections and/or original construction documents:

Building Label (RVS/ZCS)	Deficiency
1968 Classrooms [J]	• WALL ANCHORAGE: Out-of-plane connections at the top of wall are not present. • LOAD PATH: Blocking is not present between top of masonry wall to roof diaphragm. • DIAPHRAGM SPAN: Unblocked plywood diaphragm spans are greater than 40 feet.
1981 Classrooms [D]	• WALL ANCHORAGE: Out-of-plane connections at the top of wall are not present.
1996 Library Addition [J]	• WALL ANCHORAGE: Out-of-plane connections at the top of wall are not present.
1970 Storage, 1960 Bathroom [K,I]	• WALL ANCHORAGE: Out-of-plane connections at the top of wall are not present or are insufficiently detailed.
1970 Classrooms [H]	• SPAN: Unblocked wood diaphragm spans further than 40 feet.

3.12.4 Recommendations

The following are rehabilitation recommendations to address the observed deficiencies. In addition, for pursuit of Life Safety compliance we recommend the additional evaluation, field investigation, and non-structural assessment as noted in Section 2.

Building Label (RVS/ZCS)	Recommendations
1968 Classrooms [J]	• Provide new out-of-plane ties at the top of CMU walls. • Provide shear panels between existing trusses for transfer of seismic forces to CMU walls. • Provide new blocking and re-nail existing roof sheathing.
1981 Classrooms [D]	• Provide new out-of-plane ties at the top of CMU walls. • Provide new CMU shear walls at strategic locations for adequate in-plane shear capacity. • Provide new ties at glulam beams to CMU Pilasters.

1996 Library Addition [J]	• Provide new out-of-plane ties at the top of CMU walls. • Provide new ties at glulam beams to CMU walls.
1970 Storage, 1960 Bathroom [K,I]	• Provide new out-of-plane ties at the top of existing walls.
1970 Classrooms [H]	• Provide refined analysis and corresponding destructive investigation. • Provide diaphragm blocking or additional shearwall lines in-plane where determined to be deficient.

3.13 Washington Elementary School

Figure 13: Washington Elementary School

3.13.1 Seismic Evaluation Summary

The following table summarizes the building type, age, and construction for structures on this campus. It also includes a summary of the seismic assessment findings.

Building Label (RVS/ZCS)	Building Type	Year Built, Remodel, and/or Retrofit	Structural Life Safety Assessment	Relative Hazard Severity
Administration & Classrooms [A1]	C1/C2	1931,1949, 2007	Noncomplaint	Moderate
Gymnasium [A1]	C1/C2	1949, 2007	Complaint	L.S. Compliant
Classrooms [A2]	C2	1949, 2007	Noncompliant	Moderate
Classrooms [B]	W2	1995, 2007	Noncompliant	Low
Media Center [C]	RM1	1987, 2007	Noncompliant	Low

3.13.2 Structure Summary

The following summarizes the building timeline and structural systems for each portion of Washington Elementary School:

- **1931 Original**: The original school consists of concrete walls with a flexible wood second floor and roof diaphragm. Areas of the main floor framing consist of wood framing over concrete stem walls; the foundation consists of slab-on-grade with cast-in-place concrete stem walls. The second floor consists of wood joists bearing on exterior concrete walls and interior wood framed walls. The roof consists of plywood sheathing over wood joists. This building houses classrooms, an auditorium, and an administrative area with an approximate footprint of 14,800 square feet and an overall floor area of 29,600 square feet.
- **1949 Addition**: This addition consists of concrete walls with a flexible wood second floor and roof diaphragm. The roof consists of plywood sheathing over wood joists. The second-floor diaphragm consists of wood joists bearing on exterior concrete walls and interior wood framed walls. The foundation consists of slab-on-grade with concrete stem walls. This building houses classrooms and has an approximate footprint of 6,500 square feet and an overall floor area of approximately 13,000 square feet.
- **1987 Media Center Addition**: This addition consists of CMU walls with a flexible wood roof diaphragm. The roof consists of plywood sheathing over wood joists and truss joists. The foundation consists of slab-on-grade with concrete stem walls. This addition has an approximate footprint of 4,100 square feet.
- **1995 Addition**: This addition consists of wood framed walls with a flexible wood roof diaphragm. The roof consists of plywood sheathing over wood joists bearing on exterior and interior walls. The foundation consists of slab-on-grade with concrete stem walls. This addition has an approximate footprint of 3,300 square feet.
- **2007 Bond Improvements**: In 2007 a new HVAC system was installed throughout the school, new flooring was added, interior spaces were renovated, the school received new paint, an elevator was added, and the site received accessibility upgrades. In 2008, additional parking and fencing was installed to the site. The bond work also included structural upgrades, including the following at various areas: new structural sheathing on roof systems, new in-plane and out-of-plane connections at the top of the concrete walls, and tension ties. A covered seating area was also added.
- **2009 Seismic Retrofit:** In 2009, the Medford School District applied for and received a grant through the state Seismic Rehabilitation Grant Program to finish seismic upgrades to the original 1931 school. It is known that these upgrades were installed in 2011, but full project construction documents are not available at time of writing. It is known the installed improvements included new structural sheathing at walls and diaphragms, out-of-plane anchors, and new drag elements. These upgrades improve the seismic performance of the structure and reduce seismic hazard. The grant number for this project through the State of Oregon Seismic Rehabilitation Grant Program is SRGP 10S109. A final project performance report is on file with the Medford School District 549C summarizing the nature of the structural work performed to bring Building A up to Life Safety Standards based on ASCE 31 Tier 1

seismic evaluation, but the precise limits of the retrofit are unverified at time of writing. The relative seismic hazard rating may be adjusted as additional project information becomes available.

- **2021 Seismic Investigation**: ZCS performed additional investigations to confirm the extent of seismic retrofit installed within Buildings ‘A1’ and ‘A2’ during the District Bond Campaign around 2008. These investigations were limited to accessible framing cavities and attics, no destructive investigations were performed as part of these investigations. In addition, geotechnical investigations were performed on this site to evaluate site specific hazards such as Liquefaction. The results of the geotechnical investigations are summarized in Appendix A and Appendix D.

  ZCS confirmed install of new drag elements, and shear walls at the roof framing level at the classrooms portion of Buildings ‘A1’ and ‘A2’. First and second floor seismic retrofit scope at the ‘A1’ classroom building could not be confirmed without removal of wall, ceiling, and floor finishes. Confirmation of new in-plane connection hardware of Building ‘A1’ and ‘A2’ could not be confirmed without removal of existing floor finishes or roofing materials. It should be noted additional seismic deficiencies per ASCE 41-17 Tier 1 checklists were identified during our site investigations and are summarized in section 3.13.3.

  These findings within classroom portion of Buildings ‘A1’ and ‘A2’ provide confirmation that some level of seismic hazard mitigation of Washington Elementary has taken place. It should be noted a basis of design for the seismic retrofit scope for these structures was not confirmed with the Architect/Engineer of Record, all retrofit scope install could not be verified, and additional deficiencies were identified. The seismic retrofit scope of this structure appears to have been focused on a seismic hazard reduction and not an all-encompassing seismic retrofit. Therefore, the structural life safety designation of noncompliant and relative seismic hazard severity designation of low has been updated to Moderate for the ‘A1’ and ‘A2’ classroom building portions.

  ZCS confirmed new concrete wall infill, new shotcrete wall installation, out-of-plane connection hardware, stongback columns at existing pilasters, and plywood roof sheathing and drag elements were confirmed at the gymnasium portion of Building ‘A1’ as noted in the 2007 Washington Elementary School construction drawings.

  The findings at the gymnasium portion of Building ‘A1’ confirm seismic hazard mitigation has taken place. A basis of design for the seismic retrofit scope was confirmed with DCI Engineers. The retrofit scope was evaluated in accordance with ASCE 31-04 and designed to the 2006 IBC utilizing a Life Safety performance objective. With the information provided DCI Engineers and the on-site verifications the structural life safety designation for the gymnasium portion of Building ‘A1’ has been updated to compliant.

3.13.3 Observed Deficiencies:

The following list summarizes the structural seismic deficiencies observed during our visual inspections and/or original construction documents:

Building Label (RVS/ZCS)	Deficiency
1931 Administration, Classroom [A1]	• INFORMATION: Insufficient information to confirm level of retrofit design. • LOAD PATH: In-plane connection hardware could not be confirmed. • REINFORCING STEEL: Concrete walls are under-reinforced for minimum steel per Tier 1 checklists. • WALL ANCHORAGE: Out-of-plane connections at wall to roof and floor framing could not be confirmed. • WALL ANCHORAGE: Out-of-plane connections at stairway walls to roof and floor framing is not present. • DIAPHRAGM SPAN: Unblocked plywood roof diaphragm spans greater than 40 feet. • DIAPHRAGM SPAN: Straight sheathed floor diaphragm spans greater than 24 feet.
1949 Classrooms and 1987 Media Center [A2, C]	• INFORMATION: Insufficient information to confirm level of retrofit design. • LOAD PATH: In-plane connection hardware could not be confirmed. • REINFORCING STEEL: Concrete walls are under-reinforced for minimum steel per Tier 1 checklists. • WALL ANCHORAGE: Out-of-plane connections at wall to roof and floor framing could not be confirmed. • WALL ANCHORAGE: Out-of-plane connections at stairway walls to roof and floor framing is not present. • DIAPHRAGM SPAN: Unblocked plywood roof diaphragm spans greater than 40 feet. • DIAPHRAGM SPAN: Straight sheathed floor diaphragm spans greater than 24 feet.
1995 Classroom [B]	• INFORMATION: Destructive investigation required to verify conformance with record drawings and detailing.

3.13.4 Recommendations

The following are rehabilitation recommendations to address the observed deficiencies. In addition, for pursuit of Life Safety compliance we recommend the additional evaluation, field investigation, and non-structural assessment as noted in Section 2.

Building Label (RVS/ZCS)	Recommendations
1931 Administration, Classroom [A1]	• Obtain additional/supporting design information for the completed seismic hazard reduction and remodel. • LOAD PATH: Provide destructive investigations to confirm remaining concealed seismic hardware install. • REINFORCING STEEL: Provide Tier 2 analysis of under-reinforced concrete wall performance or provide new vertical elements to support walls for out-of-plane forces. • WALL ANCHORAGE: Provide selective destructive investigations at roofing perimeter to confirm remaining concealed seismic hardware install. • WALL ANCHORAGE: Provide new out-of-plane connection hardware at concrete stairway walls to existing second floor and roof diaphragms. • DIAPHRAGM SPAN: Provide diaphragm blocking or additional shear walls to reduce diaphragm spans. • DIAPHRAGM SPAN: Provide new plywood sheathing over existing straight sheathed second floor diaphragms.
1949 Classrooms and 1987 Media Center [A2, C]	• Obtain additional/supporting design information for the completed seismic hazard reduction and remodel. • LOAD PATH: Provide destructive investigations to confirm remaining concealed seismic hardware install. • REINFORCING STEEL: Provide Tier 2 analysis of under-reinforced concrete wall performance or provide new vertical elements to support walls for out-of-plane forces. • WALL ANCHORAGE: Provide selective destructive investigations at roofing perimeter to confirm remaining concealed seismic hardware install. • WALL ANCHORAGE: Provide new out-of-plane

	connection hardware at concrete stairway walls to existing second floor and roof diaphragms. • DIAPHRAGM SPAN: Provide diaphragm blocking or additional shear walls to reduce diaphragm spans. • DIAPHRAGM SPAN: Provide new plywood sheathing over existing straight sheathed second floor diaphragms.
1995 Classroom [B]	• Provide destructive investigation and refined analysis of building if required (Tier 2).

3.14 Wilson Elementary School

Figure 14: Wilson Elementary School

3.14.1 Seismic Evaluation Summary

The following table summarizes the building type, age, and construction for structures on this campus. It also includes a summary of the seismic assessment findings.

Building Label (RVS/ZCS)	Building Type	Year Built, Remodel, and/or Retrofit	Structural Life Safety Assessment	Relative Hazard Severity
Administration [A]	RM1	1958, 2007	Noncompliant	Moderate
Media Center & Gymnasium [B]	RM2	1958, 1990, 2008	Noncompliant	Moderate
Classroom [C]	RM1	1958, 2007	Noncompliant	Moderate
Cafeteria [D]	RM1	1992, 2007	Noncompliant	Moderate
Classroom [E]	RM1	1958, 2008	Noncompliant	Moderate
Classroom [F]	RM1	1963, 1992, 1995, 2008	Noncompliant	Moderate
Classroom [G]	RM1	1975, 1995, 2008	Noncompliant	Moderate
Classroom [H]	RM1	1958, 2008	Noncompliant	Moderate
Classroom [I]	W1/MOD	2015	Noncompliant	Moderate

3.14.2 Structure Summary

The following summarizes the building timeline and structural systems for each portion of Wilson Elementary School:

- **1958 Original**: The original single-story structure consists of CMU walls with a flexible wood roof diaphragm. The foundation consists of slab-on-grade with concrete stem walls. The roof is framed with wood rafters bearing on glulam beams and the diaphragm consists of structural sheathing added in 2008. This structure houses several classrooms and has a footprint of approximately 30,900 square feet.
- **1963 Through 1995 Additions & Renovations**: These buildings are multiple structures added with similar construction types at different times. The construction consists of CMU walls with flexible wood roof diaphragms. The foundations are slab-on-grade with concrete stem walls, and the roof framing consists of wood joists or wood trusses, new plywood roof sheathing was added in 2008. These structures house, classrooms, bathrooms, a dining hall, and have a footprint of approximately 15,500 square feet.
- **2008 Bond Improvements**: In 2008 Wilson Elementary received upgrades to the existing structural and non-structural systems. Interior spaces were renovated; new flooring was added with new paint and finishes both interior and exterior and a new HVAC system was installed throughout the school as well. The structural upgrades were designed by DCI Engineers in 2008 utilizing the ASCE 31 "Seismic Evaluation of Existing Buildings". These upgrades include new structural sheathing on roof systems, new in-plane and out-of-plane connections, and new tension ties at beams to the top of CMU walls. These upgrades improve the seismic performance of the structure and reduce seismic hazard.
- **2015 Modular Unit**: A single story modular classroom was added to the campus in 2015.
- **2021 Seismic Investigation**: ZCS performed additional investigations to confirm the extent of seismic retrofit installed within Building 'A' through 'H' during the District Bond Campaign around 2008. These investigations were limited to accessible framing cavities and attics, no destructive investigations were performed as part of these investigations. In addition, geotechnical investigations were performed on this site to evaluate site specific hazards such as Liquefaction. The results of the geotechnical investigations are summarized in Appendix A and Appendix D. The geotechnical investigations revealed the site has a 'very high' liquefaction potential. This additional deficiency is reflected in section 3.14.3.

  ZCS confirmed install of out-of-plane connection hardware between the existing roof framing to existing CMU walls in buildings 'A', 'B' (gym), 'C', 'E', 'G', and 'H'. It should be noted gable end wall out-of-plane hardware install could not be confirmed at Building 'C', 'E', and 'H' without removal of roofing material. Out-of-plane hardware was not present at the north exterior walls of Buildings 'A', 'C', 'E', and 'H' and was not included in the 2008 'Wilson Elementary School Re-roof / Seismic' construction drawings.

Seismic retrofit hardware install could not be confirmed at Building 'F', 'B' (library), without removal of roofing material. Install of out-of-plane anchorage was not present at Building 'B' (stage) and Building 'B' (boiler room) as noted in the 2008 construction drawings. Confirmation of new in-plane connection hardware and plywood sheathing at the roof level install could not be confirmed without removal of roofing materials.

These findings provide confirmation that seismic hazard mitigation has taken place throughout Wilson Elementary's structures. The basis of design for the seismic retrofit scope was confirmed with the Architect/Engineer of Record. The retrofit scope was evaluated in accordance with ASCE 31-04 and designed to the 2006 IBC. It should be noted, all retrofit scope items could not be verified, and additional deficiencies were identified during our site and geotechnical investigations. Therefore, the structural life safety designation of noncompliant and relative seismic hazard severity designation has been updated to moderate for Buildings 'A', 'B', 'C', 'D', 'E', 'F', 'G', 'H', and 'I'.

3.14.3 Observed Deficiencies:

The following list summarizes the structural seismic deficiencies observed during our visual inspections and/or original construction documents:

Building Label (RVS/ZCS)	Deficiency
1958 Admin, Classroom [A]	• LIQUEFACTION: Site is susceptible to very high liquefaction potential. • LOAD PATH: In-plane connection hardware at roof to CMU walls not confirmed. • DIAPHRAGM: Plywood sheathing not confirmed at roof deck. • WALL ANCHORAGE: Out-of-plane connection hardware not present at north exterior CMU walls. • WALL ANCHORAGE: Out-of-plane connection hardware not confirmed at CMU gable end walls.
1958, 1990 Gym [B]	• LIQUEFACTION: Site is susceptible to very high liquefaction potential. • LOAD PATH: In-plane connection hardware not confirmed at top of wall to roof. • DIAPHRAGM: Plywood sheathing not confirmed at roof deck. • WALL ANCHORAGE: Out-of-plane connection hardware not present at stage roof purlins to CMU walls. • WALL ANCHORAGE: Out-of-plane connection hardware not present at boiler

	room roof framing to CMU walls.
1958, 1990 Library [B]	• LIQUEFACTION: Site is susceptible to very high liquefaction potential. • LOAD PATH: In-plane connection hardware not confirmed at top of wall to roof. • DIAPHRAGM: Plywood sheathing not confirmed at roof deck. • WALL ANCHORAGE: Out-of-plane connection hardware not confirmed at roof to CMU walls.
1958 Classroom [C]	• LIQUEFACTION: Site is susceptible to very high liquefaction potential. • LOAD PATH: In-plane connection hardware at roof to CMU walls not confirmed. • DIAPHRAGM: Plywood sheathing not confirmed at roof deck. • WALL ANCHORAGE: Out-of-plane connection hardware not present at north exterior CMU walls. • WALL ANCHORAGE: Out-of-plane connection hardware not confirmed at interior CMU walls and gable end walls.
1992 Cafeteria [D]	• LIQUEFACTION: Site is susceptible to very high liquefaction potential. • INFORMATION: Insufficient information to confirm level of retrofit design.
1958 Classroom [E]	• LIQUEFACTION: Site is susceptible to very high liquefaction potential. • LOAD PATH: In-plane connection hardware at roof to CMU walls not confirmed. • DIAPHRAGM: Plywood sheathing not confirmed at roof deck. • WALL ANCHORAGE: Out-of-plane connection hardware not present at north exterior CMU walls. • WALL ANCHORAGE: Out-of-plane connection hardware not confirmed at interior CMU walls and gable end walls.
1963, 1992, 1995 Classroom [F]	• LIQUEFACTION: Site is susceptible to very high liquefaction potential. • LOAD PATH: In-plane connection hardware at roof to CMU walls not confirmed. • DIAPHRAGM: Plywood sheathing not

	confirmed at roof deck. • WALL ANCHORAGE: Out-of-plane connection hardware not confirmed at roof to CMU walls.
1975, 1995, 2008 Classroom [G]	• LIQUEFACTION: Site is susceptible to very high liquefaction potential.
1958 Classroom [H]	• LIQUEFACTION: Site is susceptible to very high liquefaction potential. • LOAD PATH: In-plane connection hardware at roof to CMU walls not confirmed. • DIAPHRAGM: Plywood sheathing not confirmed at roof deck. • WALL ANCHORAGE: Out-of-plane connection hardware not present at north exterior CMU walls. • WALL ANCHORAGE: Out-of-plane connection hardware not confirmed at interior CMU walls and gable end walls.
2015 Modular Unit [I]	• LIQUEFACTION: Site is susceptible to very high liquefaction potential. • ANCHORAGE: Insufficient information to confirm foundation anchorage.

3.14.4 Recommendations:

The following are rehabilitation recommendations to address the observed deficiencies. In addition, for pursuit of Life Safety compliance we recommend the additional evaluation, field investigation, and non-structural assessment as noted in Section 2.

Building Label (RVS/ZCS)	Recommendations
1958 Admin, Classroom [A]	• LIQUEFACTION: Provide additional geotechnical investigations across site to confirm widespread liquefaction potential. • LOAD PATH: Provide destructive investigations at select areas of roof to confirm remaining concealed seismic hardware install. • DIAPHRAGM: Provide destructive investigations at select areas of roof to confirm plywood sheathing install. • WALL ANCHORAGE: Provide new out-of-plane connection hardware at north CMU walls. • WALL ANCHORAGE: Provide destructive investigations at select areas of roof to

	confirm remaining concealed seismic hardware install at gable end walls.
1958, 1990 Gym [B]	• LIQUEFACTION: Provide additional geotechnical investigations across site to confirm widespread liquefaction potential. • LOAD PATH: Provide destructive investigations to confirm remaining concealed seismic hardware install. • DIAPHRAGM: Provide destructive investigations at select areas of roof to confirm plywood sheathing install. • WALL ANCHORAGE: Provide out-of-plane connection hardware at stage roof framing to CMU walls. • WALL ANCHORAGE: Provide out-of-plane connection hardware at boiler room roof framing to CMU walls.
1958, 1990 Library [B]	• LIQUEFACTION: Provide additional geotechnical investigations across site to confirm widespread liquefaction potential. • LOAD PATH: Provide destructive investigations to confirm remaining concealed seismic hardware install. • DIAPHRAGM: Provide destructive investigations at select areas of roof to confirm plywood sheathing install. • WALL ANCHORAGE: Provide destructive investigations at select area of roof to confirm remaining concealed seismic hardware install at gable end walls.
1958 Classroom [C]	• LIQUEFACTION: Provide additional geotechnical investigations across site to confirm widespread liquefaction potential. • LOAD PATH: Provide destructive investigations at select areas of roof to confirm remaining concealed seismic hardware install. • DIAPHRAGM: Provide destructive investigations at select areas of roof to confirm plywood sheathing install. • WALL ANCHORAGE: Provide new out-of-plane connection hardware at north CMU walls. • WALL ANCHORAGE: Provide destructive investigations at select areas of roof to confirm remaining concealed seismic hardware install at gable end walls.

1992 Cafeteria [D]	• LIQUEFACTION: Provide additional geotechnical investigations across site to confirm widespread liquefaction potential. • INFORMATION: Provide destructive investigation and refined analysis of building if required (Tier 2).
1958 Classroom [E]	• LIQUEFACTION: Provide additional geotechnical investigations across site to confirm widespread liquefaction potential. • LOAD PATH: Provide destructive investigations at select areas of roof to confirm remaining concealed seismic hardware install. • DIAPHRAGM: Provide destructive investigations at select areas of roof to confirm plywood sheathing install. • WALL ANCHORAGE: Provide new out-of-plane connection hardware at north CMU walls. • WALL ANCHORAGE: Provide destructive investigations at select areas of roof to confirm remaining concealed seismic hardware install at gable end walls.
1963, 1992, 1995 Classroom [F]	• LIQUEFACTION: Provide additional geotechnical investigations across site to confirm widespread liquefaction potential. • LOAD PATH: Provide destructive investigations at roof level to confirm remaining concealed seismic hardware install. • DIAPHRAGM: Provide destructive investigations at select areas of roof to confirm plywood sheathing install. • WALL ANCHORAGE: Provide destructive investigations at select areas of roof to confirm remaining concealed seismic hardware install.
1975, 1995, 2008 Classroom [G]	• LIQUEFACTION: Provide additional geotechnical investigations across site to confirm widespread liquefaction potential.
1958 Classroom [H]	• LIQUEFACTION: Provide additional geotechnical investigations across site to confirm widespread liquefaction potential. • LOAD PATH: Provide destructive investigations at select areas of roof to confirm remaining concealed seismic hardware install.

	• DIAPHRAGM: Provide destructive investigations at select areas of roof to confirm plywood sheathing install. • WALL ANCHORAGE: Provide new out-of-plane connection hardware at north CMU walls. • WALL ANCHORAGE: Provide destructive investigations at select areas of roof to confirm remaining concealed seismic hardware install at gable end walls.
2015 Modular Unit [I]	• LIQUEFACTION: Provide additional geotechnical investigations across site to confirm widespread liquefaction potential. • Provide field verification of foundation anchorage.

3.15 Hedrick Middle School

Figure 15: Hedrick Middle School

3.15.1 Seismic Evaluation Summary

The following table summarizes the building type, age, and construction for structures on this campus. It also includes a summary of the seismic assessment findings.

Building Label (RVS/ZCS)	Building Type	Year Built, Remodel, and/or Retrofit	Structural Life Safety Assessment	Relative Hazard Severity
Classrooms & Administration [A]	C2/S2a/ W2/RM1	1954, 1996	Noncompliant	Low
Gymnasium[1] & Cafeteria [B]	C2	1954, 1996	Noncompliant	High
Classrooms [C]	PC1a	1996	Noncompliant	Low
'Classrooms [D]	C2/RM1	1956, 1960, 1996	Noncompliant	Low

1. An SRG grant application was submitted for all or part of this structure. See Structure Summary below for additional information.

3.15.2 Structure Summary

The following summarizes the building timeline and structural systems for Hedrick Middle School:

- **1954 Original**: The original building consists of concrete walls with brick veneer, a second story flexible wood diaphragm, and a flexible wood roof diaphragm. The second floor consists of 2x T&G decking over wood joists. The roof consists of diagonal sheathing over wood joists bearing on exterior walls, interior walls, and glulam beams. The foundation is slab-on-grade with cast-in-place concrete stem walls. The original gymnasium consists of concrete walls with a flexible wood roof diaphragm. The gymnasium also features an unreinforced masonry chimney that poses a potential falling hazard in a seismic event. The floor consists of wood joists over timber beams and columns. The roof consists of 2” decking over wood purlins bearing on heavy timber trusses. The foundation consists of cast-in-place concrete footings. The original buildings house classrooms, an office, gymnasium, cafeteria, and a library with an approximate footprint of 65,600 square feet and an approximate total floor area of 89,200 square feet.
- **1956 Addition**: This addition consists of tall concrete walls to the second story with wood framed walls above to the roof diaphragm. The second story is a flexible wood diaphragm with plywood sheathing over wood joists. The roof consists of wood decking over wood joists bearing on an interior beam and exterior walls. The foundation consists of slab-on-grade with cast-in-place concrete stem walls. The approximate footprint of this addition is 1,300 square feet and a total floor area of approximately 2,600 square feet.
- **1960 Addition**: This addition consists of concrete tilt-up wall panels with a flexible wood roof diaphragm. The roof consists of 4” T&G decking over glulam beams and exterior concrete walls. The foundation consists of slab-on-grade with cast-in-place concrete stem walls. This addition houses multiple classrooms and has an approximate footprint of 8,300 square feet.
- **1996 Addition and Structural Improvements**: A new media center and classroom were added in 1996. The media center consists of reinforced masonry walls with a steel roof deck diaphragm. The classroom addition consists of concrete walls with a cast-in-place concrete slab second floor over steel deck. The roof consists of steel deck roof over steel joists. The foundations of both additions are slab-on-grade with cast-in-place concrete stem walls. The approximate footprint of these additions is 25,200 square feet and a total floor area of approximately 35,100 square feet. In conjunction with the 1996 addition, structural improvements were made to the existing structures at Hedrick Middle School. These upgrades include new structural sheathing on the roof systems, new eccentrically braced steel frames at the first story and roof, connections to concrete walls, and new footings. The gymnasium was excluded from the 1996 structural upgrades. The deficiencies of the gymnasium are outlined below.

- **2018 SRG Applications and Additional Studies**: A memo regarding Gymnasium chimney was composed by ZCS in December of 2018. The chimney is notable because of its substantial height and lack of reinforcement. Findings are on file with the District. An SRG application was filed with the state for the gymnasium in November 2018. The application package includes additional information regarding deficiencies and recommended rehabilitations and is on file with the District. Funds for this application were not awarded for the November 2018 season.

3.15.3 Observed Deficiencies:

The following list summarizes the structural seismic deficiencies observed during our visual inspections and/or original construction documents:

Building Label (RVS/ZCS)	Deficiency
1954 **Original Gym** **[B]**	• WALL ANCHORAGE: Out-of-plane connections at the top of wall are not present. • SPAN: Unblocked wood diaphragm spans farther than 40 feet. • URM CHIMNEY: An unreinforced chimney extends above the roof line posing a potential falling hazard
1954, 1956, 1960 **Classrooms** **[A, D]**	• SPAN: Unblocked wood diaphragm spans farther than 40 feet. • WALL ANCHORAGE: Insufficient record information regarding out-of-plane connections at the top of wall.
1996 Classrooms **[C]**	• SPAN: Unblocked wood diaphragm spans farther than 40 feet. • WALL ANCHORAGE: Insufficient record information regarding out-of-plane connections at the top of wall.

3.15.4 Recommendations

The following are rehabilitation recommendations to address the observed deficiencies. In addition, for pursuit of Life Safety compliance we recommend the additional evaluation, field investigation, and non-structural assessment as noted in Section 2.

Building Label (RVS/ZCS)	Recommendations
1954 Original Gym [B]	• Provide refined analysis and corresponding destructive investigation. • Provide new out-of-plane connections at the top of concrete walls. • Provide new sheathing over 2x decking at gym roof system. • Provide retrofit or demolition of non-structural hazards including the URM chimney
1954 & 1956 Classrooms [A, D]	• Provide refined analysis and corresponding destructive investigation. • Provide diaphragm blocking or additional shearwall lines in-plane where determined to be deficient. • Provide new out-of-plane connections at the top of concrete walls pending results of analysis and destructive investigation.
1996 Classrooms [C]	• Provide refined analysis and corresponding destructive investigation. • Provide diaphragm blocking or additional shearwall lines in-plane where determined to be deficient. • Provide new out-of-plane connections at the top of concrete walls pending results of analysis and destructive investigation.

3.16 McLoughlin Middle School

Figure 16: McLoughlin Middle School

3.16.1 Seismic Evaluation Summary

The following table summarizes the building type, age, and construction for structures on this campus. It also includes a summary of the seismic assessment findings. Please see Appendix A for a campus map summarizing the campus construction and evaluation results.

Building Label (RVS/ZCS)	Building Type	Year Built, Remodel, and/or Retrofit	Structural Life Safety Assessment	Relative Hazard Severity
Classrooms & Cafeteria [A]	C2	Unkn. Original, 1996	Noncompliant	Low
Classrooms & Gymnasium [B]	C2a	Est. 1947, 1996	Noncompliant	Low
Locker Room [C]	RM1/ C2a	1996	Noncompliant	Very High
Classrooms [D]	RM1	1996	Noncompliant	Low
Gymnasium [E]	RM1/S1	1996	Noncompliant	Low
Administration & Classrooms [F]	W1	1996	Noncompliant	Low

3.16.2 Structure Summary

The following summarizes the building timeline and structural systems for each portion of Mcloughlin Middle School:

- **Original**: The original date of construction was not able to be verified at time of writing. The building consists of concrete walls and wood framed interior walls with a flexible wood second floor diaphragm, a rigid concrete second floor diaphragm in the corridors and the north rooms of the south leg, and a flexible wood roof diaphragm. The roof consists of plywood sheathing over wood joists bearing on concrete walls. The floor framing consists of wood joists bearing on concrete stem walls; the foundation consists of slab-on-grade in some areas with cast-in-place concrete stem walls. The original building houses classrooms, a boiler room, gymnasium, and an administrative area. The approximate footprint of this building is 22,500 square feet and a floor area of approximately 41,600 square feet.
- **1947 Addition**: This addition consists of concrete walls with wood framed interior walls, a flexible wood second floor diaphragm and a flexible wood roof diaphragm. The roof consists of trussed joists with diagonal sheathing; the roof in the new gymnasium consists of diagonal sheathing over wood joists bearing on heavy timber trusses. The second floor at the classroom wing consists of wood joists with wood subfloor bearing on concrete walls. The floor framing of the gym consists of wood joists supported on wood beams and posts; the foundation consists of cast-in-place concrete footings and stem walls. The foundation of the remaining building consists of slab-on-grade with concrete stem walls. This addition houses classrooms and a new gym, the approximate footprint is 20,100 square feet with an approximate floor area of 33,100 square feet.
- **Unknown Locker Room Addition**: A locker room was added to the campus at an unknown period prior to 1996. It is constructed from concrete walls and pilasters with interior masonry walls. There do not appear to be any structural improvement efforts undertaken during the later 1996 work noted below.
- **1996 Addition and Structural Improvements**: Five buildings were added to McLoughlin Middle School in 1996 including a new Gymnasium, Administrative Building, Music Building, Media Center, and bathroom addition. The new gymnasium consists of partial height CMU walls at the first level with a cast-in-place concrete second floor. The second-floor walls are steel stud framed. The steel stud framed walls contain 6-inch steel girts placed at 30 inches on-center horizontally. The roof consists of metal roof deck over bar joists bearing on steel beams and columns. The foundation consists of slab-on-grade with concrete stem walls. The Administrative addition consists of wood framed walls with a flexible wood second floor and roof diaphragm. The roof consists of plywood sheathing over truss joists bearing on exterior and interior walls. The second floor consists of plywood sheathing over truss joists bearing on exterior walls and interior beams and columns. The foundation consists of slab-on-grade with concrete stem walls. The Music Building addition consists of CMU walls to the second story with wood framed walls above the second floor. The roof consists of metal decking over steel bar joists bearing on exterior and interior walls.

- **1996 Continued**. The second-floor diaphragm consists of a cast-in-place concrete slab over steel deck on steel beams bearing on exterior and interior walls. The foundation consists of slab-on-grade with CMU stem walls. The Media Center addition consists of wood framed walls with a flexible wood roof diaphragm. The roof consists of plywood sheathing over TJ joists bearing on exterior walls and an interior steel beam. The foundation consists of slab-on-grade with concrete stem walls. The bathroom addition consists of CMU walls to the second floor and wood framed walls to the roof with steel columns and a flexible wood second floor and roof diaphragm. The roof consists of plywood sheathing over TJ joists bearing on exterior and interior walls. The second floor consists of plywood sheathing over wood joists bearing on CMU walls and beams. Various structural improvements were added to the lateral system of McLoughlin Middle School during the 1996 Additions and Alterations. These upgrades include new plywood sheathing at existing diaphragms, new drag beams, braced columns, out-of-plane connections, new shear walls, new hold downs, and new footings. These upgrades have improved the various deficiencies that were observed when reviewing the original construction documents. These upgrades improve the seismic performance of the structure and reduce seismic hazard.
- **2011 Bond Improvements**: In 2011 a storage area and restrooms were added, and the office entry also received security upgrades.
- **2021 Seismic Investigation**: Geotechnical investigations were performed on this site to evaluate site specific hazards such as Liquefaction. The results of the geotechnical investigations are summarized in Appendix A and Appendix D.

3.16.3 Observed Deficiencies:

The following list summarizes the structural seismic deficiencies observed during our visual inspections and/or original construction documents:

Building Label (RVS/ZCS)	Deficiency
Classrooms, Cafeteria, & Gym [A,B]	• INFORMATION: Insufficient information to confirm the level of retrofit design
Pre-1996 Locker room [C]	• SPAN: Unblocked wood diaphragm spans farther than 40 feet. • WALL ANCHORAGE: Insufficient information regarding out-of-plane anchorage.
1996 Classroom and Gymnasium [D, E]	• SPAN: Unblocked wood diaphragm spans farther than 40 feet.
Administration & Classrooms [F]	• SPAN: unblocked diaphragm spans farther than allowable 40 ft

3.16.4 Recommendations

The following are rehabilitation recommendations to address the observed deficiencies. In addition, for pursuit of Life Safety compliance we recommend the additional evaluation, field investigation, and non-structural assessment as noted in Section 2.

Building Label (RVS/ZCS)	Recommendations
Classrooms, Cafeteria, & Gym [A,B]	• Obtain additional/supporting information for the completed retrofit
Pre-1996 Locker room [C]	• Provide refined analysis and investigation. • Provide diaphragm blocking or additional shearwall lines in-plane where determined to be deficient. • Provide new wall anchorage where determined to be deficient.
1996 Classroom and Gymnasium [D, E]	• Provide refined analysis and corresponding destructive investigation. • Provide diaphragm blocking or additional shearwall lines in-plane where determined to be deficient.
Administration & Classrooms [F]	• Provide field verification of construction and non-structural elements. Provide refined analysis (Tier 2), and diaphragm strengthening or interior shearwalls if required

3.17 Oakdale Middle School (Old Central High School)

Figure 17: Oakdale Middle School

3.17.1 Seismic Evaluation Summary

The following table summarizes the building type, age, and construction for structures on this campus. It also includes a summary of the seismic assessment findings. Please see Appendix A for a campus map summarizing the campus construction and evaluation results.

Building Label (RVS/ZCS)	Building Type	Year Built, Remodel, and/or Retrofit	Structural Life Safety Assessment	Relative Hazard Severity
Gymnasium [A]	PC1	1985	Noncompliant	Moderate
Library & Classrooms [B1]	C2	1931, 1949, 1958, 2010	Noncompliant	Low
Classrooms [B2]	C2/W2	1931, Unkn., 1949, 1958, 2010	Noncompliant	High
Auditorium [C]	C2	1931, 2010, 2011	Noncompliant	Low
Classrooms & Kitchen [D]	C2/W2	1949, 1958, 2010, 2012	Noncompliant	Low
Note: Labels E and F not used from RVS				
Grandstands [G]	PC1/C2/RM1/S1	1962	Noncompliant	High
Grandstands [H]	C1/RM1/S1	1948	Noncompliant	High

3.17.2 3.16.1 Structure Summary

The following summarizes the building timeline and structural systems for each portion of Central High School:

- **Est. 1931**: Record drawings were not available for the original classroom wings. The structures were constructed from concrete walls with flexible wood diaphragms.
- **1948 Grandstand Addition**: Grandstands were added to the east side of the field. They are constructed from concrete frames and steel columns with a heavy timber roof. The stairs and seating areas are constructed from concrete, as are the foundations.
- **1949 Boiler Room Addition**: A boiler room was added to the main classroom portion in 1949. It was constructed utilizing concrete walls, concrete foundation, and a reinforced concrete deck for the roof framing.
- **1986 Conversion**: During the conversion of Medford High to Mid-High (Central) the school received various upgrades. Due to the lack of original construction documents the construction types were determined from the Mid-High Conversion documents and the 2010-2012 School Bond documents. The existing school buildings consist of concrete walls with a flexible wood roof diaphragm and flexible and rigid second-floor diaphragms. The roof consists of 1x straight decking bearing on beams and concrete walls. The walls consist of reinforced concrete bearing walls at both the exterior and interior with wood framed partition walls. The main floor consists of slab-on-grade, reinforced concrete slab over beams, and timber framed joists with diagonal sheathing and concrete stem walls. The Central High School building houses district administrative areas, classrooms, a gymnasium, and library with an overall approximate area of 152,000 square feet.
- **2010 Bond Improvements**: During the school bond Central High received various structural, mechanical, plumbing, electrical, and aesthetics upgrades. The mechanical upgrades included new HVAC systems and improvements to existing mechanical systems. The mechanical, electrical and aesthetics upgrades included new plumbing fixtures and lines, new lighting and other electrical upgrades, new interior finishes including carpet, cabinetry, and suspended ceilings which were installed throughout Central High School. The structural upgrades focused primarily on the academics wings of the school and included new out-of-plane ties at flexible diaphragms to existing concrete walls, new shear walls, new holds downs, blocking, new structural sheathing and nailing at the roof diaphragms, and new connections to increase the lateral performance of the school. Documentation indicates that some areas were retrofit in conformance with ASCE 31, and additional clarification should be sought for scope and design level.
- **2021-2022 Seismic Retrofit**: During the time of writing Buildings ‘A’, ‘B1’, ‘B2’, ‘C’, and ‘D’ on this campus are undergoing a seismic retrofit. Building ‘A’ is funded by the state SRG program to bring the gymnasium into Immediate Occupancy compliance, and Buildings ‘B1’, ‘B2’, ‘C’, and ‘D’ are funded by the District as part of the middle school conversion/renovation project and will bring the structures into Life Safety compliance.

3.17.3 Observed Deficiencies:

The following list summarizes the structural seismic deficiencies observed during our visual inspections and/or original construction documents:

Building Label (RVS/ZCS)	Deficiency
1985 Gym [A]	• Currently undergoing a seismic retrofit
1931, 2010 Library & Classrooms [B1]	• Currently undergoing a seismic retrofit
1931/1949 Original [B2]	• Currently undergoing a seismic retrofit
1931, 2010 Auditorium [C]	• Currently undergoing a seismic retrofit
1949, 2010 Classrooms and Kitchen [D]	• Currently undergoing a seismic retrofit
1948/1962 Grandstands [G, H]	• Insufficient design information for moment connection at canopy posts.

3.17.4 Recommendations

The following are rehabilitation recommendations to address the observed deficiencies. In addition, for pursuit of Life Safety compliance we recommend the additional evaluation, field investigation, and non-structural assessment as noted in Section 2.

Building Label (RVS/ZCS)	Recommendations
1985 Gym [A]	• Currently undergoing a seismic retrofit
1931, 2010 Library & Classrooms [B1]	• Currently undergoing a seismic retrofit
1931/1949 Original [B2]	• Currently undergoing a seismic retrofit
1931, 2010 Auditorium [C]	• Currently undergoing a seismic retrofit
1949, 2010 Classrooms and Kitchen [D]	• Currently undergoing a seismic retrofit

1948/1962 Grandstands [G, H]	• Provide refined analysis and corresponding field investigation. • Provide connection strengthening as required.

3.18 North Medford High School

Figure 18: North Medford High School

3.18.1 Seismic Evaluation Summary

The following table summarizes the building type, age, and construction for structures on this campus. It also includes a summary of the seismic assessment findings.

Building Label (RVS/ZCS)	Building Type	Year Built, Remodel, and/or Retrofit	Structural Life Safety Assessment	Relative Hazard Severity
Administration [A]	W2	1965, 1985, 2009	Noncompliant	Low
Classrooms [B]	W2	1965, 1985, 2008	Compliant	L.S. Compliant
Classrooms [C]	W2	1965	Noncompliant	Low
Day Care [D]	W2	Unkn., 1992	Noncompliant	Low
Classrooms [E]	RM1	Unkn., 1985, 2009	Noncompliant	Low
Classrooms [F]	RM1	1965, 2009	Noncompliant	Moderate
Classrooms [G]	W2	1965, 1985	Noncompliant	Low
Gymnasium [H]	RM1/C1	1965, 2010	Noncompliant	High
Cafeteria	W2	1965, 1985, 2010	Noncompliant	Low

[I]				
Theater [J]	W2	1965, 1985, 2010	Noncompliant	High
Media Center [K]	W2/CFS 1	1965, 2008	Compliant	L.S. Compliant
Classrooms [L]	W2/RM1	1965, 2010	Noncompliant	High

3.18.2 Structure Summary:

The following summarizes the building timeline and structural systems for each portion of the campus:

- **School Buildings 1965**: Several buildings were constructed on the campus during this year. They included classrooms, administration building, cafeteria, fine arts building, gymnasium, home economics building, humanities building, library building, science building, and the technical arts building. The types of construction varied, but primarily consisted of either light timber framing (wood roof framing and walls), or reinforced masonry buildings with light timber roof framing.
- **Remodel 1985**: Several buildings received interior and MEP improvements. No structural drawings were included with these improvements.
- **Addition 1992**: A childcare center was added to the campus. This was constructed of light timber framing, including wood roof joists, plywood roof sheathing, and plywood sheathed wood shear walls.
- **2010 Bond Improvements**: Several of the buildings on campus were structurally strengthened during this effort. Some facilities received select improvements or improvements limited to certain areas, while others received more comprehensive seismic improvements. Those that received more comprehensive seismic improvements include Buildings A, B, E, I, and K. Those that received partial improvements or improvements in limited areas include Buildings F, H, L, and J.
- **2021 Seismic Investigation**: ZCS performed additional investigations to confirm the extent of seismic retrofit installed within Building ‘A’, ‘B’, ‘E’, ‘F’, ‘I’, ‘J’ and ‘K’ during the District Bond Campaign around 2008. These investigations were limited to accessible framing cavities and attics, no destructive investigations were performed as part of these investigations. In addition, geotechnical investigations were performed on this site to evaluate site specific hazards such as Liquefaction. The results of the geotechnical investigations are summarized in Appendix A and Appendix D.

  ZCS confirmed install of new plywood shear wall install, connection hardware, and hold down anchorage install at the administration Building ‘A’ as noted in the 2008 and 2009 ‘North Medford High School Remodel’ construction documents. Per ASCE 41-17 Tier 1 checklists, the unblocked diaphragms exceed prescribed span limits.

  New plywood shear wall install, connection hardware, and hold down anchorage install was confirmed at the humanities Building ‘B’ as noted in the 2008 construction drawings. Diaphragm blocking install was also confirmed at areas of the roof framing as noted in the 2008 construction drawings. Per Tier 1 checklist compliance, this structure's life safety designation has been updated to compliant.

Installation of out-of-plane connection hardware, in-plane connection hardware, and plywood shear walls above CMU walls to roof framing in Building 'E' was confirmed as noted 2008 'North Medford High School Seismic Upgrade' construction documents. It should be noted an additional deficiency was discovered during the site investigations; the exterior canopy lacks out-of-plane connection anchorage to the existing CMU walls. These results are summarized in section 3.18.3.

In-plane connection hardware was confirmed in the northern portion of Building 'F', interior CMU walls were found to be partial height and lacked out-of-plane connection hardware as noted in the 2009 'North Medford High School Remodel' construction documents. In addition, out-of-plane connection hardware at existing exterior CMU walls was not noted within the construction documents and not identified during the site investigations. These results are summarized in section 3.18.3. The seismic retrofit scope of this structure appears to have been focused on a seismic hazard reduction and not an all-encompassing seismic retrofit. It should be noted the 2009 remodel construction documents did not include seismic hazard mitigation scope for the southern portion of this building.

New plywood shear wall and connection hardware install was confirmed at the cafeteria Building 'I' as noted in the 2010 North Medford High School Cafeteria Building construction drawings. It should be noted additional deficiencies were noted during our site investigations. These deficiencies include lack of out-of-plane connection hardware at CMU walls and plywood diaphragm spans exceeding limits prescribed in the ASCE 41-17 Tier 1 checklists. These results are summarized in section 3.18.3. The seismic retrofit scope of this structure appears to have been focused on a seismic hazard reduction and not an all-encompassing seismic retrofit.

ZCS confirmed out-of-plane connection hardware and strapping at the roof level of Building 'J' as noted in the 2010 'North Medford High School Remodel' construction documents. It should be noted this structure contains seismic deficiencies not addressed within the 2010 remodel construction documents. These results are summarized in section 3.18.3. The seismic retrofit scope of this structure appears to have been focused on a seismic hazard reduction and not an all-encompassing seismic retrofit.

ZCS confirmed install of kickers, plywood wall sheathing install, and connection hardware between the existing and new building portions of Building 'K' as noted in the 2009 'Noth Medford High School Remodel and Addition' construction documents. Per Tier 1 checklist compliance, this structure's life safety designation has been updated to compliant.

These findings provide confirmation that seismic hazard mitigation has taken place throughout North Medford High Schools structures. It should be noted a basis of design for the seismic retrofit scope of Buildings 'A', 'E', 'F', 'I', and 'J' was not confirmed with the Architects/Engineers of Record, and additional deficiencies were identified during site investigations. Therefore, the structural life safety designation of noncompliant and relative seismic hazard severity designations of low, moderate, and high remains unchanged for these structures.

- **SRG Applications**: An SRG application was filed with the state in February 2022. The scope of that application covered Building H. The application package includes additional information regarding deficiencies and recommended rehabilitations and is on file with the District.

3.18.3 Observed Deficiencies:

The following list summarizes the structural seismic deficiencies observed during our visual inspections and/or original construction documents:

Building Label (RVS/ZCS)	Deficiency
Administration [A]	• INFORMATION: Insufficient information to confirm level of retrofit design. • DIAPHRAGM SPAN: Unblocked plywood diaphragms exceed prescribed span limits.
Classrooms [E]	• CANOPIES: Exterior canopy lacks out-of-plane connection hardware to CMU walls.
Classrooms [F]	• INFORMATION: Insufficient information to confirm level of retrofit design. • WALL ANCHORAGE: CMU walls lack out-of-plane connection hardware. • WALL ANCHORAGE: Interior CMU walls lack out-of-plane connection hardware. Partial height interior CMU walls are not adequately supported for out-of-plane forces. • WOOD LEDGERS: Exterior canopies supported by wood ledgers do not have out-of-plane connection hardware to mitigate cross grain bending in wood ledgers. • DIAPHRAGM SPAN: Unblocked plywood diaphragms span greater than 40 feet.
Cafeteria [I]	• INFORMATION: Insufficient information to confirm level of retrofit design. • LOAD PATH: In-plane connection hardware at roof to shear walls not confirmed. • WALL ANCHORAGE: Out-of-plane connection hardware not present at interior CMU wall to roof framing at boiler room. • TRANSFER TO SHEAR WALLS: Interior CMU wall not adequately connected to roof diaphragm for transfer of seismic forces. • SPAN: Unblocked plywood diaphragms span

	greater than 40 feet.
Theater [J]	• SHEAR STRESS CHECK: Gypsum sheathed lower shear walls do not have adequate capacity for seismic forces. • TRANSFER TO SHEAR WALLS: Wood and CMU shear walls are not adequately connected to diaphragms for transfer of seismic forces. • DIAPHRAGM CONTINUITY: Steps in diaphragm levels are not reinforced for split-level irregularity. • DIAPHRAGM SPAN: Unblocked diagonally and plywood sheathed diaphragms span greater than 40 feet.
Classrooms [C, D, G]	• SPAN: Unblocked wood diaphragm spans farther than 40 feet. • LOAD PATH: Incomplete information defining in-plane and out-of-plane load path.
Gymnasium, Classrooms [H,L]	• SPAN: Diaphragms exceed allowable Tier 1 spans.

3.18.4 Recommendations

The following are rehabilitation recommendations to address the observed deficiencies. In addition, for pursuit of Life Safety compliance we recommend the additional evaluation, field investigation, and non-structural assessment as noted in Section 2.

Building Label (RVS/ZCS)	Recommendations
Administration [A]	• INFORMATION: Obtain additional/supporting design information for the completed retrofit. • DIAPHRAGM SPAN: Provide refined analysis and potentially Tier 2 evaluation to determine adequacy of unblocked diaphragms.
Classrooms [E]	• CANOPIES: Provide out-of-plane connection hardware at canopy to CMU walls.
Classrooms [F]	• INFORMATION: Obtain additional/supporting design information for the completed retrofit. • WALL ANCHORAGE: Provide out-of-plane connection hardware at top of exterior CMU walls to roof framing. • WALL ANCHORAGE: Provide out-of-plane connection hardware at interior CMU walls to roof framing. • WOOD LEDGERS: Provide out-of-plane anchorage between existing canopy and CMU

	walls. • DIAPHRAGM SPAN: Provide new blocking and fastening of plywood roof diaphragm.
Cafeteria **[I]**	• INFORMATION: Obtain additional/supporting design information for the completed retrofit. • LOAD PATH: Provide destructive investigations at roof level to confirm remaining concealed seismic hardware install. • WALL ANCHORAGE: Provide out-of-plane connection hardware at interior CMU walls to roof framing. • TRANSFER TO SHEAR WALLS: Provide new in-plane connection hardware between existing interior masonry walls and roof diaphragm. • SPAN: Provide blocking at diaphragms exceeding spans of 40 feet.
Theater **[J]**	• SHEAR STRESS CHECK: Provide new plywood sheathing at existing wood framed walls for adequate lateral capacity. • TRANSFER TO SHEAR WALLS: Provide new in-plane connection hardware at wood and CMU shear walls for transfer of seismic forces. • DIAPHRAGM CONTINUITY: Provide strengthening of diaphragms at steps in diaphragms. • DIAPHRAGM SPAN: Provide new blocking and fastening of diagonal and plywood roof diaphragms.
Classrooms **[C, D, G]**	• Provide refined analysis and supporting destructive investigation. • Provide seismic improvements as necessary potentially including additional shearwall locations and load path elements.
Gymnasium, Classrooms **[H, L]**	• Provide refined analysis and supporting destructive investigation. • Provide seismic improvements as necessary potentially including in-plane and out-of-plane connections and additional lateral load resisting elements.

3.19 South Medford High School

Figure 19: South Medford High School

3.19.1 Seismic Evaluation Summary

The following table summarizes the building type, age, and construction for structures on this campus. It also includes a summary of the seismic assessment findings.

Building Label (RVS/ZCS)	Building Type	Year Built, Remodel, and/or Retrofit	Structural Life Safety Assessment	Relative Hazard Severity
Classroom [A]	S2	2008	Compliant	L.S. Compliant
Theatre [B]	S2	2008	Compliant	L.S. Compliant
Classroom and Administration [C]	S2	2008	Compliant	L.S. Compliant
Classroom [D]	S2	2008	Compliant	L.S. Compliant
Gymnasium [E]	S2	2008	Compliant	L.S. Compliant

3.19.2 Structure Summary:

The following summarizes the building timeline and structural systems for each portion of the campus:

- **2008 Original Campus**: All buildings currently on campus were constructed as part of the bond work near 2008. The buildings are primarily framed with steel members, including open web steel joists, steel beams, and steel columns. The roof deck includes metal decking, and portions of the structures have a concrete topping slab. The main lateral force resisting system is comprised of special concentric braced frames.

3.19.3 Observed Deficiencies:

We did not observe any structural seismic deficiencies during our visual inspections and/or review of the original construction documents.

3.19.4 Recommendations

The structures were designed using a recent benchmark code (including MEP), and therefore our evaluation is that the structures meet Life Safety requirements and do not require additional steps by the District at this time.

3.20 District Admin/Maintenance Building

Figure 20: District Admin/Maintenance Building

3.20.1 Seismic Evaluation Summary

The following table summarizes the building type, age, and construction for structures. It also includes a summary of the seismic assessment findings. Please see Appendix A for a campus map summarizing the campus construction and evaluation results.

Building Label (RVS/ZCS)	Building Type	Year Built, Remodel, and/or Retrofit	Structural Life Safety Assessment	Relative Hazard Severity
Offices [A]	PC1	1959, 2011	Noncompliant	High
Warehouse [B]	PC1/RM1	1960, Est. 1986, 2011	Noncompliant	High
Administration & Shop [C]	PC1	1958, Est. 1985, 2011	Noncompliant	High

3.20.2 Structure Summary

The District Admin/Maintenance Building consists of three structures built at different times adjacent to Central High School. The following summarizes the building timeline and structural systems for each portion of the District Admin/Maintenance Building:

- **1958 Original Shop**: The original building was the Medford High School Shop and now houses the current Facilities Office and Shop. This is a one-story structure with a flexible wood roof diaphragm with concrete and CMU walls. The roof consists of plywood sheathing over timber joists bearing on glulam beams which bear on columns and exterior walls. The walls consist of concrete tilt-up panels on three sides and CMU with glazing on the South side. The foundation consists of slab-on-grade with concrete stem walls. The original structure has an approximate footprint of 15,000 square feet.
- **1959 Music Room Addition**: This addition houses the current IT department and offices. This building consists of concrete walls with a flexible wood roof diaphragm, and interior CMU and wood framed walls. The roof consists of plywood sheathing over timber roof joists bearing on glulam beams and exterior walls. The walls consist of concrete tilt-up panels at the exterior with concrete pilasters with CMU and wood framed walls at the interior. The foundation consists of slab-on-grade with concrete stem walls. The original structure has an approximate footprint of 8,800 square feet.
- **1960 Gym Addition**: This addition houses the current Warehouse and consists of concrete walls with a flexible wood roof diaphragm, areas of cast-in-place concrete second floors, and interior wood framed walls. The roof consists of T&G decking over glulam purlins hung from large glulam beams bearing on existing concrete pilasters of the existing Music Room and Shop Buildings. The walls consist of reinforced concrete extended from the top of the existing Shop and Music Building walls. The second floor at the East and West ends of the structure consists of reinforced concrete slabs on steel joists and beams. The original structure has an approximate footprint of 15,800 square feet.
- **2011 Various Improvements**: In 2011 the District Admin/Maintenance Building received various upgrades including some structural work. The second-floor slab and supporting steel joists in the East section of the Warehouse (Gym) building were removed. The East walls supporting the removed second floor slab of the Warehouse received new openings with new CMU infill and steel strong backs for support. New overhead doors were installed in the openings and a new canopy was added. The remainder of the Annex received new partition walls, doors, and windows for additional office space. The Annex received HVAC, plumbing, and electrical upgrades as part of the school bond.

3.20.3 Observed Deficiencies:

The following list summarizes the structural seismic deficiencies observed during our visual inspections and/or original construction documents:

Building Label (RVS/ZCS)	Deficiency
Original Shop [C]	• LOAD PATH: Inadequate in-plane shear capacity at South wall; large number of windows. • LOAD PATH: Wood ledgers exist which induce cross grain shear. • WALL ANCHORAGE: Areas lack out-of-plane connection to top of concrete walls
1959 Addition [A]	• LOAD PATH: Wood ledgers exist which induce cross grain shear. • WALL ANCHORAGE: Areas lack out-of-plane connection to top of concrete walls.
1960 Addition [B]	• LOAD PATH: Wood ledgers exist which induce cross grain shear. • WALL ANCHORAGE: Areas lack out-of-plane connection to top of concrete walls.

3.20.4 Recommendations

The following are rehabilitation recommendations to address the observed deficiencies. In addition, for pursuit of compliance with Life Safety or better designation we recommend the additional evaluation, field investigation, and non-structural assessment as noted in Section 2. It has been noted by District personnel that it is a District goal to retrofit of this facility to obtain Immediate Occupancy performance.

Building Label (RVS/ZCS)	Recommendations
Original 1958 Shop [C]	• Provide new out-of-plane connections at the top of concrete and masonry walls. • Provide new in-plane shear walls at South window wall by infill framing existing windows at strategic locations.
1959 Music Room Addition [A]	• Provide new out-of-plane connections at the top of concrete walls.
1960 Gym Addition [B]	• Provide new out-of-plane connections at the top of concrete walls.

3.21 Distribution Center Building

Figure 21: Distribution Center Building

3.21.1 Seismic Evaluation Summary

The following table summarizes the building type, age, and construction for structures on this campus. It also includes a summary of the seismic assessment findings. Please see Appendix A for a campus map summarizing the campus construction and evaluation results.

Building Label (RVS/ZCS)	Building Type	Year Built, Remodel, and/or Retrofit	Structural Life Safety Assessment	Relative Hazard Severity
Warehouse [A]	C2a	1959, Est. 2000	Noncompliant	Moderate

3.21.2 Structure Summary

The following summarizes the building timeline and structural systems for each portion of the District Distribution Center Building:

- **Original 1960's Construction:** The Distribution Center Building is a one-story structure consisting of concrete exterior walls with masonry and timber framed interior walls and a flexible wood roof diaphragm.

- **Original 1960's Construction, Continued**: The roof consists of deep glulam beams bearing on concrete pilasters at the exterior walls and concrete pilasters at the interior CMU walls. T&G decking bears on the glulam beams, the span on the glulam beams is approximately 60 feet with a spacing of approximately 16 feet between beams.

3.21.3 Observed Deficiencies:

The following list summarizes the structural seismic deficiencies observed during our visual inspections and/or original construction documents:

Building Label (RVS/ZCS)	Deficiency
1959 Original [A]	• LOAD PATH: Inadequate in-plane shear capacity at infill framing between top of concrete / masonry walls to roof diaphragm. • WALL ANCHORAGE: Areas lack out-of-plane connections at top of concrete and masonry walls.

3.21.4 Recommendations

The following are rehabilitation recommendations to address the observed deficiencies. In addition, for pursuit of Life Safety compliance we recommend the additional evaluation, field investigation, and non-structural assessment as noted in Section 2. Alternate repair strategies may be presented.

The Distribution Center lacks an adequate load path for in-plane shear at the infill framing between the top of concrete and masonry walls and inadequate connections for transfer of out-of-plane seismic forces. Recommendations for the Distribution Center include:

Building Label (RVS/ZCS)	Recommendations
1959 Original [A]	• Provide new out-of-plane connections at the top of concrete and masonry walls. • Provide new in-plane shear walls at existing infill framing. • Provide destructive investigation.

4 Building Condition Summary

The following section summarizes the building deficiency information presented above for each of the schools and facilities reviewed in Section 3.0. Each school and facility were ranked as either a high, moderate, or low relative hazard or Life Safety Compliant based on the number and degree of deficiencies present and a review of applicable Life Safety provisions. A table is provided listing the relative hazard severity at each of the considered schools and support facilities.

4.1 Building Deficiencies Discussion

Throughout the inspection process there were three observable types of deficiencies. High priority deficiencies were generally considered to increase the likelihood of structural failure and collapse during a seismic event. Low priority deficiencies were considered to be items that result in the building being less equipped to handle the effects of seismic events but would not lead to structural collapse without other deficiencies present. Low priority deficiencies will still damage a structure during a seismic event, but they generally will not result in structural failure alone. In addition to the observed deficiencies, it is believed that unseen deficiencies such as the following may be present in many of the schools based on the age of construction and associated construction techniques:

- Roof-to-wall and floor-to-wall connections
- Wall-to-foundation attachments
- Capacity of shear walls
- Capacity of diaphragms
- Seismic bracing for conduits, ductwork, HVAC, and other non-structural items

4.2 Observed Deficiency Ranking, High Priority Retrofits

After assembling a list of deficiencies in Section 3.0, the table below was created to illustrate the results of this study and identify the schools and facilities with the highest level of concern. High priority deficiencies are those that have the highest risk of collapse potential and include items such as unreinforced masonry walls and a lack of lateral load path to the foundation. Low priority deficiencies typically involve structures with redundant structural systems and light-weight buildings. Low priority deficiencies can also include more modern construction that contains detailing close to current standards but does not meet benchmark codes.

The building inspections performed for this report were limited to visual observations and review of available construction documents only. As such, the deficiencies listed above are not expected to be all-encompassing. Previous seismic investigations and knowledge of construction methods during the eras in which the structures were built have allowed us to consider expected deficiencies that were unobservable given the scope of our investigation. These deficiencies are common, and their inclusion is useful in ranking and determining a rough cost for improvements at each school.

The following table illustrates an aggregate hazard level for each campus. Campuses are arranged in order of severity (and secondly alphabetically). It should be noted that ZCS recommends pursuing retrofit opportunities in order of campus-wide severity and also upon building specific severity as can be identified in Appendix A.

School	**Structural Life Safety Compliant**	**Relative Hazard Severity[1]**
Jacksonville Elementary School[5]	Noncompliant	High
District Admin/Maintenance Building[3]	Noncompliant	High
Griffin Creek Elementary School[2]	Noncompliant	Moderate
Howard Elementary School[2,5]	Noncompliant	Moderate
Kennedy Elementary School[5]	Noncompliant	Moderate
Oak Grove Elementary School[2]	Noncompliant	Moderate
Ruch Community School[2]	Noncompliant	Moderate
Hedrick Middle School[5]	Noncompliant	Moderate
Oakdale Middle School (Old Central High School)[6]	Noncompliant	Moderate
North Medford High School[2, 5]	Noncompliant	Moderate
Washington Elementary School[2]	Noncompliant	Moderate
Distribution Center Building[3]	Noncompliant	Moderate
Wilson Elementary School	Noncompliant	Moderate
Abraham Lincoln Elementary School	Noncompliant	Low
Jackson Elementary School[2]	Noncompliant	Low
Jefferson Elementary School[2]	Noncompliant	Low
Lone Pine Elementary School[2]	Noncompliant	Low
Roosevelt Elementary School[2]	Noncompliant	Low
Mcloughlin Middle School	Noncompliant	Low
Hoover Elementary School	Compliant	L.S. Complaint
South Medford High School	Compliant	L.S. Compliant

1. Relative Hazard Severity level has been provided based upon an aggregation of the hazard risks across the campus. Portions of the campus may not share this hazard severity, and building-specific scores have been provided in the school specific sections.
2. These campuses have some buildings or structures that meet the compliance criteria as defined in this report.
3. The Admin/Maintenance Facility and Distribution Center are not eligible for a Seismic Retrofit Grant Application (SRG). At this time, SRG applications are limited to educational and emergency services buildings and do not include ancillary or supporting service buildings.
4. At time of writing this campus is undergoing a seismic retrofit project that has attained substantial completion. The anticipated effects of this project are represented in the current assessment summary, reflected in Section 3.0, and in the recommended prioritization list.
5. This campus or a portion thereof has applied for a Seismic Rehabilitation Grant from the IFA.
6. At time of writing this campus is undergoing a renovation and seismic retrofit project. The anticipated effects of this project are represented in the current assessment summary, reflected in Section 3.0, and in the recommended prioritization list.

4.3 Low Cost Opportunities

In addition to addressing the more sever seismic hazards, there are lower-level efforts than can be taken to confirm or potentially upgrade a building's designation. For buildings ranked low, we recommend pursuing the following steps:

- For buildings retrofit as part of the 2008 bond projects, contact the Engineers of Records to confirm the level of seismic retrofit completed and obtain corresponding documentation. If the retrofit design was limited to hazard reduction, the designation of Low would likely remain. If, however, it was designed to meet Life Safety requirements or beyond, the designation would improve.
- For structures with concealed seismic scope install, provide destructive demolition of noted roofs, ceilings, and walls to confirm remaining concealed hardware. Confirmation of concealed seismic scope items may improve the buildings hazard designation.
- For buildings noted to have limited deficiencies, such as diaphragms which exceed span limits, provide additional analysis including Tier 2 analysis to determine performance of structural systems noted to be deficient per Tier 1 checklists. If found to be satisfactory through further analysis these deficiencies may be removed and the building designation improved to Life Safety compliant.
- For wood framed benchmark buildings, provide additional field inspection, destructive investigation, and/or additional analysis.

For structures that retain Low designation after the above steps, our recommendation would be to pursue a seismic retrofit design that may be incorporated into future capital improvement projects. For these structures, a retrofit may be relatively inexpensive to incorporate into a re-roofing project or re-siding project.

The following tables summarize information from Section 3 indicating our recommendations for current low-cost opportunities. There are many potential low-cost opportunities throughout the District and therefore this list is not exhaustive; we recommend tracking progress of these opportunities and continuing to develop this list as items are addressed.

Campus	Building Label (RVS/ZCS)	Low Cost Opportunity
Jackson Elementary School	[C, D, G, F]	• Provide additional field and foundation investigation. • Provide destructive investigation, and Tier 2 Analysis to develop requirements to address noted deficiencies.
Lone Pine Elementary School	[C, Southern Portion]	• Provide destructive investigation, review of non-structural elements, and Tier 2 Analysis to develop

		retrofit plan.
	[G, F]	• Provide additional field and foundation investigation.
Roosevelt Elementary School	[B]	• Provide destructive investigation, and Tier 2 Analysis to develop requirements to address noted deficiencies.
	[D]	• Provide destructive investigation, review of non-structural elements, and Tier 2 Analysis.
Wilson Elementary School	[A through H]	• Provide destructive investigations at roof levels to confirm remaining seismic retrofit scope install. • Provide additional geotechnical investigations to determine extents of liquefaction potential.
	[I]	• Provide additional field and foundation investigation. • Provide additional geotechnical investigations to determine extents of liquefaction potential.
North Medford High School	[A]	• Provide refined analysis and Tier 2 analysis of existing unblocked diaphragms to determine adequacy.
	[E]	• Provide out of plane connection hardware between existing canopy and CMU walls.

4.4 Seismic Rehabilitation Grant Applications

Senate Bills 4 and 5 in Oregon authorized the state to issue bonds as the funding mechanism for a program to fund the seismic rehabilitation of schools and emergency services buildings. The table below summarizes applications that have been submitted to the state previously but did not receive grant awards. While not successfully funded, they are available for resubmittal for future rounds of funding. Funding amounts and timelines are determined by the state legislature.

Campus	Year of submittal	Buildings Included in Application
Jacksonville Elementary School: Phase 1	2016, 1/2018, 11/2018	D

Jacksonville Elementary School: Phase 2	2018	A (partial), B
Jacksonville Elementary School: Phase 3	2020	A, B, C, D (Partial)
Howard Elementary School	2018 2020	A A
Kennedy Elementary School	2018 2020	B A, B, C, D, E, F, K
Hedrick Middle School	2018	B (partial)
Griffin Creek Elementary School	2019	C, D, E, F

5 Conclusion

The findings described in this report have been limited to the seismic lateral force resisting structural systems present at each facility and were the result of visual observations and/or review of available construction documents. The deficiencies noted are not intended to be exhaustive and hazard ratings are subject to change pending the availability of additional information.

From our review, we find the school structures to be in good condition and generally safe for occupancy. Given the current condition of the structures, the code governing existing buildings does not mandate immediate upgrades unless the building is scheduled for repairs, alterations, additions, or a change in occupancy. However, to comply with ORS 455.400 and increase student safety, we recommend generating a priority list for capital projects to systematically address deficiencies as funds become available. Additionally, incremental updates should be considered during projects that may make performing the work easier. Concurrent with addressing relatively severe seismic hazards, we also recommend pursuing lower cost opportunities such as increasing a Low designation to Life Safety. The recommended improvements listed in this report reflect items that do not pose a substantial immediate risk to the life safety of occupants (unless noted otherwise) outside of code lateral events. It should be noted that structural deficiencies in schools of this age group are fully expected and the severity of the deficiencies noted above are common.

Attention should be paid to the potential for upcoming seismic retrofit grant programs. Several of the schools noted above are good candidates for programs that can fund some or all of the expenses related to seismic retrofit of school buildings.

Please contact our office if you would like to discuss our findings.

Appendix A: Campus Naming Key and Summary

ZCS ENGINEERING ARCHITECTURE

School Name: Abraham Lincoln Elementary School
School District: Medford School District

Building Type: School	
Street Address: 3101 McLoughlin Dr.	
City: Medford	
Latitude: 42.363014	
Longitude: -122.836929	
Date: 1/29/18	

RVS and District Wide Assessment Naming Convention:

School Name: Griffin Creek Elementary School
School District: Medford School District

Building Type: School	
Street Address: 2430 Griffin Creek Rd.	
City: Medford	
Latitude: 42.294337	
Longitude: -122.907480	
Date: 01/28/19	

RVS and District Wide Assessment Naming Convention:

ZCS ENGINEERING ARCHITECTURE

School Name: Hoover Elementary School
School District: Medford School District

Building Type: School	
Street Address: 2323 Siskiyou Blvd.	
City: Medford	
Latitude: 42.320630	
Longitude: -122.838840	
Date: 01/29/19	

RVS and District Wide Assessment Naming Convention:

ZCS ENGINEERING ARCHITECTURE

School Name: Howard Elementary School
School District: Medford School District

Building Type: School	
Street Address: 286 Mace Rd.	
City: Medford	
Latitude: 42.357684	
Longitude: -122.890960	
Date: 01/29/19	

RVS and District Wide Assessment Naming Convention:

School Name: Jackson Elementary School
School District: Medford School District

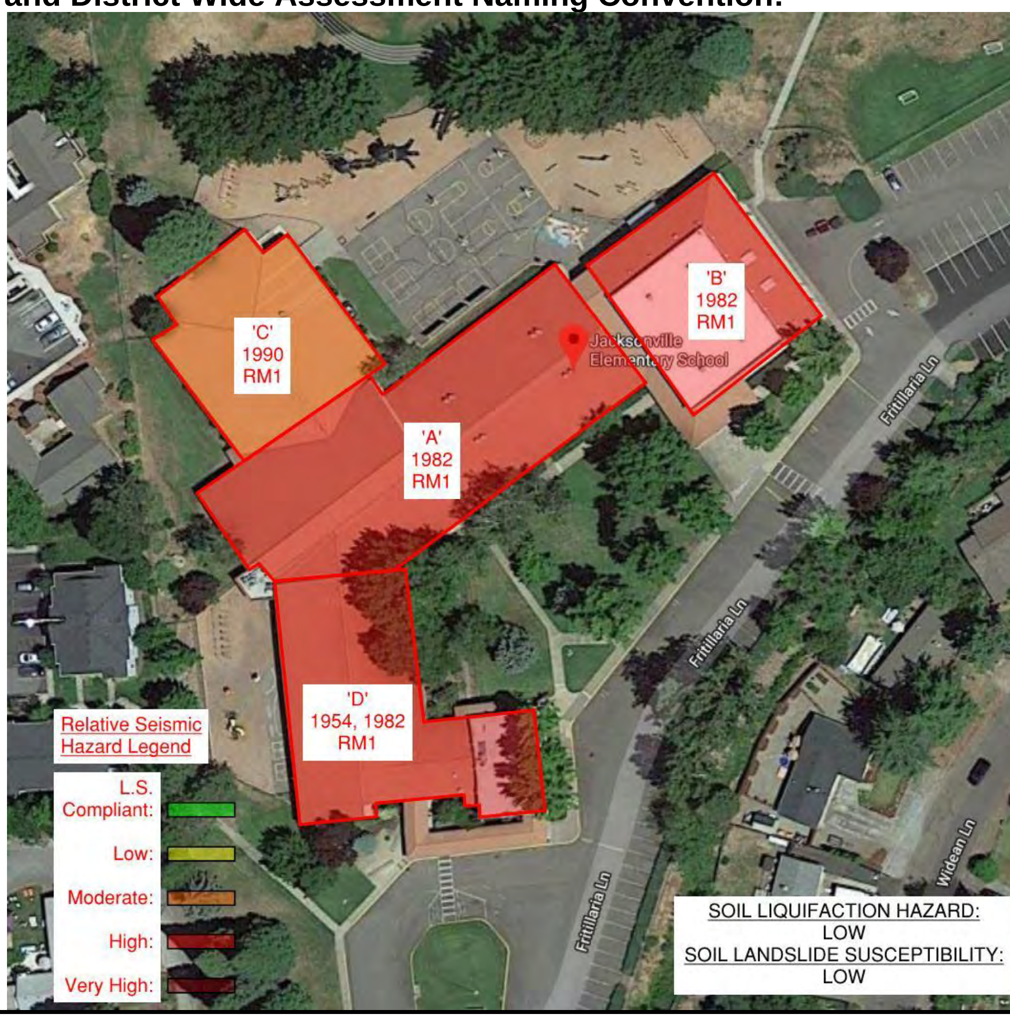
Building Type: School	
Street Address: 630 W. Jackson St	
City: Medford	
Latitude: 42.33138	
Longitude: 122.88723	
Date: 01/25/19	

RVS and District Wide Assessment Naming Convention:

School Name: Jacksonville Elementary School
School District: Medford School District

Building Type: School	
Street Address: 655 Hueners Ln.	
City: Medford	
Latitude: 42.317776	
Longitude: -122.957920	
Date: 01/29/19	

RVS and District Wide Assessment Naming Convention:



School Name: Jefferson Elementary School
School District: Medford School District

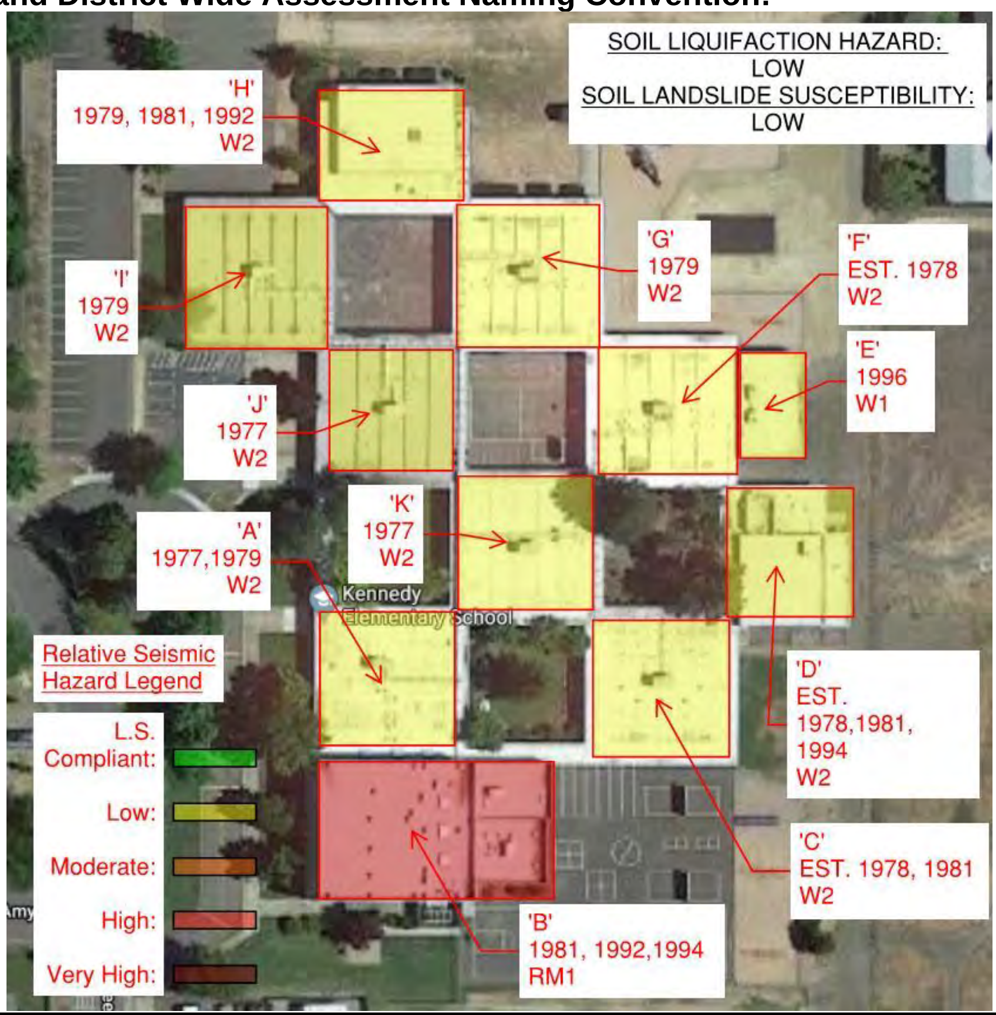
Building Type: School	
Street Address: 333 Holmes Ave.	
City: Medford	
Latitude: 42.308698	
Longitude: -122.868848	
Date: 01/28/19	

RVS and District Wide Assessment Naming Convention:

School Name: Kennedy Elementary School
School District: Medford School District

Building Type: School	
Street Address: 2860 N. Keene Way Dr.	
City: Medford	
Latitude: 42.358562	
Longitude: -122.851732	
Date: 01/25/19	

RVS and District Wide Assessment Naming Convention:



School Name: Lone Pine Elementary School
School District: Medford School District

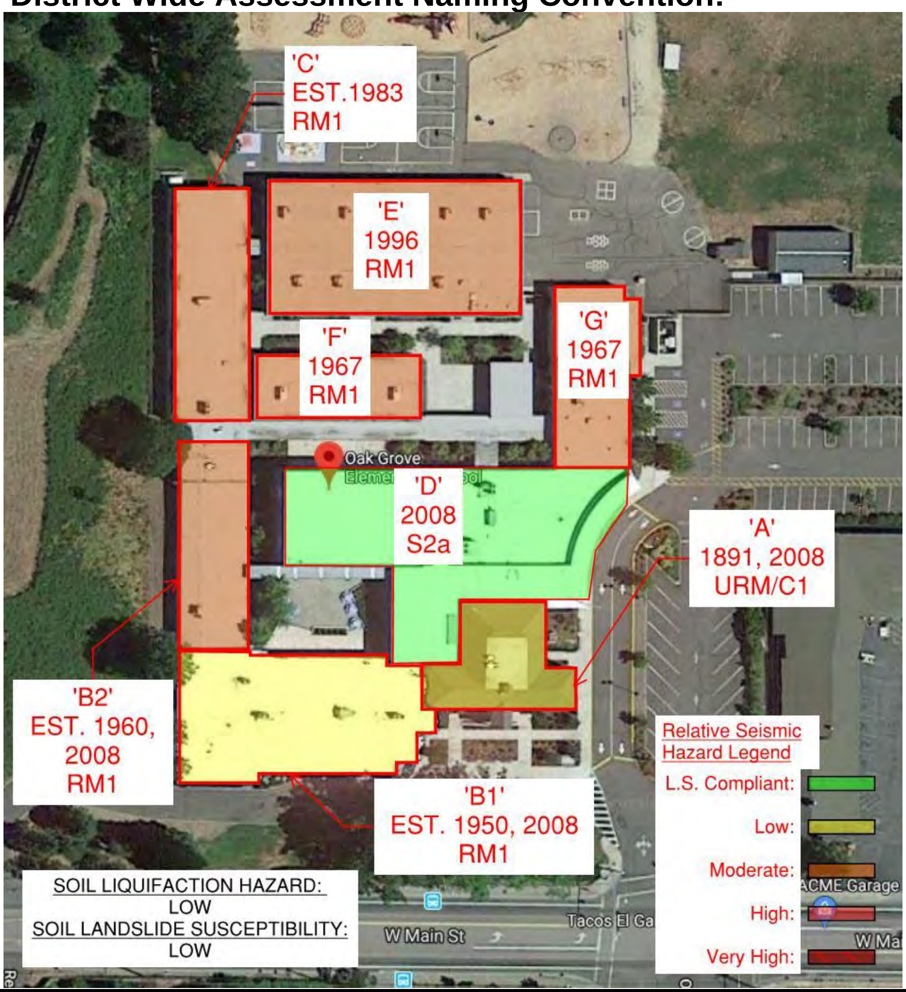
Building Type: School	
Street Address: 3158 Lone Pine Rd.	
City: Medford	
Latitude: 42.345016	
Longitude: -122.831943	
Date: 01/25/19	

RVS and District Wide Assessment Naming Convention:

School Name: Oak Grove Elementary School
School District: Medford School District

Building Type: School	
Street Address: 2838 W. Main St.	
City: Medford	
Latitude: 42.324514	
Longitude: -122.909560	
Date: 01/28/19	

RVS and District Wide Assessment Naming Convention:



School Name: Roosevelt Elementary School
School District: Medford School District

Building Type:
School

Street Address:
1212 Queen Anne Ave.

City:
Medford

Latitude:
42.329925

Longitude:
-122.857287

Date:
01/25/19

RVS and District Wide Assessment Naming Convention:

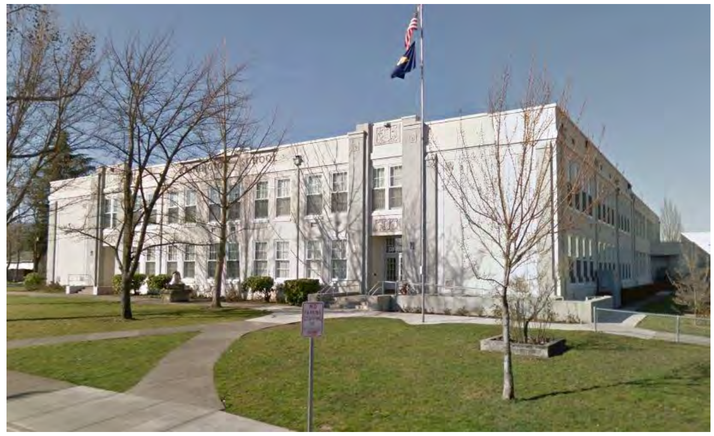
School Name: Ruch Community School
School District: Medford School District

Building Type: School	
Street Address: 156 Upper Applegate Rd.	
City: Jacksonville	
Latitude: 42.235907	
Longitude: -122.044389	
Date: 01/28/19	

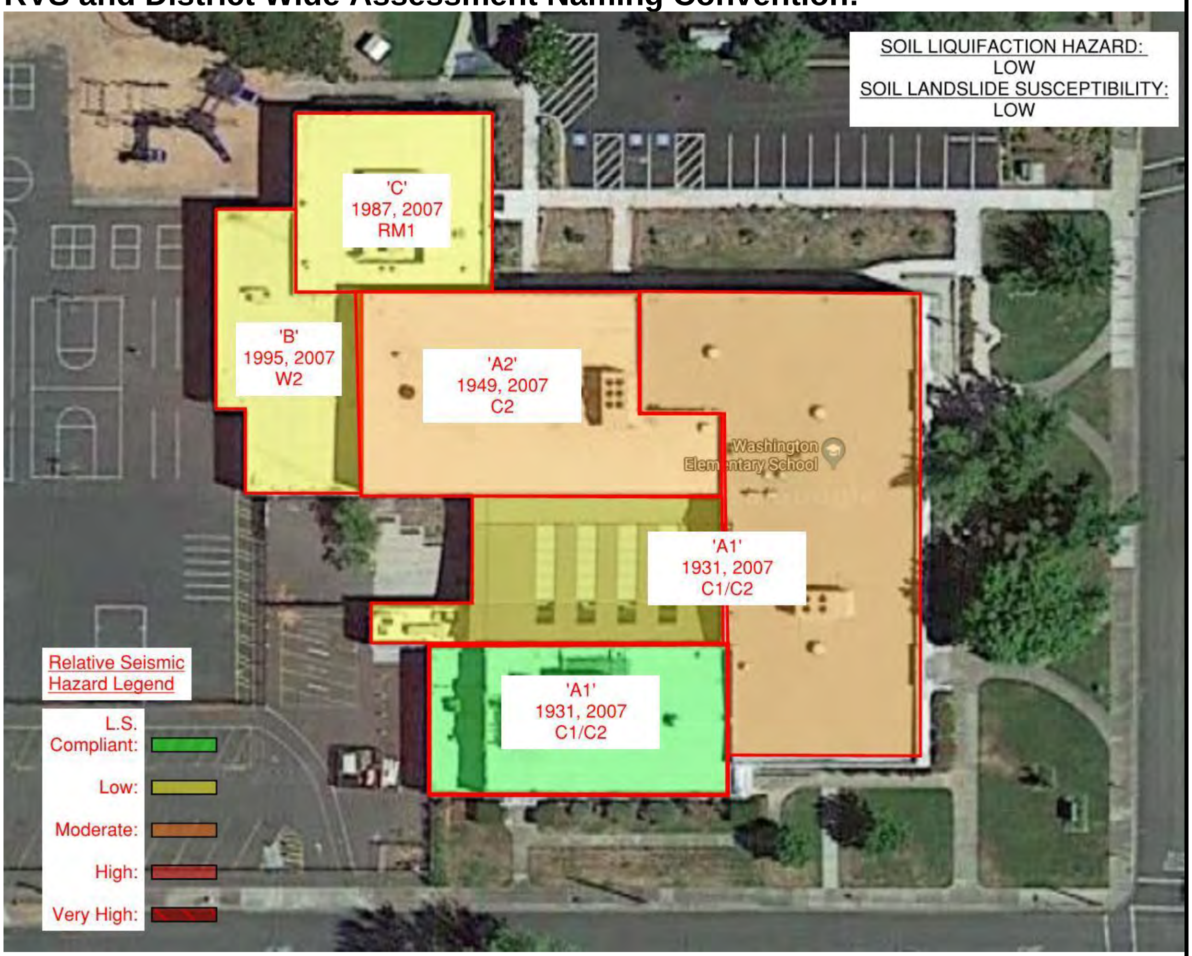
RVS and District Wide Assessment Naming Convention:

ZCS ENGINEERING ARCHITECTURE

School Name: Washington Elementary School
School District: Medford School District

Building Type: School	
Street Address: 610 S. Peach St.	
City: Medford	
Latitude: 42.317013	
Longitude: -122.883029	
Date: 01/28/19	

RVS and District Wide Assessment Naming Convention:



ZCS ENGINEERING ARCHITECTURE

School Name: Wilson Elementary School
School District: Medford School District

Building Type: School	
Street Address: 1400 Johnson St.	
City: Medford	
Latitude: 42.345529	
Longitude: -122.866350	
Date: 01/28/19	

RVS and District Wide Assessment Naming Convention:

School Name: Hedrick Middle School
School District: Medford School District

Building Type: School	
Street Address: 1501 E. Jackson St.	
City: Medford	
Latitude: 42.331838	
Longitude: -122.853064	
Date: 01/25/19	

RVS and District Wide Assessment Naming Convention:

School Name: McLoughlin Middle School
School District: Medford School District

Building Type:
School

Street Address:
320 W. 2nd St.

City:
Medford

Latitude:
42.329159

Longitude:
-122.880897

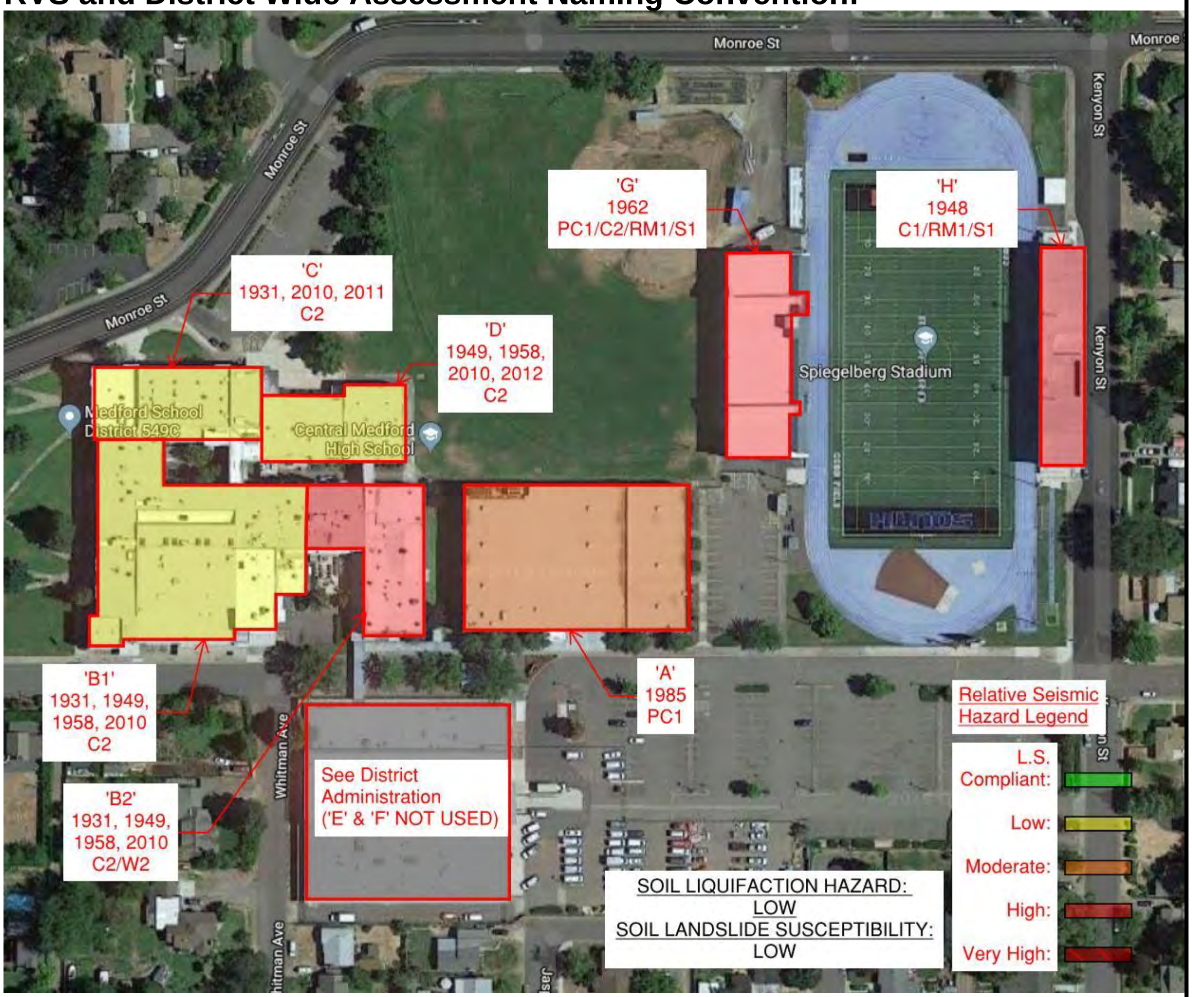
Date:
01/28/19

RVS and District Wide Assessment Naming Convention:

School Name: Oakdale Middle School (Old Central Medford High School)
School District: Medford School District

Building Type: School	
Street Address: 815 S. Oakdale Ave.	
City: Medford	
Latitude: 42.316372	
Longitude: -122.872157	
Date: 01/28/19	

RVS and District Wide Assessment Naming Convention:



School Name: North Medford High School
School District: Medford School District

Building Type:
School

Street Address:
1900 N. Keene Way Dr.

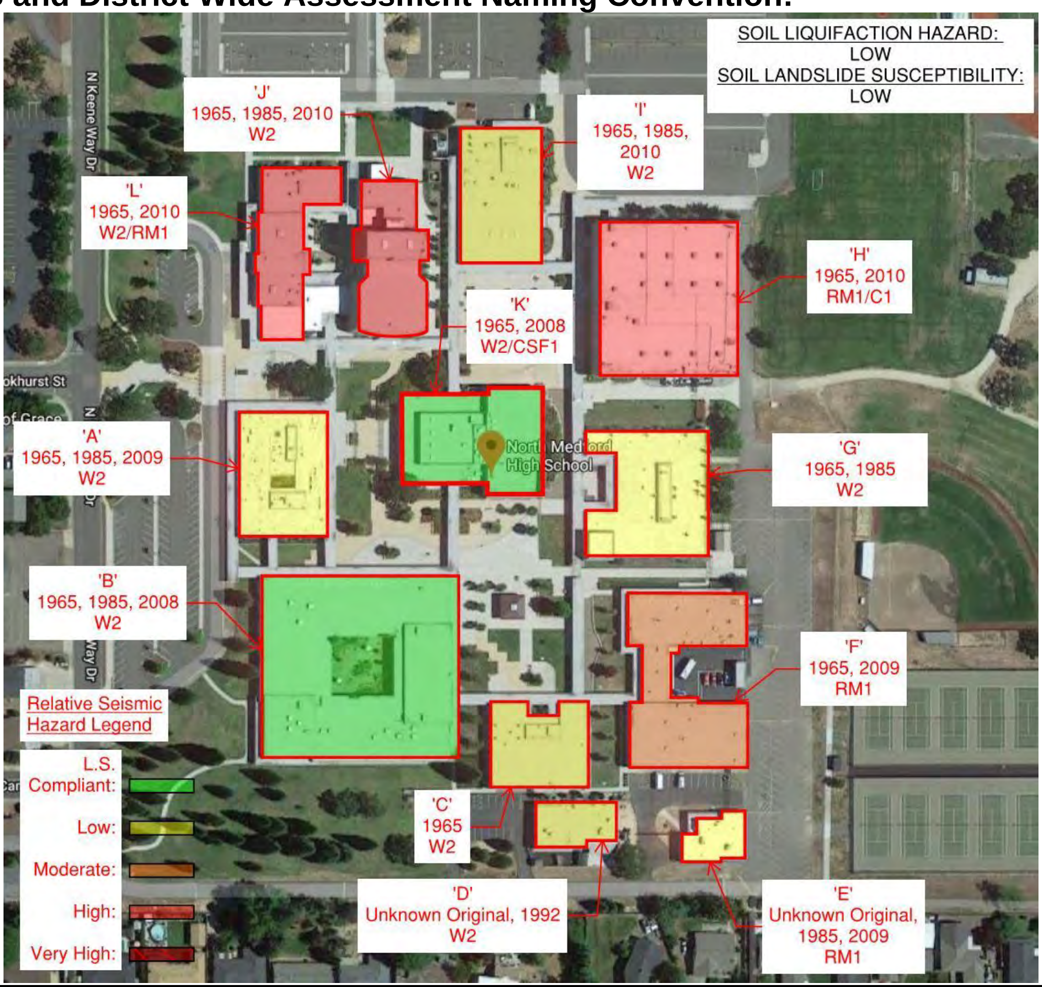
City:
Medford

Latitude:
42.347189

Longitude:
-122.850166

Date:
01/28/19

RVS and District Wide Assessment Naming Convention:



School Name: South Medford High School
School District: Medford School District

Building Type: School	
Street Address: 1551 Cunningham Ave.	
City: Medford	
Latitude: 42.304630	
Longitude: **-122.893061**	
Date: 01/28/19	

RVS and District Wide Assessment Naming Convention:

ZCS ENGINEERING ARCHITECTURE

School Name: Medford School District 549C Administration
School District: Medford School District

Building Type: Admin. Facility	
Street Address: 815 S. Oakdale Ave.	
City: Medford	
Latitude: 42.315379	
Longitude: -122.872389	
Date: 01/29/19	

RVS and District Wide Assessment Naming Convention:

ZCS ENGINEERING ARCHITECTURE

School Name: Distribution Center
School District: Medford School District

Building Type: Maint/Storage	
Street Address: 750 N. Columbus Ave.	
City: Medford	
Latitude: 42.331718	
Longitude: -122.889751	
Date: 01/30/19	

RVS and District Wide Assessment Naming Convention:

Appendix B: Building Type Definitions

Table 3-1. Common Building Types

Wood Light Frames	
W1	These buildings are single- or multiple-family dwellings one or more stories high with plan areas less than or equal to 3,000 ft^2 (280 m^2). Building loads are light, and the framing spans are short. Floor and roof framing consists of wood joists or rafters on wood studs spaced no more than 24 in. (61 cm) apart. The first-floor framing is supported directly on the foundation system or is raised up on cripple studs and post-and-beam supports. The foundation is permitted to consist of a variety of elements. Chimneys, where present, consist of solid brick masonry, masonry veneer, or wood frame with internal metal flues. Seismic forces are resisted by wood frame diaphragms and shear walls. Floor and roof diaphragms consist of straight or diagonal lumber sheathing, tongue-and-groove planks, oriented strand board, plywood, or other materials. Shear walls are permitted to consist of straight or lumber sheathing, plank siding, oriented strand board, plywood, stucco, gypsum board, particleboard, fiberboard, or similarly performing materials. Interior partitions are sheathed from floor to floor with plaster or gypsum board. Older construction often has open-front garages at the lowest story and is permitted to be split-level.
W1a (Multistory, Multiunit, Residential)	These buildings are multistory, similar in construction to W1 buildings, but have plan areas on each floor of more than 3,000 ft^2 (280 m^2). Older construction often has open-front garages at the lowest story.
Wood Frames, Commercial and Industrial	
W2	These buildings are commercial or industrial buildings with a floor area of 5,000 ft^2 (465 m^2) or more. There are few, if any, interior walls. The floor and roof framing consists of wood or steel trusses, glulam or steel beams, and wood posts or steel columns. The foundation system is permitted to consist of a variety of elements. Seismic forces are resisted by flexible diaphragms and exterior walls sheathed with plywood, oriented strand board, stucco, plaster, or straight or diagonal wood sheathing, or they are permitted to be braced with various forms of bracing. Wall openings for storefronts and garages, where present, are framed by post-and-beam framing.
Steel Moment Frames	
S1 (with Stiff Diaphragms)	These buildings consist of a frame assembly of steel beams and steel columns. Floor and roof framing is stiff, including cast-in-place concrete slabs or metal deck with concrete fill supported on steel beams, open web joists, or steel trusses. Seismic forces are resisted by steel moment frames that develop their stiffness through rigid or semirigid beam–column connections. Where all connections are moment-resisting connections, the entire frame participates in seismic force resistance. Where only selected connections are moment-resisting connections, resistance is provided along discrete frame lines. Columns are oriented so that each principal direction of the building has columns resisting forces in strong axis bending. Diaphragms consist of rigid construction that is stiff relative to the frames. The exterior of the structure is permitted to be concealed; the environmental closure walls consist of any type, including both ductile, flexible systems, and rigid, nonductile systems (e.g., unreinforced masonry either interior or exterior to the frame line). Where the interior of the structure is finished, frames are concealed by ceilings, partition walls, and architectural column furring. The foundation system is permitted to consist of a variety of elements.
S1a (with Flexible Diaphragms)	These buildings are similar to S1 buildings, except that diaphragms are untopped metal deck or metal deck with lightweight insulating concrete, poured gypsum, wood, or similar nonstructural topping and are flexible relative to the frames. Support for the diaphragm is permitted to be solid elements or truss members made of wood and/or metal.
Steel Braced Frames	
S2 (with Stiff Diaphragms)	These buildings have a frame of steel columns, beams, and braces. Braced frames develop resistance to seismic forces by the bracing action of the diagonal members. The braces induce forces in the associated beams and columns such that all elements work together in a manner similar to a truss; all element stresses are primarily axial. Diaphragms transfer seismic loads to braced frames. The diaphragms consist of concrete or metal deck with concrete fill and are stiff relative to the frames. The foundation system is permitted to consist of a variety of elements. Three variations in the configuration and design of braced frames exist. These variations are • Concentrically braced frames: Component work lines intersect at a single point or at multiple points such that the distance between intersecting work lines (or eccentricity) is less than or equal to the width of the smallest component connected at the joint.

continues

Table 3-1 *(Continued).* Common Building Types

	• Eccentrically braced frames: Component work lines do not intersect at a single point, and the distance between the intersecting work lines (or eccentricity) exceeds the width of the smallest component connecting at the joint. Some of the members are subjected to shear and flexural stresses because of that eccentricity. • Buckling-restrained braced frames: Special types of concentrically braced frames where the steel bracing members are encased within a rigid casing that is intended to prevent buckling of the steel brace.
S2a (with Flexible Diaphragms)	These buildings are similar to S2 buildings, except that diaphragms consist of wood or cold-framed steel framing; untopped metal deck; or metal deck with lightweight insulating concrete, poured gypsum, or similar nonstructural topping, and are flexible relative to the frames.
Metal Building Frames	
S3	These buildings use transverse steel moment frames. They are one story high, but they sometimes have mezzanines. The roof and walls consist of lightweight metal, fiberglass, or cementitious panels. The frames are designed for maximum efficiency, and the beams and columns are permitted to consist of either web-tapered or prismatic built-up sections with thin plates. The frames are built in segments and assembled in the field with bolted or welded joints. Seismic forces in the transverse direction are resisted by the moment frames. Seismic forces in the longitudinal direction are resisted by wall panel shear elements or rod bracing. Diaphragm forces are resisted by untopped metal deck, roof panel shear elements, or a system of tension-only rod bracing. The foundation system is permitted to consist of a variety of elements.
Dual Frame Systems with Backup Steel Moment Frames and Stiff Diaphragms	
S4	These buildings consist of a frame assembly of steel beams and steel columns. The floor and roof diaphragms consist of cast-in-place concrete slabs or metal deck with or without concrete fill. Framing consists of steel beams, open web joists, or steel trusses. Seismic forces are resisted primarily by either steel braced frames or constructed-in-place shear walls in combination with backup steel moment frames. These walls are bearing walls where the steel frame does not provide a complete vertical support system. The steel moment frames are designed to work together with the steel braced frames or concrete shear walls in proportion to their relative rigidity. The steel moment frames provide a secondary seismic-force-resisting system based on the stiffness of the frame and the moment capacity of the beam–column connections. Such moment frames were typically designed to be capable of resisting 25% of the building's seismic forces. The foundation system is permitted to consist of a variety of elements.
Steel Frames with Infill Masonry Shear Walls	
S5 (with Stiff Diaphragms)	This is an older type of building construction that consists of a frame assembly of steel beams and steel columns. The floor and roof diaphragms consist of cast-in-place concrete slabs or metal deck with concrete fill and are stiff relative to the walls. Framing consists of steel beams, open web joists, or steel trusses. Walls consist of infill panels constructed of solid clay brick, concrete block, or hollow clay tile masonry. Infill walls are permitted to completely encase the frame members and present a smooth masonry exterior with no indication of the frame. The seismic performance of this type of construction depends on the interaction between the frame and infill panels. The combined behavior is more like a shear wall structure than a frame structure. Solidly infilled masonry panels form diagonal compression struts between the intersections of the frame members. If the walls are offset from the frame and do not fully engage the frame members, diagonal compression struts do not develop. The strength of the infill panel is limited by the shear capacity of the masonry bed joint or the compression capacity of the strut. The postcracking strength is determined by an analysis of a moment frame that is partially restrained by the cracked infill. The foundation system is permitted to consist of a variety of elements.
S5a (with Flexible Diaphragms)	These buildings are similar to S5 buildings, except that diaphragms consist of wood sheathing or untopped metal deck, or the diaphragms have large aspect ratios and are flexible relative to the walls.

continues

Table 3-1 ***(Continued).*** **Common Building Types**

Steel Plate Shear Walls	
S6	These buildings have a frame of steel columns, beams, and shear walls. Shear walls are constructed with steel plates with horizontal and vertical boundary elements adjacent to the webs. The boundary elements are designed to remain essentially elastic under maximum forces that can be generated by the fully yielded webs. Diaphragms transfer seismic forces to braced frames. The diaphragms consist of concrete or metal deck with concrete fill and are stiff relative to the shear walls. The foundation system is permitted to consist of a variety of elements.
Cold-Formed Steel Light-Frame Construction	
CFS1 (Shear Wall System)	These buildings have cold-formed steel light-frame walls supporting the majority of the lateral loads. Floor and roof framing consists of cold-formed steel joists or rafters on cold-formed steel studs spaced no more than 24 in. (61 cm) apart, wood or cold-formed steel trusses, structural steel or cold-formed steel beams, and structural steel or cold-formed steel columns. The first-floor framing is supported directly on the foundation system or is raised up on cripple studs and post-and-beam supports. The foundation is permitted to consist of a variety of elements. Chimneys, where present, consist of solid brick masonry, masonry veneer, or cold-formed steel frame with internal metal flues. Seismic forces are resisted by wood structural panel or metal deck diaphragms, and wood structural panel sheathed shear walls or steel sheet sheathed shear walls. Floor and roof sheathing consists of wood structural panels or metal deck. Interior surfaces are sheathed with plaster or gypsum board. Buildings of this type that have precast concrete plank diaphragms shall not be permitted to be classified as this common building type and shall not be permitted to be evaluated using Tier 1 or Tier 2 procedures.
CFS2 (Strap-Braced Wall System)	These buildings have cold-formed steel light-frame strap walls supporting the majority of the lateral loads. Floor and roof framing consists of cold-formed steel joists or rafters on cold-formed steel studs spaced no more than 24 in. (61 cm) apart, wood or cold-formed steel trusses, structural steel or cold-formed steel beams, and structural steel or cold-formed steel columns. The first-floor framing is supported directly on the foundation system or is raised up on cripple studs and post-and-beam supports. The foundation is permitted to consist of a variety of elements. Chimneys, where present, consist of solid brick masonry, masonry veneer, or cold-formed steel frame with internal metal flues. Seismic forces are resisted by diaphragms with wood structural panels or metal deck, and walls with diagonal flat strap bracing. Floor and roof sheathing consists of wood structural panels or metal deck. Interior surfaces are sheathed with plaster or gypsum board. Buildings of this type that have precast concrete plank diaphragms shall not be permitted to be classified as this common building type and shall not be permitted to be evaluated using Tier 1 or Tier 2 procedures.
Concrete Moment Frames	
C1	These buildings consist of a frame assembly of cast-in-place concrete beams and columns. Floor and roof framing consists of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs. Seismic forces are resisted by concrete moment frames that develop their stiffness through monolithic beam–column connections. In older construction, or in levels of low seismicity, the moment frames are permitted to consist of the column strips of two-way flat slab systems. Modern frames in levels of high seismicity have joint reinforcing, closely spaced ties, and special detailing to provide ductile performance. This detailing is usually not present in older construction. The foundation system is permitted to consist of a variety of elements.
Concrete Shear Walls	
C2 (with Stiff Diaphragms)	These buildings have floor and roof framing that consists of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs. Buildings may also have steel beams, steel columns, cold-formed steel light-frame construction, and concrete slabs for the gravity framing. Floors are supported on concrete columns or bearing walls. Seismic forces are resisted by cast-in-place concrete shear walls. In older construction, shear walls are lightly reinforced but often extend throughout the building. In more recent construction, shear walls occur in isolated locations, are more heavily reinforced, and have concrete slabs that are stiff relative to the walls. The foundation system is permitted to consist of a variety of elements.
C2a (with Flexible Diaphragms)	These buildings are similar to C2 buildings, except that diaphragms consist of wood sheathing, or have large aspect ratios, and are flexible relative to the walls.

continues

Table 3-1 *(Continued).* Common Building Types

Concrete Frames with Infill Masonry Shear Walls	
C3 (with Stiff Diaphragms)	This is an older type of building construction that consists of a frame assembly of cast-in-place concrete beams and columns. The floor and roof diaphragms consist of cast-in-place concrete slabs and are stiff relative to the walls. Walls consist of infill panels constructed of solid clay brick, concrete block, or hollow clay tile masonry. The seismic performance of this type of construction depends on the interaction between the frame and the infill panels. The combined behavior is more like a shear wall structure than a frame structure. Solidly infilled masonry panels form diagonal compression struts between the intersections of the frame members. If the walls are offset from the frame and do not fully engage the frame members, the diagonal compression struts do not develop. The strength of the infill panel is limited by the shear capacity of the masonry bed joint or the compression capacity of the strut. The postcracking strength is determined by an analysis of a moment frame that is partially restrained by the cracked infill. The shear strength of the concrete columns, after racking of the infill, is permitted to be limited by the semiductile behavior of the system. The foundation system is permitted to consist of a variety of elements.
C3a (with Flexible Diaphragms)	These buildings are similar to C3 buildings, except that diaphragms are flexible and consist of wood sheathing or untopped metal deck or have large aspect ratios and are flexible relative to the walls.
Precast or Tilt-Up Concrete Shear Walls	
PC1 (with Flexible Diaphragms)	These buildings have precast concrete perimeter wall panels and often, interior walls, that are typically cast on site and tilted into place. The panels are interconnected by weldments, cast-in-place concrete pilasters, or collector elements. Floor and roof framing consists of wood joists, glulam beams, steel beams, or open web joists. Framing is supported on interior steel or wood columns and perimeter concrete bearing walls. The floors and roof consist of wood sheathing or untopped metal deck. Seismic forces are resisted by the precast concrete perimeter wall panels. Wall panels are permitted to be solid or have large window and door openings that cause the panels to behave more as frames than as shear walls. In older construction, wood framing is attached to the walls with wood ledgers. The roof framing is permitted to have tension-capable connections between elements. The foundation system is permitted to consist of a variety of elements.
PC1a (with Stiff Diaphragms)	These buildings are similar to PC1 buildings, except that diaphragms consist of precast elements, cast-in-place concrete, or metal deck with concrete fill and are stiff relative to the walls.
Precast Concrete Frames	
PC2 (with Shear Walls)	These buildings consist of a frame assembly of precast concrete girders and columns with the presence of shear walls. Floor and roof framing consists of precast concrete planks, tees, or double-tees supported on precast concrete girders and columns, some or all of which are permitted to be pre- or post-tensioned. Seismic forces are resisted by precast or cast-in-place concrete shear walls, which are permitted to also bear gravity loads. Diaphragms consist of precast elements interconnected with welded inserts, cast-in-place closure strips, or reinforced concrete topping slabs. The foundation system is permitted to consist of a variety of elements.
PC2a (without Shear Walls)	These buildings are similar to PC2 buildings, except that concrete shear walls are not present. Seismic forces are resisted by precast concrete moment frames that develop their stiffness through beam–column joints rigidly connected by welded inserts or cast-in-place concrete closures. Diaphragms consist of precast elements interconnected with welded inserts, cast-in-place closure strips, or reinforced concrete topping slabs. The foundation system is permitted to consist of a variety of elements.
Reinforced Masonry Bearing Walls with Flexible Diaphragms	
RM1	These buildings have bearing walls that consist of reinforced brick or concrete block masonry. The floor and roof framing consists of steel or wood beams and girders, cold-formed steel light-frame construction, or open web joists and are supported by steel, wood, or masonry columns. Seismic forces are resisted by the reinforced brick or concrete block masonry shear walls. Diaphragms consist of straight or diagonal wood sheathing, plywood, or untopped metal deck and are flexible relative to the walls. The foundation system is permitted to consist of a variety of elements.

continues

Table 3-1 *(Continued).* Common Building Types

Reinforced Masonry Bearing Walls with Stiff Diaphragms	
RM2	These building are similar to RM1 buildings, except that the diaphragms consist of metal deck with concrete fill, precast concrete planks, tees, or double-tees, with or without a cast-in-place concrete topping slab and are stiff relative to the walls. The floor and roof framing is supported on interior steel or concrete frames or interior reinforced masonry walls. The foundation system is permitted to consist of a variety of elements.
Unreinforced Masonry Bearing Walls	
URM (with Flexible Diaphragms)	These buildings have perimeter bearing walls that consist of unreinforced clay brick, stone, or concrete masonry. Interior bearing walls, where present, also consist of unreinforced clay brick, stone, or concrete masonry. In older construction, floor and roof framing consists of straight or diagonal lumber sheathing supported by wood joists, which, in turn, are supported on posts and timbers. In more recent construction, floors consist of structural panel or plywood sheathing rather than lumber sheathing. The diaphragms are flexible relative to the walls. Where they exist, ties between the walls and diaphragms consist of anchors or bent steel plates embedded in the mortar joints and attached to framing. The foundation system is permitted to consist of a variety of elements.
URMa (with Stiff Diaphragms)	These buildings are similar to URM buildings, except that the diaphragms are stiff relative to the unreinforced masonry walls and interior framing. In older construction or large, multistory buildings, diaphragms consist of cast-in-place concrete. In levels of low seismicity, more recent construction consists of metal deck and concrete fill supported on steel framing. The foundation system is permitted to consist of a variety of elements.

3.2.5.2 Shared Element Condition. Data shall be collected on adjacent structures that share common vertical- or seismic-force-resisting elements with the building to permit investigation of the implications of the adjacent structure's influence on the performance of the investigated building in accordance with the selected evaluation procedure.

3.2.5.3 Hazards from Adjacent Buildings. Data on hazards posed to the subject building by adjacent buildings and their elements shall be collected to permit consideration of their potential to damage the subject building as a result of an earthquake. If there is a potential for such hazards from an adjacent building, the Authority Having Jurisdiction over the subject building shall be informed of the effect of such hazards on achieving the selected Performance Objective.

3.3 BENCHMARK BUILDINGS

Buildings designed and constructed or evaluated in accordance with the benchmark provisions of this section shall be deemed to comply with the provisions of this standard for the Structural Performance Levels indicated. However, an evaluation of nonstructural elements in accordance with Section 17.19 shall be performed where required by this standard.

This section shall consider the provisions under which the structure was originally designed, retrofitted, or previously evaluated. Buildings that have been fully retrofitted shall be evaluated using the standards used for the retrofit, not the original design provisions. The edition of a design code or provisions or the retrofit standard that sets the benchmark year shall be as indicated in Table 3-2 for Life Safety performance for the BSE-1E Seismic Hazard Level and Table 3-3 for Immediate Occupancy performance for the BSE-1E Seismic Hazard Level. Buildings that satisfy the benchmark criteria in Table 3-2 shall be deemed to comply with Collapse Prevention performance for the BSE-2E Seismic Hazard Level.

The design professional shall document the evidence used to determine that the building complies with the provisions of this section. The existing building shall comply with Sections 3.3.1 through 3.3.4. If the building is determined to be noncompliant with any of these sections or compliance cannot be determined, the structure does not meet the Benchmark Building provisions of this section.

3.3.1 Existing Documents. Review of the record drawings of the structure shall be performed to confirm that the primary elements of the seismic-force-resisting system and their detailing were intended to be designed in accordance with the applicable provisions listed in Tables 3-2 or 3-3.

3.3.2 Field Verification. Field verification shall be performed to confirm that the building was constructed in general conformance with record drawings and that no modifications have been made that significantly affect the expected performance of the lateral-force-resisting system.

3.3.3 Condition Assessment. Field verification confirms that significant deterioration of structural materials has not occurred.

3.3.4 Geologic Site Hazards. There shall be no liquefaction, slope failure, or surface fault rupture hazard present at the building site. Alternatively, if such a hazard is present, the hazard has been mitigated by the design of the lateral-force-resisting system, including foundations.

3.4 EVALUATION AND RETROFIT PROCEDURES

Seismic evaluation or retrofit of the building shall be performed to demonstrate compliance with the selected Performance Objective in accordance with the requirements of the following sections. Section 3.4.1 covers the limitations on the use of the Tier 1 and Tier 2 procedures. Section 3.4.2 addresses the Tier 1 screening procedure for evaluation. Section 3.4.3 addresses the Tier 2 deficiency-based procedures for evaluation and retrofit. Section 3.4.4 addresses the Tier 3 systematic procedures for evaluation and retrofit.

Appendix C: Benchmark Building Code

Table 3-2. Benchmark Building Codes and Standards for Life Safety Structural Performance at BSE-1E

	Building Seismic Design Provisions				Seismic Evaluation or Retrofit Provisions		
Building Type[a,b,c]	NBC SBC	UBC	IBC	NEHRP	FEMA 178	FEMA 310[d] (1998e)/ ASCE 31[d]	FEMA 356[e] (2000)/ ASCE 41[e]
Wood frame, wood shear panels (Types W1 and W2)	1993	1976	2000	1985	[f]	1998	2000
Wood frame, wood shear panels (Type W1a)	[f]	1997	2000	1997	[f]	1998	2000
Steel moment-resisting frame (Types S1 and S1a)	[f]	1994[g]	2000	1997	[f]	1998	2000
Steel concentrically braced frame (Types S2 and S2a)	[f]	1997	2000	[f]	[f]	1998	2000
Steel eccentrically braced frame (Types S2 and S2a)	[f]	1988[g]	2000	1997	[f]	[f]	2000
Buckling-restrained braced frame (Types S2 and S2a)	[f]	[f]	2006	[f]	[f]	[f]	2000
Metal building frames (Type S3)	[f]	[f]	2000	[f]	1992	1998	2000
Steel frame with concrete shear walls (Type S4)	1993	1994	2000	1985	[f]	1998	2000
Steel frame with URM infill (Types S5 and S5a)	[f]	[f]	2000	[f]	[f]	1998	2000
Steel plate shear wall (Type S6)	[f]	[f]	2006	[f]	[f]	[f]	2000
Cold-formed steel light-frame construction—shear wall system (Type CFS1)	[f]	1997[h]	2000	1997[h]	[f]	[f]	2000[h]
Cold-formed steel light-frame construction—strap-braced wall system (Type CFS2)	[f]	[f]	2003	2003	[f]	[f]	[f]
Reinforced concrete moment-resisting frame (Type C1)[i]	1993	1994	2000	1997	[f]	1998	2000
Reinforced concrete shear walls (Types C2 and C2a)	1993	1994	2000	1985	[f]	1998	2000
Concrete frame with URM infill (Types C3 and C3a)	[f]	[f]	2000	[f]	[f]	1998	2000
Tilt-up concrete (Types PC1 and PC1a)	[f]	1997	2000	[f]	[f]	1998	2000
Precast concrete frame (Types PC2 and PC2a)	[f]	[f]	2000	[f]	1992	1998	2000
Reinforced masonry (Type RM1)	[f]	1997	2000	[f]	[f]	1998	2000
Reinforced masonry (Type RM2)	1993	1994	2000	1985	[f]	1998	2000
Unreinforced masonry (Type URM)	[f]	[f]	2000	[f]	[f]	[f]	2000
Unreinforced masonry (Type URMa)	[f]	[f]	2000	[f]	[f]	1998	2000
Seismic isolation or passive dissipation	[f]	1991	2000	[f]	[f]	[f]	2000

Note: NBC = *National Building Code*. SBC = *Standard Building Code*. UBC = *Uniform Building Code*. IBC = *International Building Code*. NEHRP = FEMA 368 and 369, *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings* (BSSC 2000), FEMA 178, FEMA 310, FEMA 356, ASCE 31-03, ASCE 41-06, and ASCE 41-13.

[a] Building type refers to one of the common building types defined in Table 3-1.
[b] Buildings on hillside sites shall not be considered Benchmark Buildings.
[c] For buildings in Very Low Seismicity, the benchmark provisions shall be limited to the IBC, FEMA 310/ASCE 31, and FEMA 356/ASCE 41.
[d] Life Safety Structural Performance Level for the seismic hazard as defined by those provisions.
[e] Life Safety Structural Performance Level for the BSE-1 seismic hazard as defined by those provisions.
[f] No benchmark year; buildings shall be evaluated using this standard.
[g] Steel moment-resisting frames and eccentrically braced frames with links adjacent to columns shall comply with the 1994 UBC Emergency Provisions, published September/October 1994, or subsequent requirements.
[h] Cold-formed steel shear walls with wood structural panels only.
[i] Flat slab concrete moment frames shall not be considered Benchmark Buildings.

Table 3-3. Benchmark Building Codes and Standards for Immediate Occupancy Structural Performance at BSE-1E

Building Type[a,b]	Seismic Evaluation or Retrofit Provisions	
	FEMA 310[c] (1998e)/ ASCE 31[c]	FEMA 356[d] (2000)/ ASCE 41[d]
Wood frame, wood shear panels (Types W1 and W2)	1998	2000
Wood frame, wood shear panels (Type W1a)	1998	2000
Steel moment-resisting frame (Types S1 and S1a)	1998	2000
Steel concentrically braced frame (Types S2 and S2a)	1998	2000
Steel eccentrically braced frame (Types S2 and S2a)	e	2000
Buckling-restrained braced frame (Types S2 and S2a)	e	2000
Metal building frame (Type S3)	1998	2000
Steel frame with concrete shear walls (Type S4)	1998	2000
Steel frame with URM infill (Types S5 and S5a)	1998	2000
Steel plate shear wall (Type S6)	e	2000
Cold-formed steel light-frame construction—shear wall system (Type CFS1)	e	e
Cold-formed steel light-frame construction—strap-braced wall system (Type CFS2)	e	e
Reinforced concrete moment-resisting frame (Type C1)[f]	1998	2000
Reinforced concrete shear walls (Types C2 and C2a)	1998	2000
Concrete frame with URM infill (Types C3 and C3a)	1998	2000
Tilt-up concrete (Types PC1 and PC1a)	1998	2000
Precast concrete frame (Types PC2 and PC2a)	1998	2000
Reinforced masonry (Type RM1)	1998	2000
Reinforced masonry (Type RM2)	1998	2000
Unreinforced masonry (Type URM)	e	2000
Unreinforced masonry (Type URMa)	1998	2000
Seismic isolation or passive dissipation	e	2000

Sources: FEMA 310, FEMA 356, ASCE 31-03, ASCE 41-06, and ASCE 41-13.
[a] Building type refers to one of the common building types defined in Table 3-1.
[b] Buildings on hillside sites shall not be considered Benchmark Buildings.
[c] Immediate Occupancy Structural Performance Level for the seismic hazard as defined by those provisions.
[d] Immediate Occupancy Structural Performance Level for the BSE-1 seismic hazard as defined by those provisions.
[e] No benchmark year; buildings shall be evaluated using this standard.
[f] Flat slab concrete moment frames shall not be considered Benchmark Buildings.

A building defined as one of the common building types, or those buildings that have seismic isolation or supplemental energy dissipation systems installed, that meet the requirements of Section 3.3, Benchmark Buildings, shall be deemed to meet the structural performance objective as defined in that section. The nonstructural performance must still be evaluated.

3.4.1 Limitations on the Use of Tier 1 and Tier 2 Evaluation and Retrofit Procedures. The Tier 1 screening and Tier 2 deficiency-based procedures shall only be used with a Performance Objective that satisfies at least one of the following conditions:

1. The Performance Objective involves a Seismic Hazard Level less than or equal to BSE-1E with a Structural Performance Level up to and including Immediate Occupancy (S-1) and/or a Nonstructural Performance Level up to and including Position Retention (N-B).
2. The Performance Objective involves a Seismic Hazard Level greater than BSE-1E but less than or equal to BSE-2E with a Structural Performance Level up to and including Life Safety (S-3) and/or a Nonstructural Performance Level up to and including Life Safety (N-C).

The selected Seismic Hazard Level shall be compared to BSE-1E or BSE-2E by comparing the respective values of S_S and S_1.

In addition, the Tier 1 and Tier 2 procedures shall only be used for buildings that conform to the limitations of Table 3-4 and of Section 3.4.1.1 or 3.4.1.2.

In many cases, deficiency-based retrofit represents a cost-effective improvement in seismic performance, and it often requires less detailed evaluation or partial analysis to qualify for a specific performance level. Partial Retrofit Objective measures, which target high-risk building deficiencies such as parapets and other exterior falling hazards, are included as deficiency-based techniques. Partial Retrofit Objective measures need not be limited to buildings that conform to the limitations of Table 3-4. Acceptance of the specific partial retrofit method for regulatory purposes depends on the Authority Having Jurisdiction.

Regardless of whether it is permitted for use, the Tier 1 screening in Chapter 4 is a good starting point for the identification of potential deficiencies for any building type covered here and being evaluated using this standard.

3.4.1.1 Buildings Conforming to One of the Common Building Types. Where a building conforms to one of the common building types contained in Table 3-1, the limitations in Table 3-4 with regard to building size, Structural Performance Level, and Level of Seismicity determine whether the Tier 1 screening and Tier 2 deficiency-based procedures are allowed to demonstrate compliance with the Performance Objectives of this standard.

Appendix D: Geotechnical Investigations

THE GALLI GROUP
Geotechnical Consulting

PRELIMINARY SEISMIC RETROFIT STUDY
JACKSON ELEMENTARY SCHOOL
MEDFORD, OREGON

For: Medford School District
c/o Stephen L. Chase, Project Manager
ZCS Engineering & Architecture
127 NW D Street
Grants Pass, Oregon 97526

c/o Stephen L. Chase, Project Manager
ZCS Engineering & Architecture
127 NW D Street
Grants Pass, Oregon 97526

By: **THE GALLI GROUP**
612 NW Third Street
Grants Pass, OR 97526
(541) 955-1611

02-6007-01
June 30, 2021

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PRELIMINARY SEISMIC RETROFIT STUDY
JACKSON ELEMENTARY SCHOOL
MEDFORD, OREGON

1.0 INTRODUCTION

This report presents results of our geotechnical and geological evaluation of the Jackson Elementary School for a potential seismic retrofit of portions of the school campus. The subject school is located at 630 W. Jackson Street, on the northwest corner of the intersection of West Jackson Street and Summit Avenue in Medford Oregon. Please see Figure 1, Vicinity Map, for a more precise location.

The purpose of our investigation and this report was to accomplish a limited site surface and subsurface evaluation (one boring) and conduct a planning level seismic risk assessment (office studies) in order to provide preliminary geotechnical and geologic information and evaluate the likelihood and consequences of geotechnical/geologic related seismic failures, including liquefaction and landslide potential during the design seismic event, for consideration of the potential seismic retrofit.

2.0 SITE AND PROJECT DESCRIPTION

The site is currently occupied by a functioning elementary school, which consists of 7 structures connected via covered walkways or with direct connections. The structures are surrounded by play fields, access roads, parking, walkways and open space. The site is relatively flat with undeveloped portions of the site consisting of well-maintained lawns, landscape areas and a few trees.

We understand the district is conducting a preliminary review to determine the level of seismic retrofit needed for the structures on this campus. Their review will largely be based on the evaluation of the potential geologic hazards (such as liquefaction) provided in this report, and an evaluation of the potential structural damage to these facilities associated with the design seismic event. The findings will also likely be used to determine if grant funding will be pursued to complete the seismic retrofit work.

3.0 FIELD EXPLORATION

On June 11, 2021, Associate Engineer, Dennis Duru, M.Sc., E.I.T and our drilling crew, visited the site to accomplish the subsurface investigation. One (1) exploratory boring was drilled in the grass field, west of Building G. A utility locate was completed prior to our investigation and our representative coordinated with school personnel to identify the field exploration location away from the marked and known utility locations. See Figure 2, Site Plan, for a layout of the site and the location of the boring. The boring was drilled with our ATV-mounted, solid-stem auger drill rig and penetrated to depth of 5.5 feet. Upon completion, the boring was backfilled with drill spoils.

Standard Penetration Testing (SPT) was accomplished in each boring. This entails driving a 1½-inch diameter steel split spoon sampler by dropping a 140-pound weight for a 30-inch drop. The total number of blows it takes to drive the sampler the last 12 inches of an 18-inch drive is called the SPT N-value. These can be correlated with density and soil strength parameters from testing on thousands of other projects.

Our representative identified the final exploration location, logged subsurface soils and water conditions and obtained soil samples for transport to our laboratory. Visual classifications of the soils were made in the field and are presented in the Boring Logs in Appendix A, at the end of this report.

4.0 LABORATORY TESTING

Moisture content tests were accomplished on soil samples obtained by Standard Penetration Testing. No other tests were accomplished.

5.0 SUBSURFACE CONDITIONS

5.1 SOIL

The boring encountered a surficial layer of brown silty Sand to the depth of 2.0 feet. This was then underlain by dense to very dense, sandy Gravel (weathered conglomerate Sandstone, see site geology in Section 6.0).

Please see more specific soils information in the Boring Logs in Appendix A. Please note that the soils are shown as distinct layers in the Boring Logs while in nature they may change more gradually. Soils conditions may also change somewhat between the locations investigated.

5.2 GROUNDWATER

Generally, the soils encountered were damp. No groundwater was encountered in the single boring accomplished. Review of nearby well log information shows that static groundwater levels in the area are at approximately 10 feet below the ground surface.

6.0 GEOLOGIC OR SEISMIC INDUCED HAZARDS

Summary of Site Geology and Seismicity. The project area is located in the Medford East 30x60 minute USGS topographic quadrangle (see Vicinity Map, Figure 1). Mapped geologic units at the project area consist primarily of Alluvial Fan deposits and bedrock members of the Hornbrook Formation (Wiley et al, 2011). The Marine Sandstone, Siltstone and Conglomerate members are the mapped bedrock unit at the project (Wiley et al, 2011; OGDC-6, 2015). Weathered conglomerate Sandstone bedrock was encountered in the auger boring at this site at a relatively shallow depth of 5 feet below ground surface.

The project site is in proximity to several zones of active seismicity. The region is affected by the Cascadia Subduction Zone (CSZ), an active subduction zone off the Oregon coast, considered capable of Magnitude 8.5 or greater earthquakes. Average recurrence intervals for such great earthquakes, as determined by recent investigations, range between 300-600 years. The last "great" earthquake was interpreted to be approximately 300 years ago. The CSZ is the main seismic event for consideration for this seismic retrofit.

The project area has an additional tectonic source from earthquakes occurring along active Basin and Range faults as close as 50 kilometers to the southeast. This region has produced numerous earthquakes, including significant events near Klamath Falls and Warner Valley. Such events occur generally once every one to two decades.

Flood Hazard. The site is not near streams or rivers. Therefore, it is not within a 100-year floodplain.

Landslides/Slope Instability. There are no slopes close to the site. Therefore, possibility of slope failure, rock fall or slide run out damage at the site is low.

Liquefaction and Lateral Spread. The project is underlain by dense to very dense sandy Gravel. The dense sandy Gravel encountered in our boring is the top of the weathered Sandstone bedrock and will not liquefy in a seismic event. Therefore, liquefaction and lateral spread is not considered to be a potential hazard for this site. See more in a later section of this report.

Expansive Soils. The upper soils within the subsurface consist of silty Sands. These will have zero to mild expansion potential based on our experience with soils with similar visual properties.

Ground Rupture. No Quaternary faults were identified at the project site. Therefore, the risk of damage at the site due to ground rupture is considered very low.

Ground Shaking. Project structures including foundations and retaining walls must be designed for the potential for very strong ground shaking during the anticipated seismic event**.** The peak site modified horizontal acceleration (PGA_M), is 0.358g. This is based on the Site Class C designation, determined for the project from subsurface drilling and evaluation of SPT data. The PGA_M value can be used with an appropriate seismic coefficient in pseudo static analysis for design of the seismic upgrades.

Seismic Ground Amplification or Resonance. No unusually hazardous amplification or resonance effects on seismic waves have been associated with soil/bedrock subsurface conditions in the project area.

Tsunami and Seiche. The site is approximately 80 miles inland from the coast, and not subject to tsunami hazard. The site is not located adjacent to a large lake or body of water, and therefore, not subject to seiche hazard.

7.0 LIQUEFACTION EVALUATION

The liquefaction phenomenon occurs in saturated, loose (low density, uncompacted or poorly compacted), cohesionless soils. When loose, cohesionless soils are saturated which is the case when the soil is below the water table, then water fills the soil pores. In response to compression when a load is applied to the soil, the water increases in pressure and attempts to migrate towards zones of low pressure which is usually upwards towards the ground surface. However, if the applied load is rapid and large enough, or if it is repeated many times (cyclic loading) like in an earthquake, such that there is not enough time for the water to dissipate before the next cycle of loading is applied, then the water pressure may build up to a degree where they become greater than the grain-to-grain contact stresses of the soil. The grain-to-grain contact stresses are the source of the shear strength that supports structure foundations and overburden soils in these soil structures. This buildup of excess pore water pressure results in total or partial loss of the soil strength, and the soil may be observed to flow like a liquid, hence “liquefaction”. At this point, the soil will lose all its shear strength, be deformed, and will not be able to support structures.

The site is underlain by dense to very dense Sand and Gravel. No groundwater was observed in the boring to the depth drilled. The conditions for liquefaction to occur were not observed at this site during our limited subsurface investigation. The sand and gravel soils are not loose, they are dense and very dense. Such soils will not undergo further densification enough to cause liquefaction during a seismic event. Also, water was not observed and could be below 10 feet deep. Therefore, liquefaction cannot take place.

Therefore, in our professional opinion, the site conditions found in the boring will not result in wide spread liquefaction during a seismic event that will have significant adverse impacts on the structures.

8.0 CONCLUSIONS

In our professional opinion, based on our field investigation, office review and previous work in the area, the soils conditions at the site are suitable for a conventional seismic retrofit. Crushed rock structural fill over the silty Sand and Gravel will provide adequate support of new foundations, grade beams and/or buttresses (or small diameter piles could be used to limit overexcavation). In our opinion, this school site is not subject to large scale liquefaction that will cause a significant adverse impact to the structures.

If a full seismic retrofit geotechnical design report is needed, additional tasks to be accomplished would be as follows:

1. 2 or 3 additional borings.
2. Laboratory testing for strength and settlement evaluation.
3. Evaluation of data for developing design parameters.

These could be used to provide a full scale Seismic Retrofit Design Report.

8.1 LIMITATIONS

The analyses, conclusions and recommendations contained in this report are based on site conditions as they existed at the time of the study, and assume soils, rock and groundwater conditions exposed and observed in the boring during our investigation are representative of soils and groundwater conditions throughout the site. If during construction, subsurface conditions or assumed design information is found to be different, we should be advised at once so that we can review this report and reconsider our recommendations in light of the changed conditions. If there is a significant lapse of time (5 years) between submission of this report and the start of work at the site, if the project is changed, or if conditions have changed due to acts of God or construction at or adjacent to the site, it is recommended that this report be reviewed in light of the changed conditions and/or time lapse.

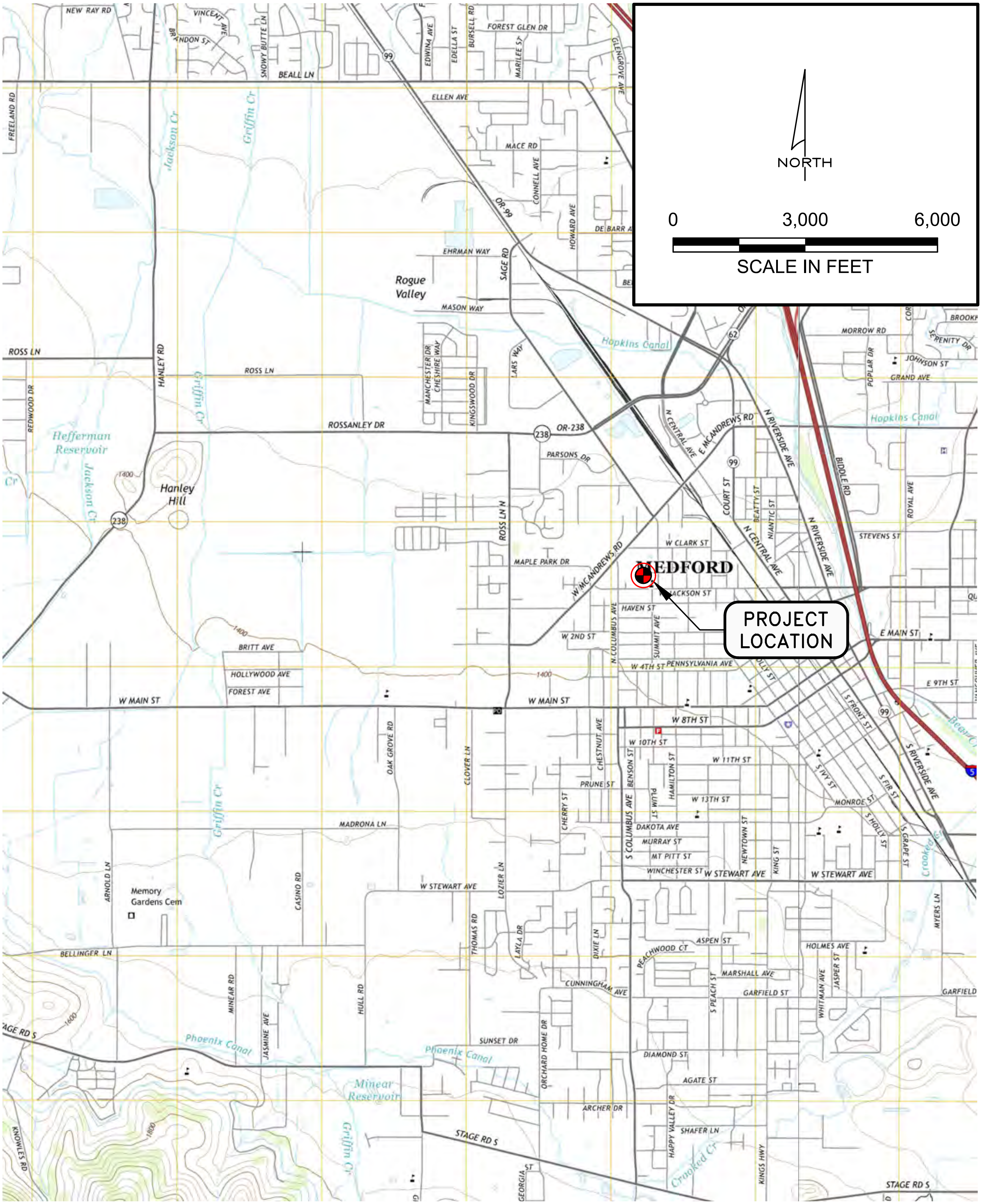
This report was prepared for the use of the School District and their design team for evaluating the need for a full scale Seismic Retrofit evaluation and report. It should be made available to contractors for information and factual data only. This report should not be used for contractual purposes as a warranty of site subsurface conditions. It should also not be used at other sites or for projects other than the one intended.

We have performed these services in accordance with generally accepted geotechnical engineering and professional geology practices in southern Oregon, at the time the study was accomplished. No other warranties, either expressed or implied, are provided.

THE GALLI GROUP
GEOTECHNICAL CONSULTING

Dennis Duru, M.S., E.I.T.
Staff Associate

Melvin Galli III, P.E.
Principal Engineer



THE GALLI GROUP
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612 NW 3rd Street
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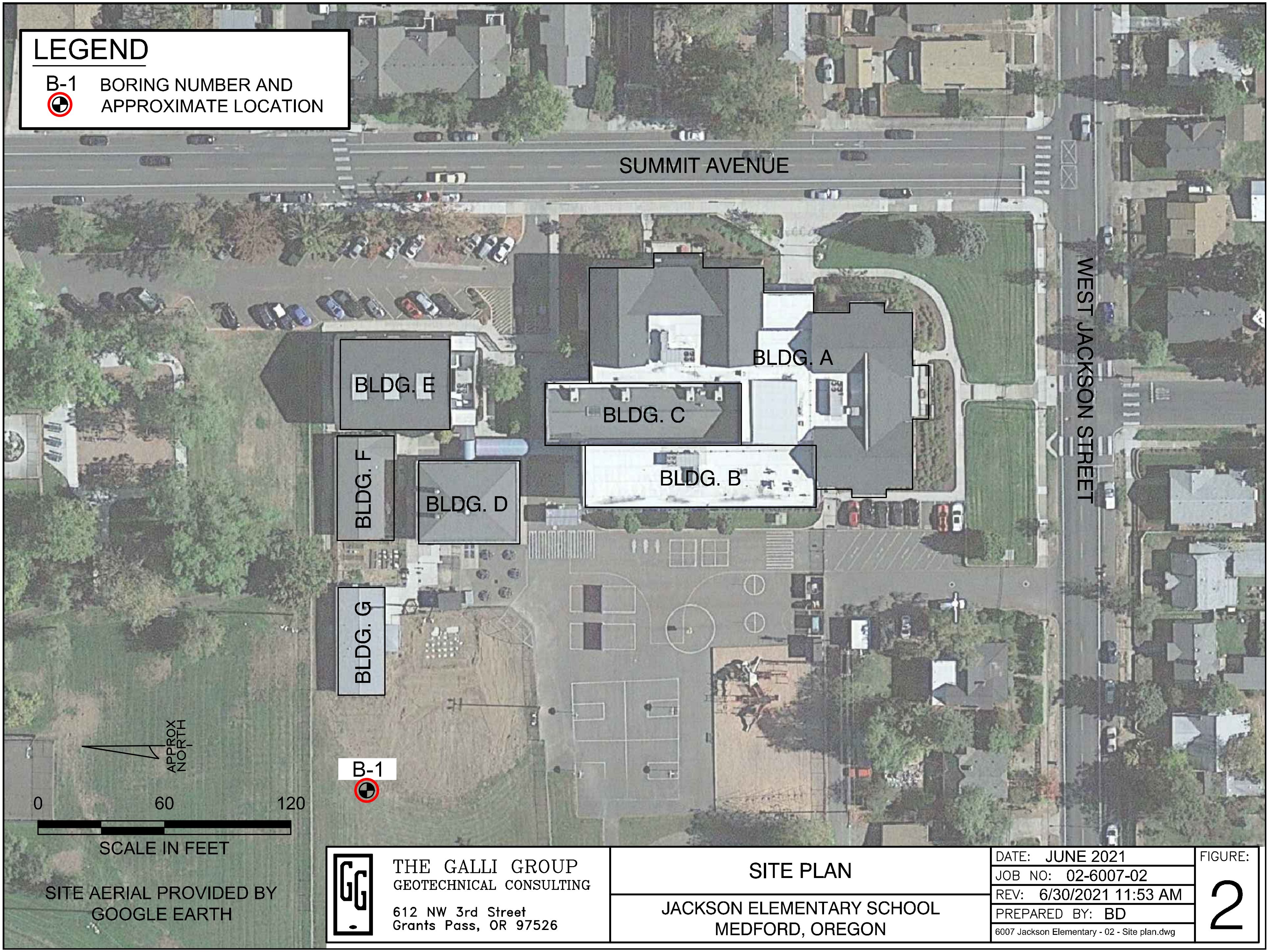
VICINITY MAP

JACKSON ELEMENTARY SCHOOL
MEDFORD, OREGON

DATE:	JUNE 2021
JOB NO:	02-6007-02
REV:	6/28/2021 4:03 PM
PREPARED BY:	BD
6007 Jackson Elementary - 01 - Vicinity.dwg	

FIGURE: 1



THE GALLI GROUP GEOTECHNICAL CONSULTING 612 NW 3rd Street Grants Pass, OR 97526	SITE PLAN JACKSON ELEMENTARY SCHOOL MEDFORD, OREGON	DATE: JUNE 2021 JOB NO: 02-6007-02 REV: 6/30/2021 11:53 AM PREPARED BY: BD 6007 Jackson Elementary - 02 - Site plan.dwg	FIGURE: 2

APPENDIX A

BORING LOGS

THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

BORING LOG
B-1

Project: Jackson Elmentary School
Client: Medford School District
Location: The field south of building "G" (see site plan)
Driller: TGG (Ken, Nick)
Drill Rig: ATV Mounted Rig, 4" Diameter SSA.
Depth To Water> Initial : None At Completion : None

Project No.: 02-6007-01
Date: 6/11/2021
Elevation:
Logged By: Dennis Duru

USCS	Description	Depth	Sample No. and Type	NMC	Standard Penetration Test N
OL	Grass rootzone.	0 – 0.25			
SM	Dense, light brown, silty Sand; some gravel, moist.	0.25 – 2.0			
SW/GW	Dense to very dense, light brown, sandy Gravel; damp. (top of weathered conglomerate Sandstone)	2.0 – 5.5	S-1	10%	40
			S-2	10%	65
	Bottom of boring at 5.5 feet. No groundwater encountered.	5.5			

Standard Penetration Test CURVE: 10, 30, 50; 65 →

Depth: 0, 1, 2, 3, 4, 5, 6, 7

Legend of Samplers: Grab sample SPT sample Shelby tube sample

This information pertains only to this boring and should not be interpreted as being indicative of the site.

THE GALLI GROUP
Geotechnical Consulting

GEOTECHNICAL & GEOLOGICAL INVESTIGATION REPORT
SEISMIC RETROFIT DESIGN
JACKSONVILLE ELEMENTARY SCHOOL
JACKSONVILLE, OREGON

For: Ron Havniear
Facility Manager
Medford School District
Medford, OR 97501

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02-5758-01
November 27, 2019

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THE GALLI GROUP
Geotechnical Consulting

GEOTECHNICAL & GEOLOGICAL INVESTIGATION REPORT SEISMIC RETROFIT DESIGN JACKSONVILLE ELEMENTARY SCHOOL JACKSONVILLE, OREGON

1.0 INTRODUCTION

This report presents results of our geotechnical and geology evaluation of the site for the seismic retrofit to the Jacksonville Elementary School in Jacksonville, Oregon. The subject property is located on the north side of Huener's Lane, northwest of its intersection with Fritillaria Lane, east of Hwy. 238 in Jacksonville, Oregon. Please see Figure 1, Vicinity Map, for a more precise site location.

The purpose of this investigation and report was to evaluate the site surface and subsurface conditions with three (3) borings and conduct office studies in order to provide geotechnical and geologic recommendations for design and construction of the seismic retrofit. This may include improving the foundations and/or adding embedded footings/buttresses for lateral and vertical resistance of loads generated in a seismic event.

2.0 SITE AND PROJECT DESCRIPTION

The site is currently occupied by a functioning elementary school, which apparently contains one main large structure, an outdoor play area, drive lanes and parking lots.

We understand the project to consist of carrying out a Seismic Retrofit to the school in order to bring it up to current code requirements for public schools. This will likely require structural upgrades including improved foundations and/or embedded footings/buttresses for resistance of lateral loads generated in a seismic event.

3.0 FIELD EXPLORATION

On November 6, 2019, Staff Associate, Dennis Duru, M.Sc. E.I.T. and our drilling crew, visited the site to accomplish the subsurface investigation. A total of three (3) exploratory borings were drilled to depths between 5.0 and 8.0 feet around the structure at the locations shown on Figure 2, Site Plan. **Note:** One additional boring at the front of the school was planned. It was omitted due to numerous utility conflicts. The drilling was accomplished with our ATV-mounted solid stem auger drill rig. Borings terminated in the dense Sand and Gravel. All holes were refilled after drilling with drill spoils and pea gravel. The areas were left clean of most soil debris.

A utility locate was completed prior to our investigation and our representative identified the field exploration locations away from the marked utilities (we did not accomplish a 4th boring due to the heavy presence of utilities around the surrounding areas). Borings were advanced with sample collection and testing being accomplished at various depths. Standard Penetration Testing (SPT) was accomplished in each boring. This entails driving a 1½ inch I.D, 2-inch O.D., steel split spoon sampler by dropping a 140-pound weight for a 30-inch drop. The total number of blows it takes to drive the sampler the last 12 inches of an 18-inch drive is called the SPT N-value. These can be correlated with soil strength and density parameters from testing on thousands of other projects.

Our representative identified the final exploration locations, logged subsurface soils and water conditions and obtained soil samples for transport to our laboratory. Visual classifications of the soils were made in the field and are presented in the Boring Logs in Appendix A, at the end of this report. Please note that in the logs, soil changes are depicted as distinct layers, while in nature they likely are more gradual.

4.0 LABORATORY TESTING

Soil samples were tested for natural moisture content. One Expansion Index (EI) test was accomplished on a bulk sample at approximately 1.2 feet in B-1. Results were an EI=70. This means these soils are moderately expansive (change in volume with change in moisture content).

5.0 SUBSURFACE CONDITIONS

5.1 SOIL

The borings encountered soft to medium stiff clayey silt somewhat expansive in the upper 2 feet, which transitioned to medium dense clayey/silty gravel and/or sand. The borings then terminated in the very dense Sand and Gravel which is the top of a weathered meta-sedimentary rock formation. Foundations will likely be founded on crushed rock over the medium dense clayey/silty sand and gravel layer. Please see more specific soils information in the Boring Logs in Appendix A. Please note that the soils are shown as distinct layers in the Boring Logs while in nature they may change more gradually. Soils conditions may also change somewhat between the locations investigated.

5.2 GROUNDWATER

Generally, the upper soils encountered were moist and the soils deeper are dry to damp. No groundwater was encountered in all the borings. During wet weather, water will tend to perch on the top of the weathered rock layer. The upper silty soils could become unworkable when they become saturated during the wetter months of the year.

6.0 GEOLOGIC AND SEISMIC DESIGN

6.1 ASCE 7/16 DESIGN EARTHQUAKE

The design earthquake for the project area is based upon established values and methodology in ASCE 07-16.

The Maximum Considered Earthquake (MCE_R) and spectral response accelerations were established as set forth in Chapters 11 and 20 of ASCE 7/16 and were partly obtained from the online ASCE 7 Hazard Tool (ASCE, 2019).

Table 1- DESIGN EARTHQUAKE (ASCE 7-16)

Project Area: Jacksonville Elementary School, Jacksonville, Oregon	Lat. 42.317833 Long. -122.958157
Risk Category	III
Mapped Spectral Response Acceleration, MCER Short Period Ss, 0.2s (from Figure 22-1) ASCE 7-16	66.3% of g =0.663g
MCER 1 sec Period S1, (from Figure 22-2) ASCE 7-16	38% of g = 0.380g
Site Class	C
Site Coefficients Fa, Short Period (Table 11.4-1 ASCE 7-16)	1.24
Site Coefficients Fv, 1 sec Period (Table 11.4-2 ASCE 7-16)	1.5
Spectral Response Acceleration, SMS, Short Period (Fa*Ss equation 11.4-1 ASCE 7-16)	1.24*0.663 =0.822g
Spectral Response Acceleration, SM1, 1 sec Period (Fv*Ss equation 11.4-1 ASCE 7-16)	1.5*0.380 =0.570g
Design Spectral Acceleration SDS, Short Period ((2/3)*SMS equation 11.4-3 ASCE 7-16)	(2/3)*0.822=0.548g
Design Spectral Acceleration SD1, 1 sec Period ((2/3)*SM1 equation 11.4-3 ASCE 7-16)	(2/3)*0.570=0.380g
MCEG, PGA (Figure 22-9 ASCE 7-16)	31.3% of g = 0.313g
Site coefficient, FPGA (Table 11.8-1 ASCE 7-16)	1.2
MCEG adjusted for site class effects, PGAm (FPGA*PGA equation 11.8-1 ASCE 7-16)	1.200*0.313 = 0.376g
Seismic Design Category (Table 11.6-1 and 11.6-2 ASCE 7-16)	0.5<=SDS = D 0.2<=SD1 = D
Per the requirements of Section 11.6 of the ASCE 7-16 code, the more severe seismic category is assigned which is **CATEGORY D**	

6.2 GEOLOGIC OR SEISMIC INDUCED HAZARDS

Landslides/Slope Instability. There are no slopes in the immediate vicinity of the site. Therefore, there is no possibility of slope failure, rock fall or slide run out damage.

Expansive Soils. The upper soils encountered are mildly to moderately expansive. The Expansion Index test yielded an EI=34.

Liquefaction and Lateral Spread. The upper parts of the project's subsurface is underlain by silty clayey Sand and Gravel up to approximately 8 feet. These were medium dense to very dense. In addition, the dense weathered rock is shallow below these layers, and ground water was not present in the borings. Therefore, liquefaction and lateral spread is not considered to be a potential hazard at the site.

Ground Rupture. No Quaternary faults were identified at the project site. Therefore, the risk of damage at the site due to ground rupture is considered very low.

Ground Shaking. A Site Class of C should be used for the project, based on a field reconnaissance and subsurface SPT data obtained at the project. Seismic design recommendations are provided in Table 1. The PGA_m for the site is 0.376g.

Seismic Ground Amplification or Resonance. No hazardous amplification or resonance effects from seismic waves have been associated with the soil subsurface conditions in the project area. Some amplification is possible. However, the design parameters in the Seismic Design table should account for such added loading. The risk of damage at the site from unexpectedly severe shaking due to seismic wave amplification is low.

Tsunami and Seiche. The site is approximately 80 miles inland from the coast, and not subject to tsunami hazard. The site is not located adjacent to a large lake or body of water, and, therefore, not subject to seiche hazard.

7.0 CONCLUSIONS

In our professional opinion, based on our field investigation, and office review, and previous work in the area, the soils conditions at the site are suitable for the proposed seismic retrofit, provided the recommendations of our report are incorporated in the design and construction of the project. Crushed rock backfill over the medium dense silt/clay sand and gravel mixtures will provide good foundation and buttress support.

8.0 GEOTECHNICAL RECOMMENDATIONS

The subject site has good soils for support of the structure. The following sections provide methods and data for proper design of the seismic retrofit items.

8.1 SITE PREPARATION AND GRADING

This site has structures that may require structural and foundation upgrades for the seismic upgrade. Therefore, normal methods of demolition, debris removal, clearing, grubbing, stripping for organic removal and subgrade soil preparation will apply in areas of the work.

8.1.1 Manmade Fill & Debris Considerations

All old fill and debris encountered during construction must be removed. Soil that is clear of debris could be used in landscape berms. All other debris or debris laden soil must be wasted off site or used in landscape berms. The full extent of any waste fill removal (if any) will be determined during site stripping and excavation operations.

8.1.2 Clearing, Grubbing and Stripping

The site area that will be used for the work must be stripped and cleared of all vegetation, sod and organic topsoil. Additional stripping (or excavations) will most likely be required to remove root balls from bushes (very little of this is expected). The stripped materials removed shall be hauled from the site or stockpiled for use in landscape areas only (such as landscape mounds). This material must not be used in structural fill.

Holes or depressions resulting from the removal of underground obstructions that extend below the finish subgrade and will be beneath support items shall be cleared of all loose material and dished to provide access for compaction equipment. These areas shall then be backfilled and compacted to grade with structural fill, as described later in this report. It is recommended that grubbing and stripping of the site, old fill removal and compaction of depressions below finish subgrade, be observed and/or decided by the geotechnical engineer or his representative from The Galli Group.

8.2 STRUCTURAL FILL PLACEMENT AND COMPACTION

8.2.1 Beneath Structures

Structural fill is defined as any fill placed and compacted to specified densities and used in areas that will be under foundations, floors or structure support items. It appears that new footings and buttresses will have structural fill below, beside and/or above them. The subgrade needs to be prepared properly and the fill must be placed and compacted correctly for proper long-term performance.

Structural Fill Materials. Ideally, and particularly for wet weather construction, structural fill must consist of a free-draining granular material (non-expansive) with a maximum particle size of six inches. The material shall be reasonably well-graded with less than 5 percent fines (silt and clay size passing the No. 200 mesh sieve). During dry weather, any organic-free, non-expansive, will less than 12% passing the No. 200 sieve, compactable granular material, meeting the maximum size criteria, is typically acceptable for this purpose. Locally available crushed rock, jaw-run crushed "shale" (low-grade rock) and good quality Sandy Decomposed Granite [DG]) have performed adequately for most applications of structural fill. See Section 8.6 for import fill specifications (aggregate base, aggregate subbase, sandy Gravels and Decomposed Granite).

Structural Fill Placement. All structural fill shall be placed in horizontal lifts not exceeding 8 inches loose thickness (less, if necessary to obtain proper compaction) for heavy compaction equipment and four inches for light and hand-operated equipment. Each lift must be compacted by mechanical means to a minimum of 98 percent of the maximum dry density, as determined by ASTM Test Method D-698 (Standard Proctor).

Beneath Footings. Structural fill placed beneath footings or other structural elements must extend beyond all sides of such elements a distance equal to at least ½ the total depth of the structural fill beneath the structural element in question for vertical support (i.e. for 2 feet of structural fill beneath footings, extend the fill at least 1 foot past all edges of the footing).

To facilitate the earthwork and compaction process, the earthwork contractor shall place and compact fill materials at or slightly above their optimum moisture content. If fill soils are too high on the wet side of optimum, they can be dried by continuous windrowing and aeration or by intermixing lime or Portland Cement to absorb excess moisture and improve soil properties. If soils become dry during the summer months, a water truck should be available to help keep the moisture content at or near optimum during compaction operations.

Fill Placement Observation and Testing Methods. The required construction monitoring of the structural fill utilizing standard nuclear density gauge testing and standard laboratory compaction curves (ASTM D-698 specified) is applicable to materials 2-inch size and smaller. Larger (2½" or above) jaw-run "shale", crushed rock or larger broken decomposed granite (DG) do not yield consistent results with this type of testing. The high percentage of rock particles greater than ¾'s of an inch in these materials causes laboratory and field density test results to be erratic and does not provide an adequate representation of the density achieved. Therefore, construction specifications for this type of material typically specify method of placement and compaction coupled with visual observation during the placement and compaction operations of lifts, instead of nuclear density testing.

Observation of Fill Placement. For these larger rock materials, we recommend the 8-inch lift (after being "worked in") be compacted by a minimum of 3 passes with a heavy vibratory roller. One "pass" is defined as the roller moving across an area once in both directions. **Note:** Much of the work on this seismic upgrade project will likely consist of narrower (2 to 4 feet wide) strip footings. Therefore, a hoe pack will likely be needed. The placement and compaction must be observed by our representative. After compaction, as specified above, is completed the entire area shall be proofrolled with a loaded dump truck to verify density has been achieved (if a truck can fit). *All areas which exhibit movement or compression of the rock material more than ¼ inch, under proofrolling, must be reworked or removed and replaced as specified above. When a* *proofroll is not feasible, then observing the hoepak work the top of the lift, with heavy down pressure will be sufficient for density verification.*

Nuclear Density Testing of Fill. Field density testing by nuclear density gage would be adequate for verifying compaction of 2-inch to ¾-inch minus crushed base rock, expansive clay and silt soils, Decomposed Granite and other materials 2 inches or smaller in size. Therefore, typical % compaction specifications would suffice. Testing must be accomplished in a systematic manner on all lifts as they are placed. Testing only the upper lifts is not adequate.

8.2.2 Non-Structural Fill

Any waste soil, organic strippings or other deleterious soil (such as wet or dried out expansive clay) would be considered non-structural fill. These materials may make reasonable landscape soils and lawn topsoil material. This material may be placed in landscape areas and waste soil areas such as berms with slopes at 3.0H:1.0V or flatter. It must not be placed under footings or other structural members. It is recommended that when these soils are used they be given a moderate level of compaction (92 to 94 percent) to help seal them from surface water.

8.3 FOUNDATION SUPPORT

During the site investigation, borings encountered somewhat expansive, soft, clayey silt in the upper 2 feet. Medium dense, clayey and silty sand and gravel were then encountered. The upper soft clayey silt soils will provide poor footing support and must be removed beneath all foundations. Conventional spread footings on compacted crushed rock over the medium dense silty/clayey sand and gravel soils are adequate for the proposed retrofits. Therefore, foundations must be founded on at least 12-inch layer of compacted structural fill which extends to a depth of 4 feet below the surface.

Footings on Compacted Crushed Rock

1. Excavate footings a minimum of 12 inches below bottom of footing, removing all clayey silt and penetrating into the native, silty/clayey Gravel and Sand or silty Sand and Gravel.
2. All areas over-excavated beneath footings must extend beyond all edges of the footing a distance equal to at least ½ the depth of the structural fill to be placed (i.e., 2 feet deep over-ex and rock fill requires the width to be 1-foot wider on all sides of the footing).
3. Redensify the footing subgrade disturbed during excavation.
4. Place a woven geotextile support fabric (ACF 180 or equivalent) on the compacted subgrade; pull tight.
5. Backfill with at least 12 inches of compacted crushed rock structural fill compacted to at least 98% of ASTM D-698.
6. Footings placed on the crushed rock as listed above may be designed for a bearing pressure of 2,000 pounds per square foot. This may be increased by 33% for transitory wind and seismic loading.

7. The base of spread footings shall be buried a minimum of 16-inches below finish grade in order to provide lateral support and frost protection.
8. We recommend minimum lateral dimensions of 16 inches for continuous load bearing footings and 24 inches for isolated piers constructed in this manner.

Anticipated Settlements. For properly constructed foundations founded on redensified and structural fill covered native soils, we anticipate maximum total and differential settlement to be less than approximately 3/4-inch and 1/2-inch, respectively.

Foundation Drains. We typically recommend all footings be installed with a footing drain to intercept groundwater seepage. Footing drains consisting of a rigid, smooth-wall perforated pipe surrounded by drain rock (one side and above), all wrapped in a non-woven geotextile fabric and should be placed adjacent to the footings. This is addressed more fully later in this report (Section 8.5). Please see Figure 3.

8.4 LATERAL LOAD RESISTANCE

8.4.1 Foundation Members

Lateral loads exerted upon these members can be resisted by passive pressure acting on buried portions of the foundations and other buried structures and by friction between the bottom of structural elements and the underlying soil.

We recommend the use of passive equivalent fluid pressures of the following values for portions of the structure and foundations embedded into the native soils.

- Native Soils (below 24") 250 pcf
- Dense Compacted Crushed Rock (below 18") 450 pcf

A coefficient of friction of 0.55 can be used for elements poured neat against angular crushed rock structural fill. These should be reduced to 0.20 for areas over a vapor barrier or 0.35 over native soils.

8.4.2 Buttress or Thrust Block

Concrete buttresses or thrust blocks may also be used for lateral resistance. These consist of an embedded concrete deadman block of reinforced concrete that can resist loads in all directions. These can be designed utilizing the lateral bearing capacities provided below.

Lateral Bearing Capacity

Depth	Capacity (ksf)
1½ to > 3 feet	0.5 up to 8 kips
3 to > 5 feet	1.0 up to 15 kips
5 to > 8 feet	2.0 up to 25 kips

(1) These must be poured "neat" against the dense clean native excavation sidewall.
(2) These may also be formed and poured and then backfilled around with compacted crushed rock.

8.5 FOUNDATION DRAINS

All exterior foundations and embedded structures should have proper drainage.

Footing Drains. Foundation drainage should consist of a rigid smooth wall perforated pipe surrounded by at least 8 inches of drain rock on top and on one side, all wrapped in a non-woven geotextile, designed as a filter fabric (such as Mirafi 140N or equivalent). The perforated pipe should be located on the footing next to the stem wall (or beside the footing), provided this is at least 12 inches below underslab drain rock. See Figure 3.

All drains shall be tightlined and positively sloped to an approved stormwater disposal location into the public storm drain system or detention pond. **Note:** In no case shall water be collected and/or directed or discharged close to the foundations. Such improper water discharge can cause added water related problems.

We strongly recommend against connecting roof drains or surface area drains to foundation drains unless to a discharge line away from the structure. Foundation drains should consist of rigid smooth-wall perforated pipe. The rigid smooth-wall pipe can be cleaned out by means of a "roto-rooter" type system should it become plugged with sediment or fine roots. We recommend cleanouts be placed periodically by the designer to facilitate cleaning and maintenance of the drains.

8.6 MATERIAL SPECIFICATIONS

The following materials specifications shall apply to the materials as used on this project.

Aggregate Base Rock (Acceptable for Structural Fill)

- Angular Crushed Rock (¾ or 1" Minus); R=85 or greater; Well Graded (No Gaps and at least 60% retained on the No. 4 sieve)
- Exceeds the fracture, durability and sand equivalent requirements outlined in Section 00641 of the Oregon Standard Specifications for Construction
- Maximum passing the No. 200 sieve ≤ 5%
- Compacted to 98% of the maximum dry density as determined by ASTM D698 or AASHTO T-99

Aggregate Subbase Rock (Acceptable for Structural Fill)

- Angular Clean Crushed (jaw run) hard "Shale" (4" Minus Jaw-Run) or Crushed Rock (2" to 4" Minus); R=50 or greater; Angular and Reasonably Well Graded
- At Least 60% retained on the No. 4 Sieve.
- Exceeds the fracture, durability and sand equivalent requirements outlined in Section 00641 of the Oregon Standard Specifications for Construction
- Maximum passing the No. 200 sieve ≤ 10% Total; ≤ 3% Clay Size
- During wet weather; passing No. 200 sieve ≤ 5%.
- Compacted to 98% of the maximum dry density as determined by ASTM D698 or AASHTO T-99; initial lift may not attain 95% due to soft subgrade; Engineer to decide in the field.
- Care must be taken to avoid very silty subbase that will not support construction loads, especially when wet (will not meet specifications).

Decomposed Granite
- Sandy DG with little to no clay.

Embankment Fill (Acceptable for Structural Fill During Dry Weather)
- Reasonably well graded (not open work)
- Has at least 60% retained on the No. 4 sieve.
- Has no more than 30% passing No. 200 sieve.
- Passing No. 200 sieve must have less than 20% clay size.

On-Site Soil Fill
- None available.

Clean Sand
- Clean washed sand or sand and gravel, less than 1% passing No. 200.
- Gravel to be rounded or subrounded (no fracture faces), 1" or less.
- Must have less than 30% gravel by weight.

Note: Some fill materials will be difficult to nearly impossible to compact during wet weather. *The contractor must select the type of structural fill that will be able to be placed and compacted to specified conditions during the weather conditions that can take place during the construction schedule.*

Drain Rock (For drainage sections)
- Clean washed rounded or angular openwork drain rock.
- Gradation to be 1/4" and greater, sized to not move into and through perforations in the pipe.
- 1/4" to 3/4" clean crushed, 3/4" to 1" clean rounded rock, 1" to 2" clean angular rock are all acceptable.
- Clean means washed rock with NO coating of silt, clay or sand.

Note: All types may be used in all applications of drain rock that are not beneath Asphaltic Concrete paved areas. In all AC areas angular clean drain rock must be used for AC support.

Note: Drainage layer drain rock that is beneath the floor slab must be the angular clean drain rock.

Geotextile Filter Fabric
- Non-woven geotextile filter fabric for wrapping drainage sections and separation of openwork rock from sands or soils fines.
- Meet specifications as per Mirafi 140N or equivalent.
- Overlap all edges at least 24 inches (12" for drainage section envelope).
- Secure in place such that overlaps will not move during covering operation.

Geotextile Support Fabric

- Woven geotextile support fabric designed for separation of crushed rock and subgrade soil and for rock section support.
- Meet specifications as per ACF180 woven support fabric.
- Overlap edges at least 2 feet and ends at least 5 feet.
- Align roll lengthwise with direction of traffic in all drive lanes.
- Pull tight full length and keep tight during placement of crushed rock above fabric.
- Do not drive on the fabric until it is covered with rock.

Perforated Pipe

- 3", 4" or 6" rigid wall, smooth interior perforated pipe.
- Secure all joints with solvent weld glue. DO NOT use only compression push together fittings.
- Slope to drain per specifications in report or on plan sheets.
- Align perforations in the downward direction.
- Must always be placed within filter fabric wrap unless specifically specified otherwise.
- Protect from construction traffic until buried at least 2 times pipe diameter (minimum 8 inches) of angular rock fill.

Wall Sheet Drain

- Polymer sheet drain with filter fabric attached 1 or 2 sides, designed for drainage of vertical embedded foundation walls.
- For walls up to 10 feet tall. Must meet specifications as for American Wick Drain's AMERDRAIN 200 or 220.
- Install and splice and patch per manufacturer's recommendations.
- Install with fabric side towards the backfill.
- Attach to wall per manufacturer's recommendations.
- Extend down wall all the way to bottom of drainage section around perforated pipe.
- Protect from damage when backfilling with crushed rock larger than 2-inch minus.
- Repair all damaged areas prior to final backfill.

These materials shall be used on this project as specified in this report and on project plans or specifications.

NOTE: DEVIATIONS FROM SPECIFIED MATERIALS MUST BE APPROVED IN WRITING BY THE GEOTECHNICAL ENGINEER, OWNER AND OWNER'S OTHER CONSULTANTS/DESIGN ENGINEERS PRIOR TO USE AT THE SITE.

9.0 ADDITIONAL SERVICES AND LIMITATIONS

9.1 ADDITIONAL SERVICES

We should review construction plans and specifications for this project as they are being developed. In addition, The Galli Group 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. Additionally, to observe compliance with the intent of our recommendations, design concepts, and the plans and specifications, all construction operations dealing with earthwork, foundations and rock placement and compaction should be observed by a representative from The Galli Group.

For this project, we anticipate additional services could include the following:

- Review of final construction plans and specifications for compliance with geotechnical recommendations.
- Possible project team meetings to clarify issues and proceed smoothly into and through the construction process.
- Observation of onsite excavations to verify stability is acceptable.
- Observation and/or testing of overexcavation, subgrade preparation, structural fill placement and compaction, subdrains and site drainage.
- Periodic construction field reports, as requested by the client and required by the building department.

We would provide these additional services on a time-and-expense basis in accordance with our current Standard Fee Schedule and General Conditions at the time of construction. If we are not retained to provide these services we cannot be held responsible for the decisions by others or for geotechnical related issues in the constructed product that we have not verified.

9.2 LIMITATIONS

The analyses, conclusions and recommendations contained in this report are based on site conditions and assumed development plans as they existed at the time of the study, and assume soils and groundwater conditions exposed and observed in the borings during our investigation are representative of soils and groundwater conditions throughout the site. If during construction, subsurface conditions or assumed design information is found to be different, we should be advised at once so that we can review this report and reconsider our recommendations in light of the changed conditions. If there is a significant lapse of time (5 years) between submission of this report and the start of work at the site, if the project is changed, or if conditions have changed due to acts of God or construction at or adjacent to the site, it is recommended that this report be reviewed in light of the changed conditions and/or time lapse.

This report was prepared for the use of the School District and their design and construction team for the design and construction of the project. It should be made available to contractors for information and factual data only. This report should not be used for contractual purposes as a warranty of site subsurface conditions. It should also not be used at other sites or for projects other than the one intended.

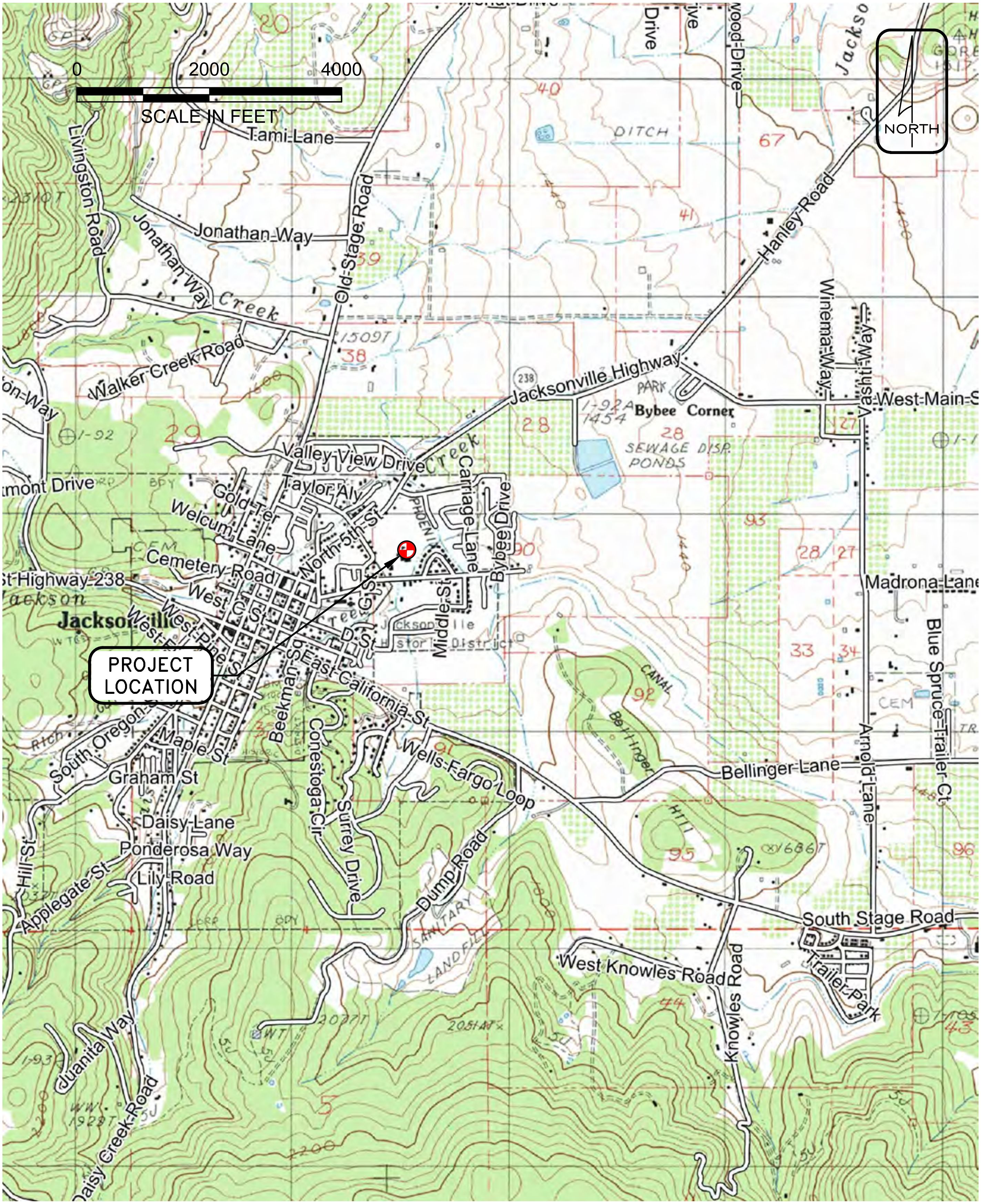
We have performed these services in accordance with generally accepted geotechnical engineering practices in southern Oregon, at the time the study was accomplished. No other warranties, either expressed or implied, are provided.

THE GALLI GROUP
GEOTECHNICAL CONSULTING

Dennis Duru, M.Sc., E.I.T.
Staff Associate

William F. Galli, P.E., G.E.
Senior Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
12162
OREGON
July 15, 1983
WILLIAM F. GALLI
EXPIRES: 6/30/21



THE GALLI GROUP GEOTECHNICAL CONSULTING 612 NW 3rd Street Grants Pass, OR 97526	VICINITY MAP JACKSONVILLE ELEMENTARY SCHOOL JACKSON COUNTY, OREGON	DATE: NOVEMBER 2019 JOB NO: 02-5758-01 REV: 11/25/2019 11:48 AM PREPARED BY: MG3/BD 5758-01 Jacksonville Elem.- 01 - Vicinity.dwg	FIGURE: 1

LEGEND

B-1 — BORING NUMBER AND APPROXIMATE LOCATION

APPROX NORTH

0 100 200

AERIAL PROVIDED BY GOOGLE IMAGES

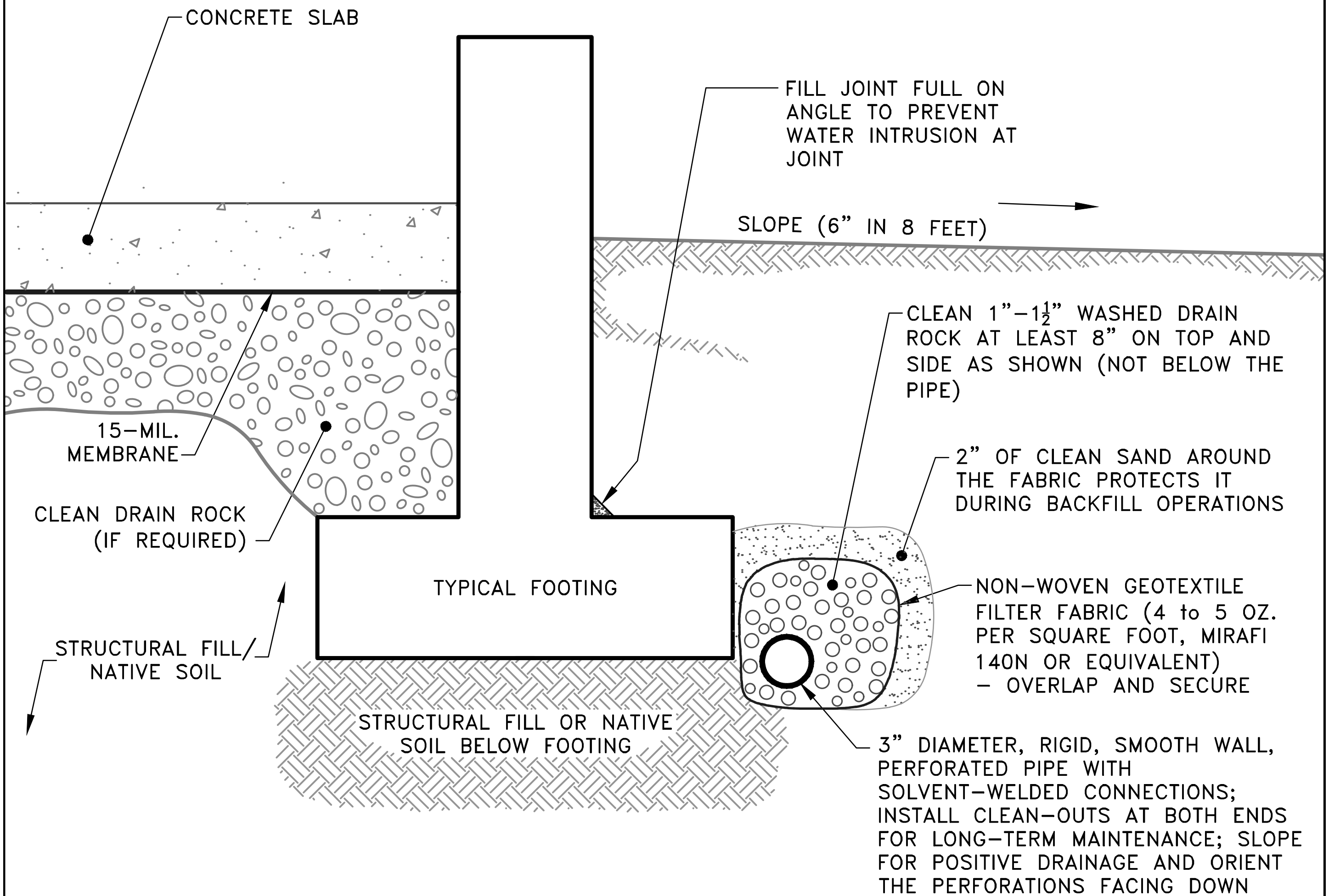
THE GALLI GROUP
GEOTECHNICAL CONSULTING
612 NW 3rd Street
Grants Pass, OR 97526

SITE PLAN WITH BORING LOCATIONS

JACKSONVILLE ELEMENTARY SCHOOL
JACKSON COUNTY, OREGON

DATE:	NOVEMBER 2019
JOB NO:	02-5758-01
REV:	11/25/2019 9:33 AM
PREPARED BY:	BD
5758-01 Jacksonville Elem.- 02 - Site Plan.dwg	

FIGURE: 2



NOTES:

(1) VAPOR BARRIER TO BE STEGO INDUSTRIES 15mil STEGO WRAP OR EQUIVALENT. OWNER MAY CHOOSE TO USE 6mil VISQUENE, UNDERSTANDING IT WILL NOT WORK AS WELL.

(2) CAPILLARY BREAK ROCK BELOW VAPOR BARRIER TO BE 1/4" TO 3/4" CLEAN CRUSHED ROCK OR EQUIVALENT.

FOR ILLUSTRATION PURPOSES ONLY – NOT FOR CONSTRUCTION
NOT TO SCALE

THE GALLI GROUP
GEOTECHNICAL CONSULTING
612 NW 3rd Street
Grants Pass, OR 97526

TYPICAL FOUNDATION DRAIN
SLAB ON GRADE FLOOR

JACKSONVILLE ELEMENTARY SCHOOL
JACKSON COUNTY, OREGON

DATE: NOVEMBER 2019
JOB NO: 02-5758-01
REV: 11/25/2019 9:33 AM
PREPARED BY: BD
5758-01 Jacksonville Elem.- 03 - Slabog.dwg

FIGURE: 3

APPENDIX A

BORING LOGS

THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

BORING LOG
B-1

Project: JACKSONVILLE ELEMENTARY SCHOOL
Client: MEDFORD SCHOOL DISTRICT
Location: SEE FIGURE 2
Driller: TGG (AARON, KEN)
Drill Rig: ATV MOUNTED, 4" DIA SSA
Depth To Water> Initial :

Project No.: 02-5758-01
Date: 11/6/2019
Elevation:
Logged By: DENNIS DURU
At Completion :

Graphic Log	USCS	Description	Depth	Sample No. and Type	NMC	Standard Penetration Test N	CURVE (10, 30, 50)
	OL	Grass rootzone.	0 – 0.2				
	OH/OL	Soft, dark brown, clayey SILT; moist.	0.2 – 2.5	S-1 (1.5)	17%		
	GC	Medium dense, brown, clayey Gravel; moist	2.5 – 4.0				
	GP/SP	Very dense, brown, Sand and Gravel; damp.	4.0 – 5.0	S-1	3.3%	68	68
		Bottom of Boring at 5.0 Feet. No Free Groundwater Encountered.					

Depth scale: 0, 1.5, 3, 4.5, 6, 7.5, 9, 10.5

Legend of Samplers: Grab sample SPT sample Shelby tube sample

This information pertains only to this boring and should not be interpreted as being indicative of the site.

THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

BORING LOG
B-2

Project: JACKSONVILLE ELEMENTARY SCHOOL
Client: MEDFORD SCHOOL DISTRICT
Location: SEE FIGURE 2
Driller: TGG (AARON, KEN)
Drill Rig: ATV MOUNTED, 4" DIA SSA
Depth To Water> Initial : At Completion :

Project No.: 02-5758-01
Date: 11/6/2019
Elevation:
Logged By: DENNIS DURU

USCS	Description	Depth	Sample No. and Type	NMC	Standard Penetration Test N
OL	Grass rootzone.	0.1			
OH/OL	Soft, dark brown, clayey SILT; moist.	1.0			
GC	Medium dense, brown, clayey Gravel; moist	3.0			
GP/SP	Very dense, gray, Sand and Gravel; damp.	5.0	S-1 (SPT)	7.6%	30
	Bottom of Boring at 4.0 Feet. No Free Groundwater Encountered.				

Depth scale: 0, 1.5, 3, 4.5, 6, 7.5, 9, 10.5

Standard Penetration Test CURVE: 10, 30, 50

Legend of Samplers: Grab sample SPT sample Shelby tube sample

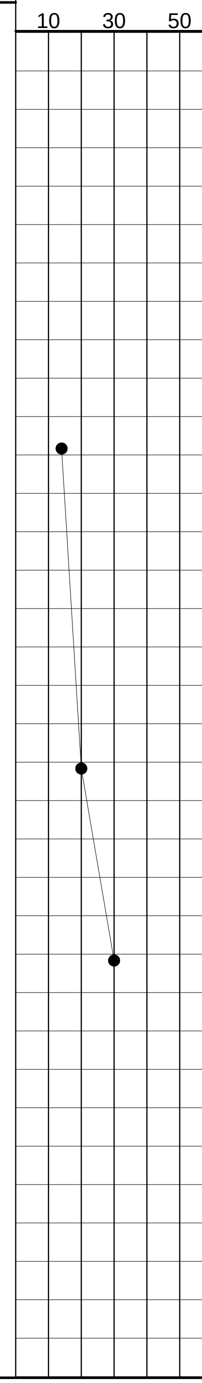
This information pertains only to this boring and should not be interpreted as being indicative of the site.

THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

BORING LOG
B-3

Project: JACKSONVILLE ELEMENTARY SCHOOL
Project No.: 02-5758-01
Client: MEDFORD SCHOOL DISTRICT
Date: 11/6/2019
Location: SEE FIGURE 2
Elevation:
Driller: TGG (AARON, KEN)
Logged By: DENNIS DURU
Drill Rig: ATV MOUNTED, 4" DIA SSA
Depth To Water> Initial :
At Completion :

Graphic Log	USCS	Description	Depth	Sample No. and Type	NMC	Standard Penetration Test N	CURVE
	OL	Grass rootzone.	0.1				
	GM/SM	Medium dense, brown, silty Sand and Gravel; damp.	6.0	S-1	5.0	14	
				S-2	7.3% 4.7%	20	
	GP/SP	Very dense, gray, Sand and Gravel; dry.	8.0	S-3	5.5%	30	
		Bottom of Boring at 8.0 Feet. No Free Groundwater Encountered.					



Depth scale: 0, 1.5, 3, 4.5, 6, 7.5, 9, 10.5

Legend of Samplers: Grab sample SPT sample Shelby tube sample

This information pertains only to this boring and should not be interpreted as being indicative of the site.

APPENDIX B

LABORATORY TEST RESULTS

Expansion Index Worksheet (ASTM D-4829)

Client:	Medford School District
Project	Jacksonville Elementary
Job No:	02-5758-01
Test Date:	11/27/2019
Sample Location:	B-1, S-1 @ 0.2' to 2.5'
Sample Date:	11/6/2019
Description of Soil:	Dark brown, sandy Clay

Expansion Index measured (EIm):

$EI_m = \Delta H / H_{orig} * 1000$

begin dial :	0.0173
end dial:	0.0512
EI_m:	**34**

Weight of ring (g):	191.5
Wt. Wet sample in ring(g):	591.2
Sample Wet Weight (g):	399.64
Sample Length (in.):	1
Sample Diameter (in.):	4.01
Volume of sample (ft^3):	0.007309
Sample Unit Wt. (PCF):	**120.4**
Sample Dry Unit Wt. (PCF):	**109.6**

Saturation (S):

$S=(SG)(w)\gamma d)/(SG)*62.4)-\gamma d$

SG:	2.7
γd:	109.6
%*w* :	9.9
S=	**50**

As prepared for testing:

can no.	26
wet weight of soil + can (g)	263.10
dry weight of soil + can (g)	245.88
weight of can (g)	71.89
weight of dry soil (g)	173.99
weight of water (g)	17.22
moisture content (% of dry weight)	9.9

EI_{50} Calculation:

$EI_{50} = EIm - (50-Sm)*[(65+EIm)/(220-Sm)]$

EI_M	34
S	50
EI_{50} =	**34**

After testing:

can no.	4-4
wet weight of soil + can (g)	496.40
dry weight of soil + can (g)	428.12
weight of can (g)	110.10
weight of dry soil (g)	318.02
weight of water (g)	68.28
moisture content (% of dry weight)	21.5

#4 + (dry wt.)	149.1
#4 - (dry wt.)	2058.2
% Passing #4 Sieve =	**93.2**

Tested By: Lyn Chand

THE GALLI GROUP
Geotechnical Consulting

PRELIMINARY SEISMIC RETROFIT STUDY
JEFFERSON ELEMENTARY SCHOOL
MEDFORD, OREGON

For: Ron Havniear
Medford School District
815 South Oakdale Ave
Medford, OR 97501

c/o Stephen L. Chase, Project Manager
ZCS Engineering & Architecture
127 NW D Street
Grants Pass, Oregon 97526

By: **THE GALLI GROUP**
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Grants Pass, OR 97526
(541) 955-1611

02-6018-01
June 30, 2021

Table of Contents

PRELIMINARY SEISMIC RETROFIT STUDY
JEFFERSON ELEMENTARY SCHOOL
MEDFORD, OREGON

1.0 INTRODUCTION

This report presents results of our geotechnical and geological evaluation of the Jefferson Elementary School for a potential Seismic Retrofit of the school structures. The subject school is located at 333 Holmes Avenue, on the southwest corner of Holmes Avenue and South Holly Street, in Medford Oregon. Please see Figure 1, Vicinity Map, for a more precise location.

The purpose of our investigation and this report was to accomplish a limited site surface and subsurface evaluation (one boring) and conduct a planning level seismic risk assessment (office studies) in order to provide preliminary geotechnical and geologic information and evaluate the likelihood and consequences of geotechnical/geologic related seismic failures, including liquefaction and landslide potential during the design seismic event, for consideration of the potential seismic retrofit.

2.0 SITE AND PROJECT DESCRIPTION

The site is currently occupied by a functioning elementary school, which consists of approximately 6 structures, connected via covered walkways or with direct connections. The structures are surrounded by play fields, access roads, parking lots, walkways and open space. The site is relatively flat to mildly sloping with undeveloped portions of the site consisting of well-maintained lawn and scattered trees.

We understand the district is conducting a preliminary review to determine the level of seismic retrofit needed for the structures on this campus. Their review will largely be based on the evaluation of the potential geologic hazards (such as liquefaction) provided in this report, and an evaluation of the potential structural damage to these facilities associated with the design seismic event. The findings will also likely be used to determine if grant funding will be pursued to complete the seismic retrofit work.

3.0 FIELD EXPLORATION

On June 18, 2021, Associate staff, Dennis Duru, M.Sc., E.I.T and our drilling crew, visited the site to accomplish the subsurface investigation. One (1) exploratory boring was drilled in the planter area on the north side of the school buildings (Building "A"). A utility locate was completed prior to our investigation and our representative coordinated with school personnel to identify the field exploration location away from the marked and known utility locations. See Figure 2, Site Plan, for a layout of the site and the location of the boring. The boring was drilled with our ATV-mounted solid stem auger drill rig and penetrated to depth of 15.0 feet before encountering the very dense, weathered mudstone bedrock. Upon completion, the boring was backfilled with drill spoils.

Standard Penetration Testing (SPT) was accomplished during drilling, as part of the exploratory boring. This entails driving a 1½-inch diameter steel split spoon sampler by dropping a 140-pound weight for a 30-inch drop. The total number of blows it takes to drive the sampler the last 12 inches of an 18-inch drive is called the SPT N-value. These can be correlated with density and soil strength parameters from testing on thousands of other projects.

Our representative identified the final exploration location, logged subsurface soils and water conditions and obtained soil samples for transport to our laboratory. Visual classifications of the soils were made in the field and are presented in the Boring Log in Appendix A, at the end of this report.

4.0 LABORATORY TESTING

Moisture content tests were accomplished on soil samples obtained by Standard Penetration Testing. No other tests were accomplished.

5.0 SUBSURFACE CONDITIONS

5.1 SOIL

The subsurface consisted of medium stiff to stiff, brown Clay to approximately 7.5 feet. This was underlain by stiff Clay, with some gravel and sand to the depth of 15.0 feet. This was then underlain by hard, brown, cemented Clay which is the top of the mudstone bedrock of the Hornbrook Formation.

Please see more specific soils information in the Boring Logs in Appendix A. Please note that the soils are shown as distinct layers in the Boring Logs while in nature they may change more gradually. Soils conditions may also change somewhat between the locations investigated.

5.2 GROUNDWATER

Generally, the soils encountered were moist to saturated. Groundwater was encountered in the boring at 11.0 feet below ground surface.

6.0 GEOLOGIC OR SEISMIC INDUCED HAZARDS

Summary of Site Geology and Seismicity. The project area is located in the Medford East 30x60 minute USGS topographic quadrangle (see Vicinity Map, Figure 1). Mapped geologic units at the project area consist primarily of Alluvial Fan deposits and bedrock members of the Hornbrook Formation (Wiley et al, 2011). The Marine Sandstone, Siltstone and Conglomerate members are the mapped bedrock unit at the project (Wiley et al, 2011; OGDC-6, 2015). Weathered conglomerate Sandstone was encountered in the auger boring at this site at relatively shallow depths of 5 feet below ground surface.

The project site is in proximity to several zones of active seismicity. The region is affected by the Cascadia Subduction Zone (CSZ), an active subduction zone off the Oregon coast considered capable of Magnitude 8.5 or greater earthquakes. Average recurrence intervals for such great earthquakes, as determined by recent investigations, range between 300-600 years. The last "great" earthquake was interpreted to be approximately 300 years ago. The CSZ is the main seismic event for consideration for this seismic retrofit.

The project area has an additional tectonic source from earthquakes occurring along active Basin and Range faults as close as 50 kilometers to the southeast. This region has produced numerous earthquakes, including significant events near Klamath Falls and Warner Valley. Such events occur generally once every one to two decades.

Flood Hazard. The site is not within a 100-year floodplain of any river or streams according to the FEMA flood mapping.

Landslides/Slope Instability. The project site is located within the southern edge of a mapped Quaternary landside (Qls). This mapped feature is present on the state landslide database (Statewide Landslide Information Database for Oregon; SLIDO, 2017). Based upon the published mapping, general geomorphology, review of 2-foot contours generated from Lidar datasets (Dogami, 2021) and aerial photos (Google Earth, 2021), as well as from the subsurface data obtained in this investigation, the mapped landslide in the project area is interpreted to be a relatively thin alluvial fan deposit of material originating upslope.

No recent movement or damage to structures has been associated with this feature in readily available published accounts or our general geotechnical and geologic knowledge of the area. It is therefore assumed this is an inactive “older” deposit. Therefore, in our professional opinion, based on the information from our limited exploration data, the risk of damage due to natural slope instability at this site is considered low. However, any proposed manmade cut or fill slopes should only be made following the recommendations from a detailed geotechnical investigation and report.

Liquefaction and Lateral Spread. The project is underlain by Clay soils with varying gravel and sand content over the mudstone bedrock. Soils with clay content and densities similar to the ones observed during our limited exploration have not been known to liquefy in a seismic event. Therefore, liquefaction and lateral spread is considered to be a low to very low potential hazard for this site. See more in Section 7.0 of this report.

Expansive Soils. The upper soils within the subsurface consist of Clay with some gravel. The upper portion of the subsurface is mapped as the soil unit 33A (Coker clay) according to the NRCS-Web soil survey. This soil consists of fat clay (CH), with liquid limits between 65 and 75. Given the liquid limit range, the upper soils would likely have high expansion potential based on our experience with soils with similar properties.

These expansive soils can have adverse impacts on foundations and all manner of concrete flatwork if the building design does not account for such soils. We recommend that each proposed building site (with clayey soils present) have the soil subgrade examined and tested (Expansion Index) prior to final design and construction. In that way, locations that have expansive soils will have the retrofit structure(s) and drainage designed accordingly.

Construction over expansive soils generally requires embedment of footings to 3 to 4 feet (final depth to be verified after EI testing) below the exterior grade and placement of floor slabs over at least 24 inches or more of compacted rock fill in order to mitigate expansion potential of the underlying soil subgrades. Maintaining the moisture content in the soil to keep it in a moist and fully swelled condition prior to being covered is also critical to proper performance of the structures.

Note: The geotechnical engineer must provide site specific laboratory testing and remedial design recommendations on projects that have potentially expansive, gravelly Clay, clayey Silt or Clay soils present.

Ground Rupture. No Quaternary faults were identified at the project site. Therefore, the risk of damage at the site due to ground rupture is considered very low.

Ground Shaking. Project structures including foundations and retaining walls must be designed for the potential for very strong ground shaking during the anticipated seismic event. The peak site modified horizontal acceleration (PGA_M), is 0.386g. This is based on the Site Class D designation, determined for the project from subsurface drilling and evaluation of SPT data. The PGA_M value can be used with an appropriate seismic coefficient in pseudo static analysis for design of the seismic upgrades.

Seismic Ground Amplification or Resonance. No unusually hazardous amplification or resonance effects on seismic waves have been associated with soil/bedrock subsurface conditions in the project area.

Tsunami and Seiche. The site is approximately 80 miles inland from the coast, and not subject to tsunami hazard. The site is not located adjacent to a large lake or body of water, and therefore, not subject to seiche hazard.

7.0 LIQUEFACTION EVALUATION

The liquefaction phenomenon occurs in saturated, loose (low density, uncompacted or poorly compacted), cohesionless soils. When loose, cohesionless soils are saturated which is the case when the soil is below the water table, then water fills the soil pores. In response to compression when a load is applied to the soil, the water increases in pressure and attempts to migrate towards zones of low pressure. However, if the applied load is rapid and large enough, or if it is repeated many times (cyclic loading) like in an earthquake, such that there is not enough time for the water to dissipate before the next cycle of loading is applied, then the water pressure may build up to a degree where they become greater than the grain-to-grain contact stresses of the soil. The grain-to-grain contact stresses are the source of the shear strength that supports structures foundations and overburden soils in these soil structures. This buildup of excess pore water pressure results in total or partial loss of the soil strength, and the soil may be observed to flow like a liquid, hence "liquefaction". At this point, the soil will lose all its shear strength, be deformed, and will not be able to support structures.

The site is underlain medium stiff to hard clay. Groundwater was observed in the boring at the 11.0 feet deep. <u>The conditions for liquefaction to occur were not observed at this site during our limited subsurface investigation</u>. The medium stiff and hard clay soils will not liquefy during a seismic event. Therefore, possibility of liquefaction that could adversely affect the site is very low.

<u>Therefore, in our professional opinion, the site conditions found in the boring will not result in wide spread liquefaction that will have significant adverse impacts on the structures during a seismic event.</u>

8.0 CONCLUSIONS

In our professional opinion, based on our field investigation, office review and previous work in the area, the soils conditions at the site are suitable for a conventional seismic retrofit. However, expansive clay would likely be an issue on the site and must be mitigated. More detailed geotechnical investigation and laboratory testing will be necessary to provide mitigation recommendations for the deep expansive clay observed. In our opinion, this school site is not susceptible to large scale liquefaction that will adversely impact the structure. However, the subsurface expansive clay soils encountered at the site could cause shrink/swell related problems for retrofitted structures, if not properly mitigated.

Additional borings around the structures on this site could possibly encounter sandy soils zones that may liquefy. However, these are likely to be moderate to small in size and should not adversely impact the overall site stability or increase the potential damage to the school structures.

If a full seismic retrofit geotechnical design report is needed, additional tasks to be accomplished would be as follows:

1. 2 or 3 additional borings.
2. Laboratory testing for determining expansive index, strength and settlement characteristics of the site soils.
3. Evaluation of data for developing design parameters.
4. Ground motion hazard analysis to determine spectral acceleration parameter for the structure.

These could be used to provide a full scale Seismic Retrofit Design Report.

8.1 LIMITATIONS

The analyses, conclusions and recommendations contained in this report are based on site conditions as they existed at the time of the study, and assume soils, rock and groundwater conditions exposed and observed in the boring during our investigation are representative of soils and groundwater conditions throughout the site. If during construction, subsurface conditions or assumed design information is found to be different, we should be advised at once so that we can review this report and reconsider our recommendations in light of the changed conditions. If there is a significant lapse of time (5 years) between submission of this report and the start of work at the site, if the project is changed, or if conditions have changed due to acts of God or construction at or adjacent to the site, it is recommended that this report be reviewed in light of the changed conditions and/or time lapse.

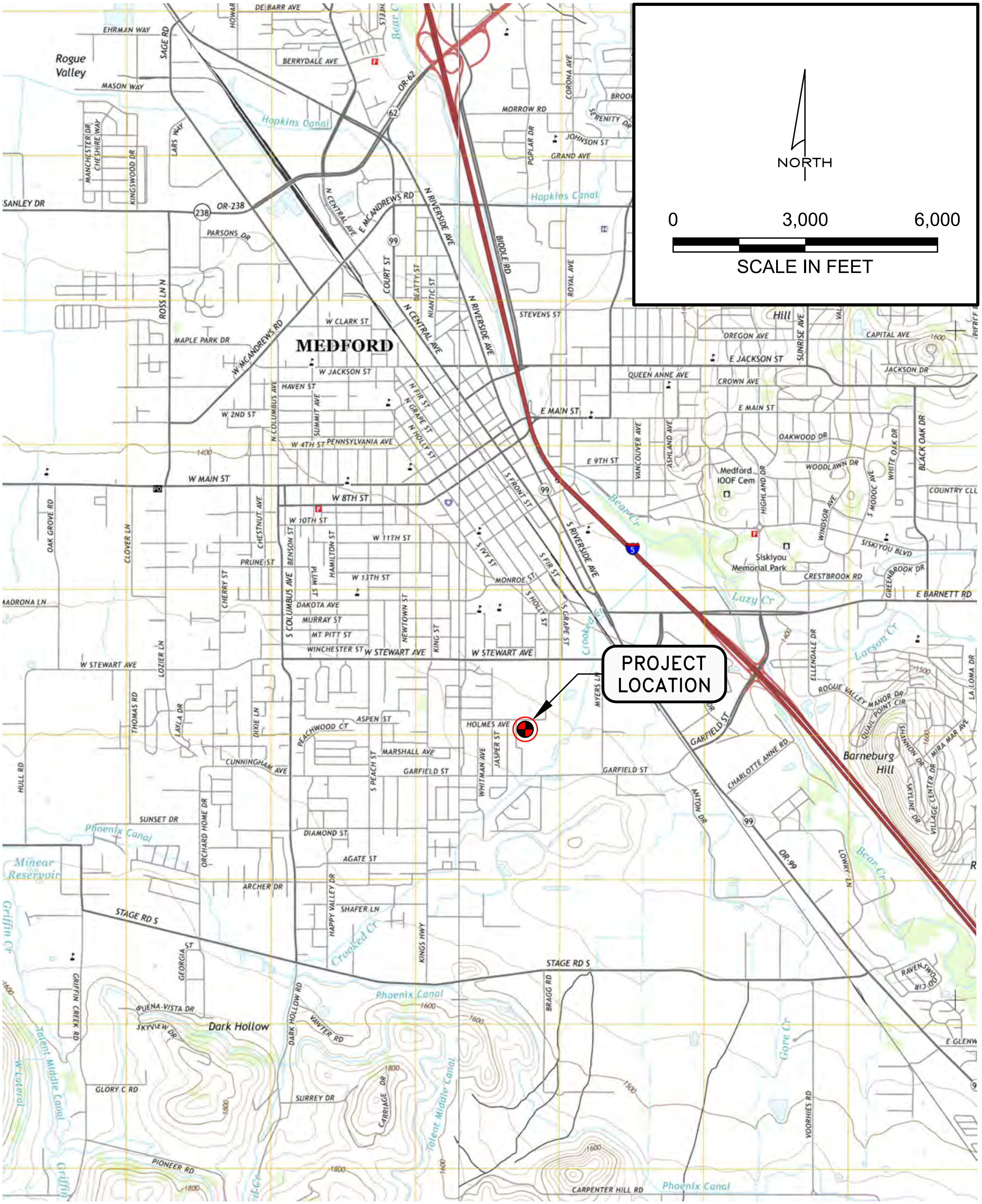
This report was prepared for the use of the School District and their design team for evaluating the need for a full scale Seismic Retrofit evaluation and report. It should be made available to contractors for information and factual data only. This report should not be used for contractual purposes as a warranty of site subsurface conditions. It should also not be used at other sites or for projects other than the one intended.

We have performed these services in accordance with generally accepted geotechnical engineering and professional geology practices in southern Oregon, at the time the study was accomplished. No other warranties, either expressed or implied, are provided.

THE GALLI GROUP
GEOTECHNICAL CONSULTING

Dennis Duru, M.S., E.I.T.
Staff Associate

Melvin Galli III, P.E.
Principal Engineer



THE GALLI GROUP
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612 NW 3rd Street
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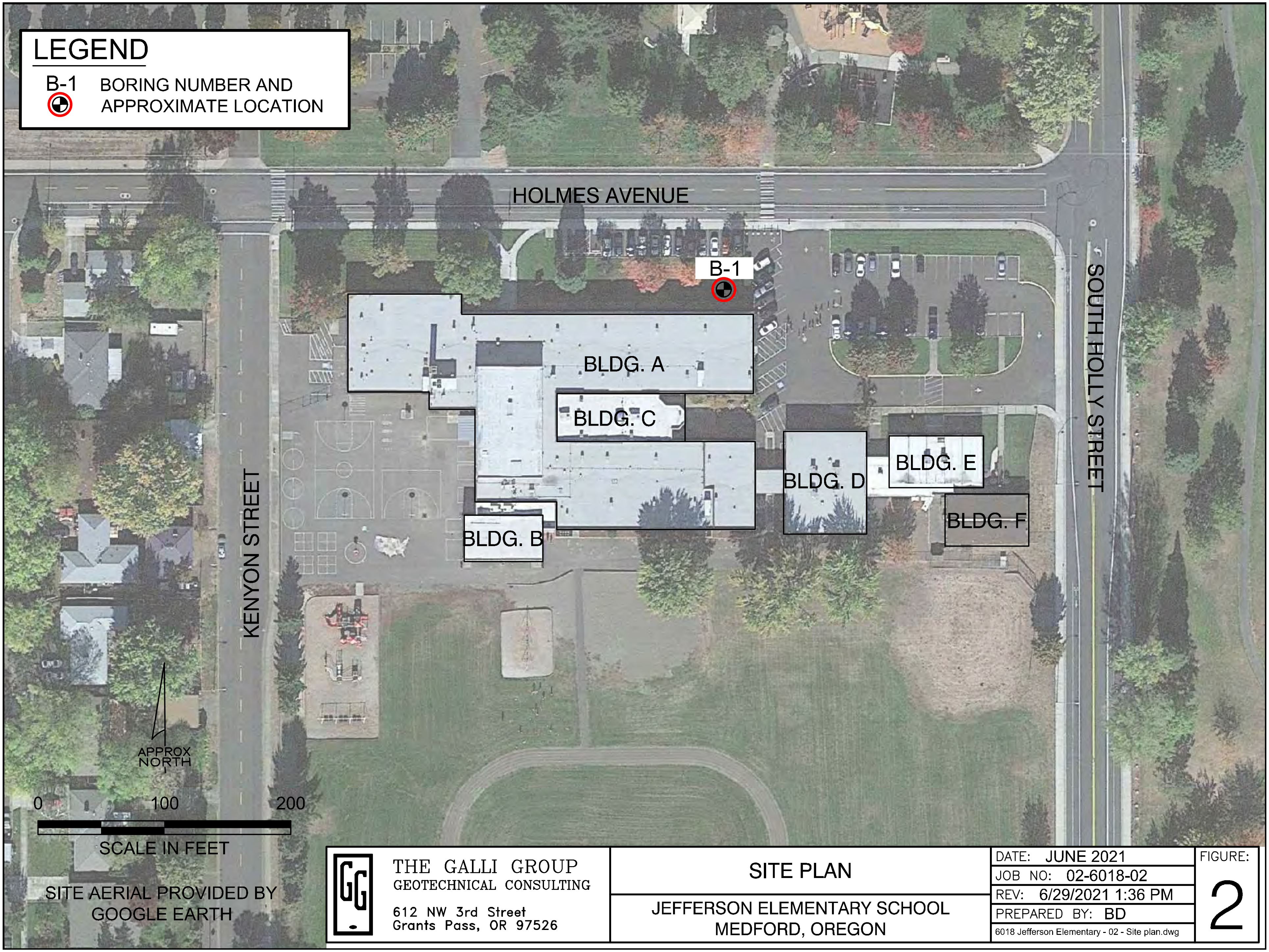
VICINITY MAP

JEFFERSON ELEMENTARY SCHOOL
MEDFORD, OREGON

DATE: JUNE 2021
JOB NO: 02-6018-02
REV: 6/29/2021 1:28 PM
PREPARED BY: BD
6018 Jefferson Elementary - 01 - Vicinity.dwg

FIGURE: 1



THE GALLI GROUP
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612 NW 3rd Street
Grants Pass, OR 97526

SITE PLAN

JEFFERSON ELEMENTARY SCHOOL
MEDFORD, OREGON

DATE: JUNE 2021
JOB NO: 02-6018-02
REV: 6/29/2021 1:36 PM
PREPARED BY: BD
6018 Jefferson Elementary - 02 - Site plan.dwg

FIGURE: 2

APPENDIX A

BORING LOGS

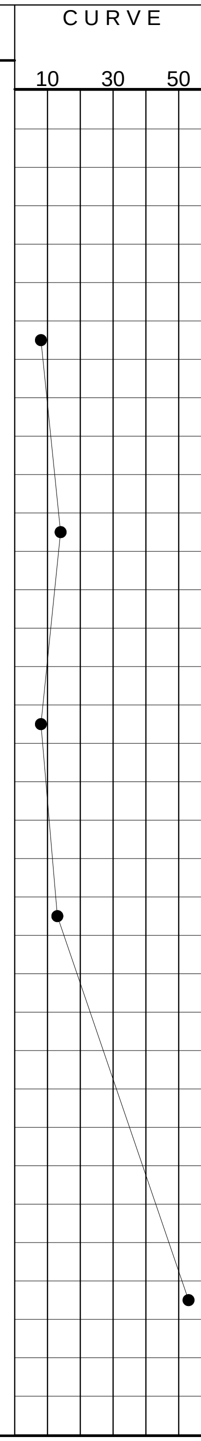
THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

BORING LOG
B-1

Project: Jefferson Elementary School
Client: Medford School District
Location: Planter area north of Building A (see Site Plan)
Driller: TGG (Ken, Nick)
Drill Rig: ATV mounted Rig, 4" diameter SSA
Depth To Water> Initial : 11.0

Project No.: 02-6018-01
Date: 6/18/2021
Elevation:
Logged By: Dennis Duru
At Completion : 11.0

Graphic Log	USCS	Description	Depth	Sample No. and Type	NMC	Standard Penetration Test N
	OL	Grass Rootzone. (0.25)	0			
	OH	Medium stiff, black, Clay; moist.				
			2.5			
				S-1	26%	8
	CL	Stiff, brown Clay; trace gravel, moist. (5.0)	5			
				S-2	17%	14
	CL	Medium stiff to stiff, brown, Clay; some gravel and sand, wet to saturated. (7.5)	7.5			
				S-3	24%	8
			10			
▼				S-4	20%	13
			12.5			
	CL	Hard, brown, cemented Clay; saturated. (15.0)	15			
				S-5	29%	53
		(16.5) Bottom of boring at 16.5 feet. Static groundwater at 11.0.	17.5			



Legend of Samplers: Grab sample SPT sample Shelby tube sample

This information pertains only to this boring and should not be interpreted as being indicative of the site.

THE GALLI GROUP
Geotechnical Consulting

PRELIMINARY SEISMIC RETROFIT REPORT
KENNEDY ELEMENTARY SCHOOL
MEDFORD, OREGON

For: Ron Havniear
Facilities Manager
Medford School District
815 S. Oakdale Avenue
Medford, OR 97504

By: **THE GALLI GROUP**
612 NW Third Street
Grants Pass, OR 97526
(541) 955-1611

02-5759-01
November 14, 2019

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PRELIMINARY SEISMIC RETROFIT REPORT
KENNEDY ELEMENTARY SCHOOL
MEDFORD, OREGON

REPORT SUMMARY

The single boring encountered 3 feet of clayey gravel, 3 feet of medium dense to dense, coarse Sand and then weathered Siltstone. No water was encountered. Based on this boring:

- Upper clays are mildly to moderately expansive.
- There will be no liquefaction.
- There will be no lateral spread.
- Dense, coarse Sand and Siltstone will provide excellent bearing strata.

We found no significant adverse geotechnically related or geological conditions at the site.

1.0 INTRODUCTION

This report presents results of our geotechnical and geology evaluation of the site for a preliminary review for a possible seismic retrofit to the Kennedy Elementary School in Medford, Oregon. The subject property is located on N. Keene Way Drive near its intersection with Amy Street. This site is mildly sloping. Please see Figure 1, Vicinity Map, for a more precise site location.

The purpose of this investigation and report was to evaluate the site surface and subsurface conditions with one boring and conduct office studies in order to provide preliminary geotechnical and geologic information for consideration of the potential seismic retrofit.

2.0 SITE AND PROJECT DESCRIPTION

The site is currently occupied by a functioning elementary school, which apparently consists of numerous, partially connected structures, sports field, drive lanes and parking lots.

We understand the project to consist of reviewing whether a complete Seismic Retrofit to the school will be required. Also, to determine if a grant for such work would be considered worthy of approval.

3.0 FIELD EXPLORATION

On November 8, 2019, Dennis Duru, Associate Engineer, M.Sc., E.I.T., and our drilling crew, visited the site to accomplish the subsurface investigation. A total of one exploratory boring was drilled to a depth of 6.5 feet near the SW corner of the school at the location shown on Figure 2, Site Plan. The drilling was accomplished with our ATV-mounted solid stem auger drill rig. The hole was refilled after drilling with drill spoils. The area was left reasonably clean of most soil debris.

4.0 LABORATORY TESTING

One sample of the clay content in the upper 3 feet was tested for expansion potential. Test results indicate that the soil has high expansion potential with a tested expansion index value (EI_{50}) of 104. Adverse impacts of these expansive soils (change in volume with change in moisture content) will be mitigated in the design section of this report. Lab test results are attached in Appendix B.

5.0 SUBSURFACE CONDITIONS

5.1 SOIL

The boring encountered a 3-foot layer of clayey Gravel and then 3 feet of medium dense to dense, coarse Sand. Below this was a 3-foot layer of medium stiff to very stiff, clayey Sand. Then weathered, cemented silt and clay (weathered Siltstone) was encountered. Please see more specific soils information in the Boring Logs in Appendix A. Please note that the soils are shown as distinct layers in the Boring Logs while in nature they may change more gradually. Soils conditions may also change somewhat between the locations investigated.

5.2 GROUNDWATER

Generally, the soils encountered were mostly moist. No groundwater was encountered in the boring. Due to the dense underlying soil/weathered rock, the water levels perch on this rock and could rise to within 3 feet of the surface during the wetter months of most years. The upper site soils are likely to disturb somewhat easily during wetter periods of the year.

6.0 GEOLOGIC OR SEISMIC INDUCED HAZARDS

Flood Hazard. The site is not near rivers or streams. There is no risk of flooding.

Landslides/Slope Instability. There are no steep slopes close to the site. Therefore, there is no possibility of slope failure, rock fall or slide run out damage.

Liquefaction and Lateral Spread. The project is underlain by mixtures of silt, clay and dense sand mixtures, some with gravels. Therefore, liquefaction and lateral spread is <u>not</u> considered to be a potential hazard at the site.

Expansive Soils. The project has 3 feet of gravels with clay and then clayey Sand. Lab testing produced an EI value of 104 on the clay portion, which indicates these soils are highly expansive.

Ground Rupture. No Quaternary faults were identified at the project site. Therefore, the risk of damage at the site due to ground rupture is considered very low.

Ground Shaking. The design of the structures shall be designed for the design PGA of 0.25g. See the following Seismic Design table.

Seismic Ground Amplification or Resonance. No hazardous amplification or resonance effects from seismic waves have been associated with the soil subsurface conditions in the project area. Some amplification is possible. However, the design parameters in the Seismic Design table should account for such added loading. The risk of damage at the site from unexpectedly severe shaking due to seismic wave amplification is low.

Tsunami and Seiche. The site is approximately 80 miles inland from the coast, and not subject to tsunami hazard. The site is not located adjacent a large lake or body of water, and, therefore, not subject to seiche hazard.

7.0 CONCLUSIONS

In our professional opinion, based on our field investigation, and office review, and previous work in the area, the soils conditions at the site are suitable for a proposed seismic retrofit, provided the recommendations of our report are incorporated in the design and construction of the project. Crushed rock backfill over the very stiff, clayey Sand will provide good support for new or enlarged foundations and buttresses.

CAUTION: Expansive Soils Are Present at the Site.

8.0 GEOTECHNICAL RECOMMENDATIONS

The clayey Sand zone encountered must be covered with at least 24 inches of compacted crushed rock to support footings. Onsite clayey soil must be removed from beneath footings and may be used as structural fill <u>in landscape areas only</u>.

The subject site has reasonably good soils for support of the structure. The following sections provide methods and data for proper design of the seismic retrofit items.

8.1 SITE PREPARATION AND GRADING

This site has structures that may require structural and foundation upgrades for the seismic upgrade. Therefore, normal methods of demolition, debris removal, clearing, grubbing, stripping for organic removal and subgrade soil preparation will apply in areas of the work.

8.1.1 Manmade Fill & Debris Considerations

All old fill and debris encountered during construction must be removed. Soil that is clear of debris could be used in landscape berms. All other debris or debris laden soil must be wasted off site or used in landscape berms. The full extent of any waste fill removal (if any) will be determined during site stripping operations.

8.1.2 Clearing, Grubbing and Stripping

The site area that will be used for the work must be stripped and cleared of all vegetation, sod and organic topsoil. Additional stripping (or excavations) will most likely be required to remove root balls from bushes (very little of this is expected). The stripped materials removed shall be hauled from the site or stockpiled for use in landscape areas only (such as landscape mounds). This material must <u>not</u> be used in structural fill or trench backfill.

Holes or depressions resulting from the removal of underground obstructions that extend below the finish subgrade and will be beneath support items shall be cleared of all loose material and dished to provide access for compaction equipment. These areas shall then be backfilled and compacted to grade with structural fill, as described later in this report. It is recommended that grubbing and stripping of the site, old fill removal and compaction of depressions below finish subgrade, be observed and/or decided by the geotechnical engineer or his representative from The Galli Group.

8.2 STRUCTURAL FILL PLACEMENT AND COMPACTION

8.2.1 Beneath Structures

Structural fill is defined as any fill placed and compacted to specified densities and used in areas that will be under foundations, or other structure support items. It appears that new footings and buttresses could have structural fill below, beside and above them. The subgrade needs to be prepared properly and the fill must be placed and compacted correctly for proper long-term performance.

Structural Fill Materials. Ideally, and particularly for wet weather construction, structural fill must consist of a free-draining granular material (non-expansive) with a maximum particle size of six inches. The material shall be reasonably well-graded with less than 5 percent fines (silt and clay size passing the No. 200 mesh sieve). During dry weather, any organic-free, non-expansive, will less than 12% passing the No. 200 sieve, compactable granular material, meeting the maximum size criteria, is typically acceptable for this purpose. Locally available crushed rock, jaw-run crushed "shale" (low-grade rock) and good quality Sandy Decomposed Granite [DG]) have performed adequately for most applications of structural fill. See Section 8.6 for import fill specifications (aggregate base, aggregate subbase, sandy Gravels and Decomposed Granite).

Structural Fill Placement. All structural fill should be placed in horizontal lifts not exceeding 8 inches loose thickness (less, if necessary to obtain proper compaction) for heavy compaction equipment and four inches for light and hand-operated equipment. Each lift must be compacted by mechanical means to a minimum of 98 percent of the maximum dry density, as determined by ASTM Test Method D-698 (Standard Proctor).

Beneath Footings. Structural fill placed beneath footings or other structural elements must extend beyond all sides of such elements a distance equal to at least ½ the total depth of the structural fill beneath the structural element in question for vertical support (i.e. for 2 feet of structural fill beneath footings, extend the fill at least 1 foot past all edges of the footing).

To facilitate the earthwork and compaction process, the earthwork contractor shall place and compact fill materials at or slightly above their optimum moisture content. If fill soils are too high on the wet side of optimum, they can be dried by continuous windrowing and aeration or by intermixing lime or Portland Cement to absorb excess moisture and improve soil properties. If soils become dry during the summer months, a water truck should be available to help keep the moisture content at or near optimum during compaction operations.

Fill Placement Observation and Testing Methods. The required construction monitoring of the structural fill utilizing standard nuclear density gauge testing and standard laboratory compaction curves (ASTM D-698 specified) is applicable to materials 2-inch size and smaller. Larger (2½" or above) jaw-run "shale", crushed rock or larger broken decomposed granite (DG) do not yield consistent results with this type of testing. The high percentage of rock particles greater than ¾'s of an inch in these materials causes laboratory and field density test results to be erratic and does not provide an adequate representation of the density achieved. Therefore, construction specifications for this type of material typically specify method of placement and compaction coupled with visual observation during the placement and compaction operations of lifts, instead of nuclear density testing.

Observation of Fill Placement. For these larger rock materials, we recommend the 8-inch lift (after being "worked in") be compacted by a minimum of 3 passes with a heavy vibratory roller. One "pass" is defined as the roller moving across an area once in both directions. **Note:** Much of the work on this seismic upgrade project will likely consist of narrower (2 to 4 feet) strip footing. Therefore, a hoe pack will likely be needed. The placement and compaction must be observed by our representative. After compaction, as specified above, is completed the entire area shall be proofrolled with a loaded dump truck to verify density has been achieved (if a truck can fit). *All areas which exhibit movement or compression of the rock material more than ¼ inch, under proofrolling, must be reworked or removed and replaced as specified above. When a proofroll is not feasible, then observing the hoepak work the top of the lift, with heavy down pressure, will be sufficient for density verification.*

Nuclear Density Testing of Fill. Field density testing by nuclear density gage would be adequate for verifying compaction of 2-inch to ¾-inch minus crushed base rock, expansive clay and silt soils, Decomposed Granite and other materials 2 inches or smaller in size. Therefore, typical % compaction specifications would suffice. Testing must be accomplished in a systematic manner on all lifts as they are placed. Testing only the upper lifts is not adequate.

8.2.2 Non-Structural Fill

Any waste soil, organic strippings or other deleterious soil (such as wet or dried out expansive clay) would be considered non-structural fill. These materials may make reasonable landscape soils and lawn topsoil material. This material may be placed in landscape areas and waste soil areas such as berms with slopes at 3.0H:1.0V or flatter. It must not be placed under footings or other structural members. It is recommended that when these soils are used, they be given a moderate level of compaction (92 to 94 percent) to help seal them from surface water.

8.3 FOUNDATION SUPPORT

During the site investigation, we encountered clayey Gravels and clayey Sand. Conventional spread footings on compacted crushed rock founded on the clayey Sand unit are adequate for the proposed retrofit. These soils will provide adequate support for the footings. Foundations must be founded at a depth of 24 inches with 24 inches of compacted structural fill which extends into the clayey Sand.

Footings on Compacted Crushed Rock

1. Excavate footings into the undisturbed clayey Sand, at least 4 feet below the surface.
2. All areas over-excavated beneath footings must extend beyond all edges of the footing a distance equal to at least ½ the depth of the structural fill to be placed (i.e., 2 feet deep over-ex and rock fill requires the width to be 1 foot wider on each side of the footing).
3. Redensify the footing subgrade disturbed during excavation.
4. Backfill with at least 24 inches of compacted crushed rock structural fill compacted to at least 98% of ASTM D-698.
5. Footings placed on the crushed rock as listed above may be designed for a bearing pressure of 2,500 pounds per square foot. This may be increased by 33% for transitory wind and seismic loading.
6. The base of spread footings shall be buried a minimum of 16-inches below finish grade in order to provide lateral support and frost protection.
7. We recommend minimum lateral dimensions of 16 inches for continuous load bearing footings and 24 inches for isolated piers constructed in this manner.

Anticipated Settlements. For properly constructed foundations founded on redensified and structural fill covered native soils, we anticipate maximum total and differential settlement to be less than approximately ¾-inch and ⅝-inch, respectively.

Foundation Drains. We typically recommend all footings be installed with a footing drain to intercept groundwater seepage. Footing drains consisting of a rigid, smooth-wall perforated pipe surrounded by drain rock (sides and above), all wrapped in a non-woven geotextile fabric and should be placed adjacent to the footings. This is addressed more fully later in this report (Section 8.5). Please see Figure 3.

8.4 LATERAL LOAD RESISTANCE

8.4.1 Foundation Members

Lateral loads exerted upon these members can be resisted by passive pressure acting on buried portions of the foundations and other buried structures and by friction between the bottom of structural elements and the underlying soil.

We recommend the use of passive equivalent fluid pressures of the following values for portions of the structure and foundations embedded into the native soils.

• Native Soils (below 12”)	200 pcf
• Clayey Sand	250 pcf
• Dense Compacted Crushed Rock (below 12”)	450 pcf

A coefficient of friction of 0.55 can be used for elements poured neat against angular crushed rock structural fill. These should be reduced to 0.20 for areas over a vapor barrier or 0.35 over native clayey Gravel and 0.35 over clayey Sands.

8.4.2 Buttress or Thrust Block

Concrete buttresses or thrust blocks may also be used for lateral resistance. These consist of an embedded concrete deadman block of reinforced concrete that can resist loads in all directions. These can be designed utilizing the lateral bearing capacities provided below.

Lateral Bearing Capacity

Depth	Capacity (ksf)
1½ to > 3 feet 3 to > 6 feet	0.5 ksf up to 8 kips 1.0 ksf up to 25 kips

(1) These must be poured "neat" against the dense clean native excavation sidewall.
(2) These may also be formed and poured and then backfilled around with compacted crushed rock structural fill.

8.5 FOUNDATION DRAINS

All exterior foundations and embedded structures should have proper drainage.

Footing Drains. Foundation drainage should consist of a rigid smooth wall perforated pipe surrounded by at least 8 inches of drain rock on top and sides, all wrapped in a non-woven geotextile designed as a filter fabric (such as Mirafi 140N or equivalent). The perforated pipe should be located on the footing next to the stem wall (or beside the footing), provided this is at least 12 inches below underslab drain rock. Please see Figure 3.

All drains shall be tightlined and positively sloped to an approved stormwater disposal location into the public storm drain system or detention pond. **Note:** In no case shall water be collected and/or directed or discharged close to the foundations. Such improper water discharge can cause added water related problems.

We strongly recommend <u>against</u> connecting roof drains or surface area drains to wall, foundation or floor subdrains unless to a discharge line away from the structure. Foundation drains should consist of rigid smooth-wall perforated pipe. The rigid smooth-wall pipe can be cleaned out by means of a "roto-rooter" type system should it become plugged with sediment or fine roots. We recommend cleanouts be placed periodically by the designer to facilitate cleaning and maintenance of the drains.

8.6 MATERIAL SPECIFICATIONS

The following materials specifications shall apply to the materials as used on this project.

Aggregate Base Rock (Acceptable for Structural Fill)

- Angular Crushed Rock (¾ or 1" Minus); R=85 or greater; Well Graded (No Gaps and at least 60% retained on the No. 4 sieve)
- Exceeds the fracture, durability and sand equivalent requirements outlined in Section 00641 of the Oregon Standard Specifications for Construction
- Maximum passing the No. 200 sieve ≤ 5%
- Compacted to 98% of the maximum dry density as determined by ASTM D698 or AASHTO T-99

Aggregate Subbase Rock (Acceptable for Structural Fill)

- Angular Clean Crushed (jaw run) hard "Shale" (4" Minus Jaw-Run) or Crushed Rock (2" to 4" Minus); R=50 or greater; Angular and Reasonably Well Graded
- At Least 60% retained on the No. 4 Sieve.
- Exceeds the fracture, durability and sand equivalent requirements outlined in Section 00641 of the Oregon Standard Specifications for Construction
- Maximum passing the No. 200 sieve ≤ 10% Total; ≤ 3% Clay Size
- <u>During wet weather; passing No. 200 sieve ≤ 5%.</u>
- Compacted to 98% of the maximum dry density as determined by ASTM D698 or AASHTO T-99; initial lift may not attain 95% due to soft subgrade; Engineer to decide in the field.
- Care must be taken to avoid very silty subbase that will not support construction loads, especially when wet (will not meet specifications).

Embankment Fill (Acceptable for Structural Fill During <u>Dry</u> Weather)

- Reasonably well graded (not open work)
- Has at least 60% retained on the No. 4 sieve.
- Has no more than 30% passing No. 200 sieve.
- Passing No. 200 sieve must have less than 20% clay size.

On-Site Soil Fill (Acceptable as Specified in Report)

- None available.

Clean Sand

- Clean washed sand or sand and gravel, less than 1% passing No. 200.
- Gravel to be rounded or subrounded (no fracture faces), 1" or less.
- Must have less than 30% gravel by weight.

Note: Some fill materials will be difficult to nearly impossible to compact during wet weather. *The contractor <u>must</u> select the type of structural fill that will be able to be placed and compacted to specified conditions during the weather conditions that can take place during the construction schedule.*

Drain Rock (For drainage sections)

- Clean washed rounded or angular openwork drain rock.
- Gradation to be 1/4" and greater, sized to not move into and through perforations in the pipe.
- 1/4" to 3/4" clean crushed, 3/4" to 1" clean rounded rock, 1" to 2" clean angular rock are all acceptable.
- Clean means washed rock with NO coating of silt, clay or sand.

Note: All types may be used in all applications of drain rock that are not beneath Asphaltic Concrete paved areas. In all AC areas angular clean drain rock must be used for AC support.

Note: Drainage layer drain rock that is beneath the floor slab must be the angular clean drain rock.

Geotextile Filter Fabric

- Non-woven geotextile filter fabric for wrapping drainage sections and separation of openwork rock from sands or soils fines.
- Meet specifications as per Mirafi 140N or equivalent.
- Overlap all edges at least 24 inches (12" for drainage section envelope).
- Secure in place such that overlaps will not move during covering operation.

Geotextile Support Fabric

- Woven geotextile support fabric designed for separation of crushed rock and subgrade soil and for rock section support.
- Meet specifications as per ACF180 woven support fabric.
- Overlap edges at least 2 feet and ends at least 5 feet.
- Align roll lengthwise with direction of traffic in all drive lanes.
- Pull tight full length and keep tight during placement of crushed rock above fabric.
- Do not drive on the fabric until it is covered with rock.

Perforated Pipe

- 3", 4" or 6" rigid wall, smooth interior perforated pipe.
- Secure all joints with solvent weld glue. DO NOT use only compression push together fittings.
- Slope to drain per specifications in report or on plan sheets.
- Align perforations in the downward direction.
- Must always be placed within filter fabric wrap unless specifically specified otherwise.
- Protect from construction traffic until buried at least 2 times pipe diameter (minimum 8 inches) of angular rock fill.

Wall Sheet Drain

- Polymer sheet drain with filter fabric attached 1 or 2 sides, designed for drainage of vertical embedded foundation walls.
- For walls up to 10 feet tall. Must meet specifications as for American Wick Drain's AMERDRAIN 200 or 220.
- Install and splice and patch per manufacturer's recommendations.
- Install with fabric side towards the backfill.
- Attach to wall per manufacturer's recommendations.
- Extend down wall all the way to bottom of drainage section around perforated pipe.
- Protect from damage when backfilling with crushed rock larger than 2-inch minus.
- Repair all damaged areas prior to final backfill.

These materials shall be used on this project as specified in this report and on project plans or specifications.

NOTE: DEVIATIONS FROM SPECIFIED MATERIALS MUST BE APPROVED IN WRITING BY THE GEOTECHNICAL ENGINEER, OWNER AND OWNER'S OTHER CONSULTANTS/DESIGN ENGINEERS PRIOR TO USE AT THE SITE.

9.0 ADDITIONAL SERVICES AND LIMITATIONS

9.1 ADDITIONAL SERVICES

We should review construction plans and specifications for this project as they are being developed. In addition, The Galli Group 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. Additionally, to observe compliance with the intent of our recommendations, design concepts, and the plans and specifications, all construction operations dealing with earthwork, foundations and rock placement and compaction should be observed by a representative from The Galli Group.

For this project, we anticipate additional services could include the following:

- Final complete Seismic Retrofit study and report if the project goes ahead.

We would provide these additional services on a time-and-expense basis in accordance with our current Standard Fee Schedule and General Conditions at the time of construction. If we are not retained to provide these services, we cannot be held responsible for the decisions by others or for geotechnical related issues in the constructed product that we have not verified.

9.2 LIMITATIONS

The analyses, conclusions and recommendations contained in this report are based on-site conditions and assumed development plans as they existed at the time of the study, and assume soils, rock and groundwater conditions exposed and observed in the boring during our investigation are representative of soils and groundwater conditions throughout the site. If during construction, subsurface conditions or assumed design information is found to be different, we should be advised at once so that we can review this report and reconsider our recommendations in light of the changed conditions. If there is a significant lapse of time (5 years) between submission of this report and the start of work at the site, if the project is changed, or if conditions have changed due to acts of God or construction at or adjacent to the site, it is recommended that this report be reviewed in light of the changed conditions and/or time lapse.

This report was prepared for the use of the School District and their design team for the evaluation and preliminary assessment of the project. It should be made available to contractors for information and factual data only. This report should not be used for contractual purposes as a warranty of site subsurface conditions. It should also not be used at other sites or for projects other than the one intended.

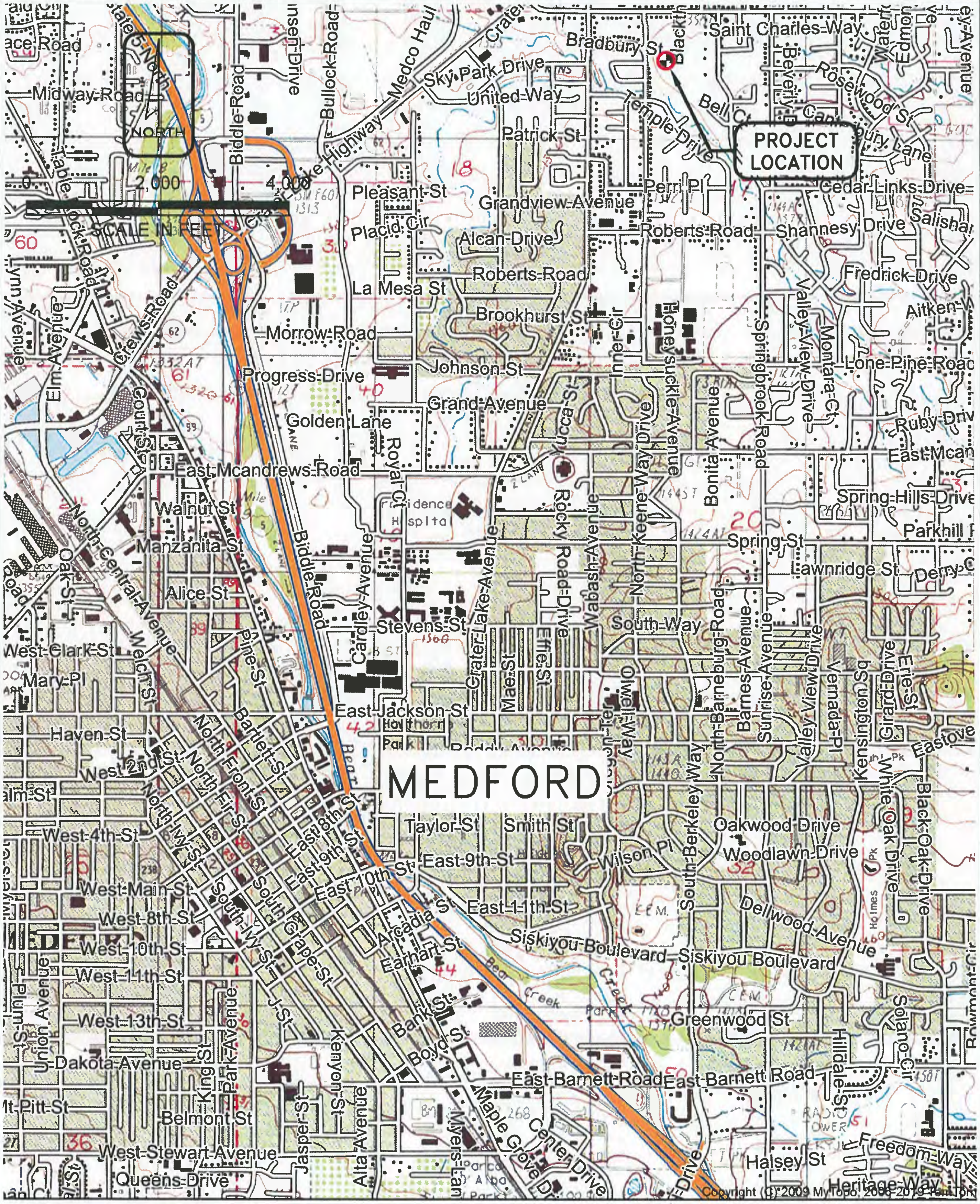
We have performed these services in accordance with generally accepted geotechnical engineering practices in southern Oregon, at the time the study was accomplished. No other warranties, either expressed or implied, are provided.

THE GALLI GROUP
GEOTECHNICAL CONSULTING

William F. Galli

William F. Galli, P.E., G.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
12162
OREGON
July 15, 1983
WILLIAM F. GALLI
EXPIRES: 06/30/21



THE GALLI GROUP GEOTECHNICAL CONSULTING 612 NW 3rd Street Grants Pass, OR 97526	VICINITY MAP KENNEDY ELEMENTARY SCHOOL MEDFORD, OREGON	DATE: NOVEMBER 2019 JOB NO: 02-5759-1 REV: 11/11/2019 12:09 PM PREPARED BY: JDT 5759-1 Kennedy Elem Sch -1-Vicinity dwg	FIGURE: 1

LEGEND

B-1	BORING NUMBER AND APPROXIMATE LOCATION

APPROX NORTH

NOT TO SCALE

SITE IMAGE PROVIDED BY GOOGLE EARTH

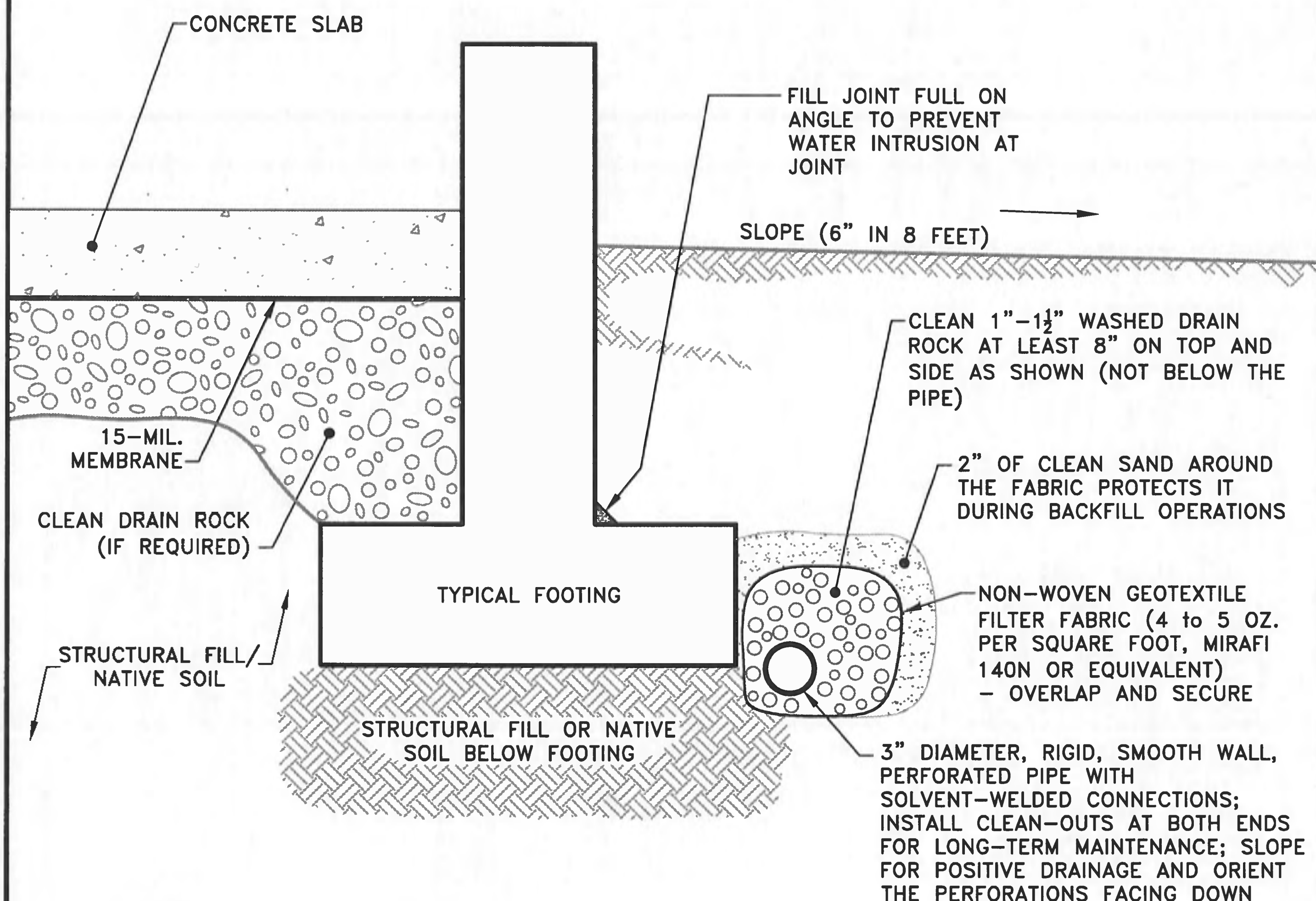
THE GALLI GROUP
GEOTECHNICAL CONSULTING
612 NW 3rd Street
Grants Pass, OR 97526

SITE PLAN WITH BORING LOCATIONS

KENNEDY ELEMENTARY SCHOOL
MEDFORD, OREGON

DATE: NOVEMBER 2019
JOB NO: 02-5759-01
REV:11/11/2019 12:41 PM
PREPARED BY: JDT
5759-1 Kennedy Elem Sch -2- Site Plan ZI.dwg

FIGURE: 2



NOTES:

(1) VAPOR BARRIER TO BE STEGO INDUSTRIES 15mil STEGO WRAP OR EQUIVALENT. OWNER MAY CHOOSE TO USE 6mil VISQUENE, UNDERSTANDING IT WILL NOT WORK AS WELL.

(2) CAPILLARY BREAK ROCK BELOW VAPOR BARRIER TO BE 1/4" TO 3/4" CLEAN CRUSHED ROCK OR EQUIVALENT.

FOR ILLUSTRATION PURPOSES ONLY – NOT FOR CONSTRUCTION
NOT TO SCALE

THE GALLI GROUP GEOTECHNICAL CONSULTING 612 NW 3rd Street Grants Pass, OR 97526	TYPICAL FOUNDATION DRAIN SLAB ON GRADE FLOOR KENNEDY ELEMENTARY SCHOOL MEDFORD, OREGON	DATE: NOVEMBER 2019 JOB NO: 02-5759-01 REV: 11/11/2019 12:33 PM PREPARED BY: JDT 5759-1 Kennedy Elem Sch -3- found drain-slab.dwg	FIGURE: 3

APPENDIX A

BORING LOG

BORING LOG

Please note that the soil descriptions given below are representative of how the field representative observed and classified them at the time of drilling. However, these should not be used as a guarantee of subsurface conditions across the site. Any interpretation or estimates made by others based on these logs, is done at their risk.

B-1

0.0 – 0.2	Topsoil/Rootzone
0.2 – 3.0	Medium dense, dark brown, clayey Gravel; moist.
3.0 – 6.0	Very stiff, light brown, clayey Sand; moist, some gravel.
	SPT 2.5’ to 4.0’; 9/12/12; N = 24
	SPT 5.0’ to 6.5’; 5/9/26; N 35
6.0 – 6.5	Very dense, light brown, cemented Silt and Clay; dry, weathered Siltstone.

No Free Groundwater or Seepage Observed.
Bottom of Boring at 6.5 feet

APPENDIX B

LABORATORY TEST RESULTS

Expansion Index Worksheet (ASTM D-4829)

THE GALLI GROUP
Geotechnical Consulting

Client:	Medford School District
Project	Kennedy Elementary
Job No:	02-5759-01
Test Date:	11/12/2019
Sample Location:	B-1/S-2 @ 5.0' to 6.0'
Sample Date:	11/8/2019
Description of Soil:	Yellow brown, sandy, silty Clay

Expansion Index measured (EIm):

$EI_m = \Delta H / H_{orig} * 1000$

begin dial :	0.0018
end dial:	0.0685
EI_m:	**67**

Weight of ring (g):	191.6
Wt. Wet sample in ring(g):	607.7
Sample Wet Weight (g):	416.13
Sample Length (in.):	1
Sample Diameter (in.):	4.01
Volume of sample (ft^3):	0.007309
Sample Unit Wt. (PCF):	**125.4**
Sample Dry Unit Wt. (PCF):	**104.8**

Saturation (S):

$S=(SG)(w)\gamma d)/(SG)*62.4)-\gamma d$

SG:	2.7
γd:	104.8
%*w* :	19.6
S=	**87**

As prepared for testing:

can no.	D-3
wet weight of soil + can (g)	406.84
dry weight of soil + can (g)	361.24
weight of can (g)	128.88
weight of dry soil (g)	232.36
weight of water (g)	45.60
moisture content (% of dry weight)	19.6

EI_{50} Calculation:

$EI_{50} = EIm - (50-Sm)*[(65+EIm)/(220-Sm)]$

EI_M	67
S	87
EI_{50} =	**104**

After testing:

can no.	555
wet weight of soil + can (g)	621.31
dry weight of soil + can (g)	519.32
weight of can (g)	179.58
weight of dry soil (g)	339.74
weight of water (g)	101.99
moisture content (% of dry weight)	30.0

#4 + (dry wt.)	47.6
#4 - (dry wt.)	1568.7

% Passing #4 Sieve = 97.1

Tested By: Lyn Chand

THE GALLI GROUP
Geotechnical Consulting

PRELIMINARY SEISMIC RETROFIT STUDY
LONE PINE ELEMENTARY SCHOOL
MEDFORD, OREGON

For: Ron Havniear
Medford School District
815 South Oakdale Ave
Medford, OR 97501

c/o Stephen L. Chase, Project Manager
ZCS Engineering & Architecture
127 NW D Street
Grants Pass, Oregon 97526

By: **THE GALLI GROUP**
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Grants Pass, OR 97526
(541) 955-1611

02-6008-01
June 30, 2021

Table of Contents

PRELIMINARY SEISMIC RETROFIT STUDY
LONE PINE ELEMENTARY SCHOOL
MEDFORD, OREGON

1.0 INTRODUCTION

This report presents results of our geotechnical and geological evaluation of the Lone Pine Elementary School for a potential seismic retrofit of the school structures. The subject school is located at 3158 Lone Pine Road, on the southeast corner of the intersection of Lone Pine Road and Brookdale Avenue in Medford, Oregon. Please see Figure 1, Vicinity Map, for a more precise location.

The purpose of our investigation and this report was to accomplish a limited site surface and subsurface evaluation (one boring) and conduct a planning level seismic risk assessment (office studies) in order to provide preliminary geotechnical and geologic information and evaluate the likelihood and consequences of geotechnical/geologic related seismic failures, including liquefaction and landslide potential during the design seismic event, for consideration of the potential seismic retrofit.

2.0 SITE AND PROJECT DESCRIPTION

The site is currently occupied by a functioning elementary school, which consists of 7 structures connected via covered walkways. The structures are surrounded by play fields, access roads, parking, walkways and open space. The site is mildly sloping with undeveloped portions of the site consisting of well-maintained lawn and a few trees.

We understand the district is conducting a preliminary review to determine the level of seismic retrofit needed for the structures on this campus. Their review will largely be based on the evaluation of the potential geologic hazards (such as liquefaction) provided in this report, and an evaluation of the potential structural damage to these facilities associated with the design seismic event. The findings will also likely be used to determine if grant funding will be pursued to complete the seismic retrofit work.

3.0 FIELD EXPLORATION

On June 11, 2021, Associate Engineer, Dennis Duru, M.Sc., E.I.T and our drilling crew, visited the site to accomplish the subsurface investigation. One (1) exploratory boring was drilled on the lawn area southwest of Building D. A utility locate was completed prior to our investigation and our representative coordinated with school personnel to identify the field exploration location away from the marked and known utility locations. See Figure 2, Site Plan, for a layout of the site and the location of the boring. The boring was drilled with our ATV-mounted, solid-stem auger drill rig and penetrated to depth of 5.0 feet before encountering the very dense, weathered sandstone bedrock. Upon completion, the boring was backfilled with drill spoils.

Standard Penetration Testing (SPT) was accomplished in each boring. This entails driving a 1½-inch diameter steel split spoon sampler by dropping a 140-pound weight for a 30-inch drop. The total number of blows it takes to drive the sampler the last 12 inches of an 18-inch drive is called the SPT N-value. These can be correlated with density and soil strength parameters from testing on thousands of other projects.

Our representative identified the final exploration location, logged subsurface soils and water conditions and obtained soil samples for transport to our laboratory. Visual classifications of the soils were made in the field and are presented in the Boring Logs in Appendix A, at the end of this report.

4.0 LABORATORY TESTING

Moisture content tests were accomplished on soil samples obtained by Standard Penetration Testing. No other tests were accomplished.

5.0 SUBSURFACE CONDITIONS

5.1 SOIL

The subsurface consisted of medium dense, black clayey Gravel to the depth of 4.0 feet. This was then underlain by medium dense to very dense, brown silty sandy Gravel (top of weathered Sandstone bedrock).

Please see more specific soils information in the Boring Logs in Appendix A. Please note that the soils are shown as distinct layers in the Boring Logs while in nature they may change more gradually. Soils conditions may also change somewhat between the locations investigated.

5.2 GROUNDWATER

Generally, the soils encountered were moist. No groundwater was encountered in the boring. Review of well log information shows that static groundwater levels range from between 15 and 60 feet below ground surface.

6.0 GEOLOGIC OR SEISMIC INDUCED HAZARDS

Summary of Site Geology and Seismicity. The project area is located in the Medford East 30x60 minute USGS topographic quadrangle (see Vicinity Map, Figure 1). Mapped geologic units at the project area consist primarily bedrock members of the Hornbrook Formation (Wiley et al, 2011). The Marine Sandstone, Siltstone and Conglomerate members are the mapped bedrock unit at the project (Wiley et al, 2011; OGDC-6, 2015). Weathered Sandstone was encountered in the auger boring at this site at relatively shallow depths of 4 feet below ground surface.

The project site is in proximity to several zones of active seismicity. The region is affected by the Cascadia Subduction Zone (CSZ), an active subduction zone off the Oregon coast considered capable of Magnitude 8.5 or greater earthquakes. Average recurrence intervals for such great earthquakes, as determined by recent investigations, range between 300-600 years. The last "great" earthquake was interpreted to be approximately 300 years ago. The CSZ is the main seismic event for consideration for this seismic retrofit.

The project area has an additional tectonic source from earthquakes occurring along active Basin and Range faults as close as 50 kilometers to the southeast. This region has produced numerous earthquakes, including significant events near Klamath Falls and Warner Valley. Such events occur generally once every one to two decades.

Flood Hazard. The site is not near streams or rivers. Therefore, it is not within a 100-year floodplain.

Landslides/Slope Instability. There are no slopes close to the site. Therefore, possibility of slope failure, rock fall or slide run out damage at the site is low.

Liquefaction and Lateral Spread. The project is underlain by clayey Gravel over medium dense to very dense silty Gravel in the shallow subsurface. Dense Gravel soils, similar to the soils observed during our limited exploration, have not been known to liquefy in a seismic event. From review of available data (well logs and NRCS web soil survey) as well as the result of our subsurface exploration, bedrock beneath the site is at a depth of approximately 4 feet. The very dense silty Gravel encountered in our boring is the top of the Sandstone bedrock of the Hornbrook Formation underlying the project site. Therefore, liquefaction and lateral spread is not considered to be a potential hazard for this site during the design seismic event. See more in Section 7.0 of this report.

Expansive Soils. The upper soils within the subsurface consist of clayey Gravel. Clay and gravel content could vary somewhat across the site. The clays will likely have moderate expansion potential, based on our experience with soils with similar visual properties.

These expansive soils can have adverse impacts on foundations and all manner of concrete flatwork if the building design does not account for such soils. We recommend that each proposed building site (with clayey soils present) have the soil subgrade examined and tested (Expansion Index) prior to final design and construction. In that way, locations that have expansive soils will have the retrofit structure(s) and drainage designed accordingly.

Construction over expansive soils generally requires embedment of footings to 3 to 4 feet (final depth to be verified after EI testing) below the exterior grade and placement of floor slabs over at least 24 inches or more of compacted rock fill in order to mitigate expansion potential of the underlying soil subgrades. Maintaining the moisture content in the soil to keep it in a moist and fully swelled condition prior to being covered is also critical to proper performance of the structures.

Note: The geotechnical engineer must provide site specific laboratory testing and remedial design recommendations on projects that have potentially expansive, gravelly Clay, clayey Silt or Clay soils present.

Ground Rupture. No Quaternary faults were identified at the project site. Therefore, the risk of damage at the site due to ground rupture is considered very low.

Ground Shaking. Project structures including foundations and retaining walls must be designed for the potential for very strong ground shaking during the anticipated seismic event. The peak site modified horizontal acceleration (PGA_M), is 0.347g. This is based on the Site Class C designation, determined for the project from subsurface drilling and evaluation of SPT data. The PGA_M value can be used with an appropriate seismic coefficient in pseudo static analysis for design of the seismic upgrades.

Seismic Ground Amplification or Resonance. No unusually hazardous amplification or resonance effects on seismic waves have been associated with soil/bedrock subsurface conditions in the project area.

Tsunami and Seiche. The site is approximately 80 miles inland from the coast, and not subject to tsunami hazard. The site is not located adjacent to a large lake or body of water, and therefore, not subject to seiche hazard.

7.0 LIQUEFACTION EVALUATION

The liquefaction phenomenon occurs in saturated, loose (low density, uncompacted or poorly compacted), cohesionless soils. When loose, cohesionless soils are saturated which is the case when the soil is below the water table, then water fills the soil pores. In response to compression when a load is applied to the soil, the water increases in pressure and attempts to migrate towards zones of low pressure which is usually upwards towards the ground surface. However, if the applied load is rapid and large enough, or if it is repeated many times (cyclic loading), like in an earthquake, such that there is not enough time for the water to dissipate before the next cycle of loading is applied, then the water pressure may build up to a degree where they become greater than the grain-to-grain contact stresses of the soil. The grain-to-grain contact stresses are the source of the shear strength that supports structure foundations and overburden soils in these soil structures. This buildup of excess pore water pressure results in total or partial loss of the soil strength, and the soil may be observed to flow like a liquid, hence "liquefaction". At this point, the soil will lose all its shear strength, be deformed, and will not be able to support structures.

The site is underlain by medium dense to very dense Sand and Gravel (weathered sandstone). No groundwater was observed in the boring to the depth drilled. The conditions for liquefaction to occur were not observed at this site during our limited subsurface investigation. Such soils will not undergo further densification to cause liquefaction, though the upper 3.5 feet of the soil may experience slight settlement during the design seismic event. Also, water was not observed during our investigation and could be below 15 feet deep according to reviewed well data. Therefore, liquefaction cannot take place.

Therefore, in our professional opinion, the site conditions found in the boring will not result in wide spread liquefaction that will have significant adverse impacts on the structures during a seismic event.

8.0 CONCLUSIONS

In our professional opinion, based on our field investigation, office review and previous work in the area, the soils conditions at the site are suitable for a conventional seismic retrofit. Crushed rock structural fill over the silty Sand and Gravel will provide adequate support of new foundations, grade beams and/or buttresses. In our opinion, this school site is not susceptible to large scale liquefaction that will cause a significant adverse impact to the school structures.

If a full seismic retrofit geotechnical design report is needed, additional tasks to be accomplished would be as followed:

1. 2 or 3 additional borings.
2. Laboratory testing for determining strength and settlement characteristics of the site soils.
3. Evaluation of data for developing design parameters.

These could be used to provide a full scale Seismic Retrofit Design Report.

8.1 LIMITATIONS

The analyses, conclusions and recommendations contained in this report are based on site conditions as they existed at the time of the study, and assume soils, rock and groundwater conditions exposed and observed in the boring during our investigation are representative of soils and groundwater conditions throughout the site. If during construction, subsurface conditions or assumed design information is found to be different, we should be advised at once so that we can review this report and reconsider our recommendations in light of the changed conditions. If there is a significant lapse of time (5 years) between submission of this report and the start of work at the site, if the project is changed, or if conditions have changed due to acts of God or construction at or adjacent to the site, it is recommended that this report be reviewed in light of the changed conditions and/or time lapse.

This report was prepared for the use of the School District and their design team for evaluating the need for a full scale Seismic Retrofit evaluation and report. It should be made available to contractors for information and factual data only. This report should not be used for contractual purposes as a warranty of site subsurface conditions. It should also not be used at other sites or for projects other than the one intended.

We have performed these services in accordance with generally accepted geotechnical engineering and professional geology practices in southern Oregon, at the time the study was accomplished. No other warranties, either expressed or implied, are provided.

THE GALLI GROUP
GEOTECHNICAL CONSULTING

Dennis Duru, M.S., E.I.T.
Staff Associate

Melvin Galli III, P.E.
Principal Engineer

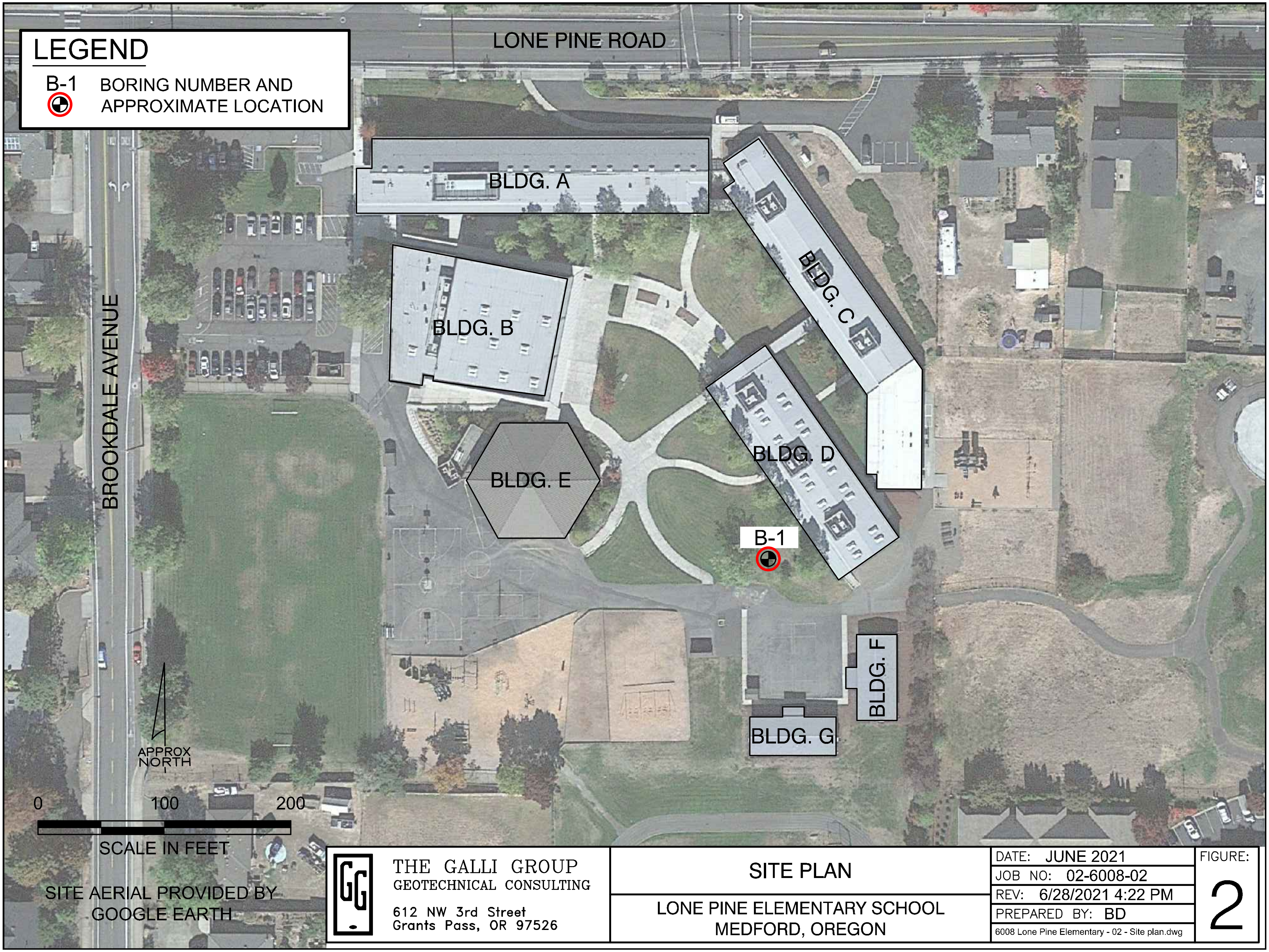
THE GALLI GROUP
GEOTECHNICAL CONSULTING
612 NW 3rd Street
Grants Pass, OR 97526

VICINITY MAP

LONE PINE ELEMENTARY SCHOOL
MEDFORD, OREGON

DATE:	JUNE 2021
JOB NO:	02-6008-02
REV:	6/28/2021 4:12 PM
PREPARED BY:	BD
6008 Lone Pine Elementary - 01 - Vicinity.dwg	

FIGURE:
1



THE GALLI GROUP
GEOTECHNICAL CONSULTING
612 NW 3rd Street
Grants Pass, OR 97526

SITE PLAN

LONE PINE ELEMENTARY SCHOOL
MEDFORD, OREGON

DATE: JUNE 2021
JOB NO: 02-6008-02
REV: 6/28/2021 4:22 PM
PREPARED BY: BD
6008 Lone Pine Elementary - 02 - Site plan.dwg

FIGURE: 2

APPENDIX A

BORING LOGS

THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

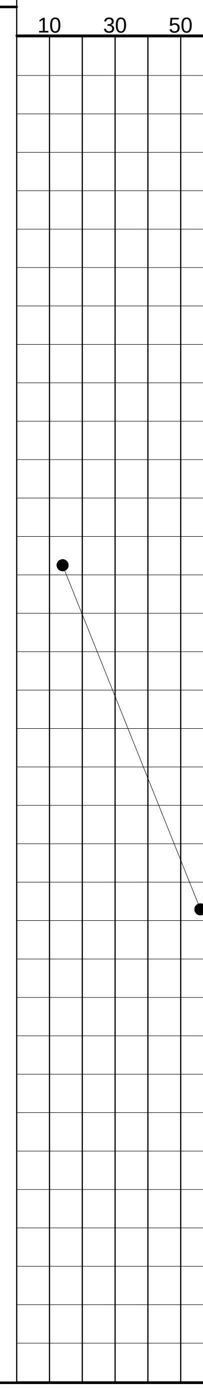
BORING LOG
B-1

Project: Lone Pine Elementary School
Client: Medford School District
Location: The lawn near the southwest corner of Building D (See Site Plan)
Driller: TGG (Ken, Nick)
Drill Rig: ATV Mounted Rig, 4" Diameter SSA
Depth To Water> Initial : None

Project No.: 02-6008-01
Date: 6/11/2021
Elevation:
Logged By: Dennis Duru
At Completion : None

USCS	Description	Depth	Sample No. and Type	NMC	Standard Penetration Test N
OL	Grass rootzone.	0–0.25			
GC	Medium dense, black, clayey Gravel; moist.	0.25–4.0	S-1	9%	14
GM	Medium dense to very dense, silty Gravel; moist.	4.0–5.1	S-2	11%	56
	Bottom of boring at 5.1 feet. No groundwater encountered.	5.1			

Standard Penetration Test CURVE



Legend of Samplers: Grab sample SPT sample Shelby tube sample

This information pertains only to this boring and should not be interpreted as being indicative of the site.

THE GALLI GROUP
Geotechnical Consulting

PRELIMINARY SEISMIC RETROFIT STUDY
OAK GROVE ELEMENTARY SCHOOL
MEDFORD, OREGON

For: Ron Havniear
Medford School District
815 South Oakdale Ave
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02-6017-01
June 30, 2021

Table of Contents

PRELIMINARY SEISMIC RETROFIT STUDY
OAK GROVE ELEMENTARY SCHOOL
MEDFORD, OREGON

1.0 INTRODUCTION

This report presents results of our geotechnical and geological evaluation of the Washington Elementary School for a potential Seismic Retrofit of portions of the school campus. The subject school is located at 2838 W. Main Street on the north side of West Main Street at its intersection with Oak Grove Road in Medford Oregon. Please see Figure 1, Vicinity Map, for a more precise location.

The purpose of our investigation and this report was to accomplish a limited site surface and subsurface evaluation (one boring) and conduct a planning level seismic risk assessment (office studies) in order to provide preliminary geotechnical and geologic information and evaluate the likelihood and consequences of geotechnical/geologic related seismic failures, including liquefaction and landslide potential during the design seismic event, for consideration of the potential seismic retrofit.

2.0 SITE AND PROJECT DESCRIPTION

The site is currently occupied by a functioning school, which consists of approximately 7 structures, connected via covered walkways or with direct connections. The structures are surrounded by play fields, access roads, parking lots, walkways and open space. The site is relatively flat to mildly sloping with undeveloped portions of the site consisting of well-maintained lawn and a few trees.

We understand the district is conducting a preliminary review to determine the level of seismic retrofit needed for the structures on this campus. Their review will largely be based on the evaluation of the potential geologic hazards (such as liquefaction) provided in this report, and an evaluation of the potential structural damage to these facilities associated with the design seismic event. The findings will also likely be used to determine if grant funding will be pursued to complete the seismic retrofit work.

3.0 FIELD EXPLORATION

On June 18, 2021, Associate Engineer, Dennis Duru, M.Sc., E.I.T, and our drilling crew, visited the site to accomplish the subsurface investigation. One (1) exploratory boring was drilled in the planter area near the southeast corner of the school buildings. A utility locate was completed prior to our investigation and our representative coordinated with school personnel to identify the field exploration location away from the marked and known utility locations. See Figure 2, Site Plan, for a layout of the site and the location of the boring. The boring was drilled with our ATV-mounted solid stem auger drill rig and penetrated to depth of 11.0 feet before encountering the hard, gravelly Clay (hardpan). Upon completion, the boring was backfilled with drill spoils.

Standard Penetration Testing (SPT) was accomplished during drilling, as part of the exploratory boring. This entails driving a 1½-inch diameter steel split spoon sampler by dropping a 140-pound weight for a 30-inch drop. The total number of blows it takes to drive the sampler the last 12 inches of an 18-inch drive is called the SPT N-value. These can be correlated with density and soil strength parameters from testing on thousands of other projects.

Our representative identified the final exploration location, logged subsurface soils and water conditions and obtained soil samples for transport to our laboratory. Visual classifications of the soils were made in the field and are presented in the Boring Log in Appendix A, at the end of this report.

4.0 LABORATORY TESTING

Moisture content tests were accomplished on soil samples obtained by Standard Penetration Testing. No other tests were accomplished.

5.0 SUBSURFACE CONDITIONS

5.1 SOIL

The subsurface consisted of medium stiff, brown, clayey Silt to approximately 4.0 feet. This was underlain by medium dense to dense, clayey, sandy Gravels, to a depth of 11.0 feet. This was then underlain by hard, brown, cemented gravelly Clay.

Please see more specific soils information in the Boring Logs in Appendix A. Please note that the soils are shown as distinct layers in the Boring Logs while in nature they may change more gradually. Soils conditions may also change somewhat between the locations investigated.

5.2 GROUNDWATER

Generally, the soils encountered were moist to saturated. Groundwater was encountered in the boring at 4.0 feet below ground surface.

6.0 GEOLOGIC OR SEISMIC INDUCED HAZARDS

Summary of Site Geology and Seismicity. The project area is located in the Medford East 30x60 minute USGS topographic quadrangle (see Vicinity Map, Figure 1). Mapped geologic units at the project area consist primarily of Alluvial Fan deposits and bedrock members of the Hornbrook Formation (Wiley et al, 2011). The Marine Sandstone, Siltstone and Conglomerate members are the mapped bedrock unit at the project (Wiley et al, 2011; OGDC-6, 2015). Weathered conglomerate Sandstone was encountered in the auger boring at this site at relatively shallow depths of 11.0 feet below ground surface.

The project site is in proximity to several zones of active seismicity. The region is affected by the Cascadia Subduction Zone (CSZ), an active subduction zone off the Oregon coast considered capable of Magnitude 8.5 or greater earthquakes. Average recurrence intervals for such great earthquakes, as determined by recent investigations, range between 300-600 years. The last "great" earthquake was interpreted to be approximately 300 years ago. The CSZ is the main seismic event for consideration for this seismic retrofit.

The project area has an additional tectonic source from earthquakes occurring along active Basin and Range faults as close as 50 kilometers to the southeast. This region has produced numerous earthquakes, including significant events near Klamath Falls and Warner Valley. Such events occur generally once every one to two decades.

Flood Hazard. The site is not within a 100-year floodplain of any river or streams according to the FEMA flood mapping.

Landslides/Slope Instability. The project site is located within a mapped Quaternary landside (Qls). This mapped feature is present on the state landslide database (Statewide Landslide Information Database for Oregon; SLIDO, 2017). Based upon the published mapping, general geomorphology, review of 2-foot contours generated from Lidar datasets (Dogami, 2021) and aerial photos (Google Earth, 2021), as well as subsurface data obtained in this investigation, the mapped landslide in the project area is interpreted to be an alluvial fan deposit of material originating upslope.

No recent movement or damage to structures has been associated with this feature in readily available published accounts or our general geotechnical and geologic knowledge of the area. It is therefore assumed this is an inactive “older” deposit. Therefore, in our professional opinion, based on the information from our limited exploration data, the risk of damage due to natural slope instability at this site is considered low. However, any proposed manmade cut or fill slopes should only be made following the recommendations from a detailed geotechnical investigation and report.

Liquefaction and Lateral Spread. The project is underlain by clayey silt, clay sandy gravels and gravelly Clay. Sandy soils with clay and gravel content in a medium dense to dense condition (similar to the ones observed during our limited exploration) have not been known to liquefy in a seismic event. Therefore, liquefaction and lateral spread is considered to be a low to very low potential hazard for this site. See more in Section 7.0 of this report.

Expansive Soils. The upper soils within the subsurface consist of clayey silt and clayey sandy Gravel. The clays will likely have moderate to high expansion potential based on our experience with soils with similar visual properties.

These expansive soils can have adverse impacts on foundations and all manner of concrete flatwork if the building design does not account for such soils. We recommend that each proposed building site (with clayey soils present) have the soil subgrade examined and tested (Expansion Index) prior to final design and construction. In that way, locations that have expansive soils will have the retrofit structure(s) and drainage designed accordingly.

Construction over expansive soils generally requires embedment of footings to 3 to 4 feet (final depth to be verified after EI testing) below the exterior grade and placement of floor slabs over at least 24 inches or more of compacted rock fill in order to mitigate expansion potential of the underlying soil subgrades. Maintaining the moisture content in the soil to keep it in a moist and fully swelled condition prior to being covered is also critical to proper performance of the structures.

Note: The geotechnical engineer must provide site specific laboratory testing and remedial design recommendations on projects that have potentially expansive, gravelly Clay, clayey Silt or Clay soils present.

Ground Rupture. No Quaternary faults were identified at the project site on review of USGS fault (US Quaternary Faults) maps, and from the one limited exploratory boring. Therefore, the risk of damage at the site due to ground rupture is considered very low.

Ground Shaking. Project structures including foundations and retaining walls must be designed for the potential for very strong ground shaking during the anticipated seismic event. The peak site modified horizontal acceleration (PGA_M), is 0.363g. This is based on the Site Class C designation, determined for the project from subsurface drilling and evaluation of SPT data. The PGA_M value can be used with an appropriate seismic coefficient in pseudo static analysis for design of the seismic upgrades.

Seismic Ground Amplification or Resonance. No unusually hazardous amplification or resonance effects on seismic waves have been associated with soil/bedrock subsurface conditions in the project area.

Tsunami and Seiche. The site is approximately 80 miles inland from the coast, and not subject to tsunami hazard. The site is not located adjacent to a large lake or body of water, and therefore, not subject to seiche hazard.

7.0 LIQUEFACTION EVALUATION

The liquefaction phenomenon occurs in saturated, loose (low density, uncompacted or poorly compacted), cohesionless soils. When loose, cohesionless soils are saturated, which is the case when the soil is below the water table, then water fills the soil pores. In response to compression when a load is applied to the soil, the water increases in pressure and attempts to migrate towards zones of low pressure. However, if the applied load is rapid and large enough, or if it is repeated many times (cyclic loading), like in an earthquake, such that there is not enough time for the water to dissipate before the next cycle of loading is applied, then the water pressure may build up to a degree where it becomes greater than the grain-to-grain contact stresses of the soil. The grain-to-grain contact stresses are the source of the shear strength that supports structure foundations and overburden soils in these soil structures. This buildup of excess pore water pressure results in total or partial loss of the soil strength, and the soil may be observed to flow like a liquid, hence "liquefaction". At this point, the soil will lose all its shear strength, be deformed, and will not be able to support structures.

The site is underlain medium stiff clayey Silt and medium dense to dense, clayey, sandy Gravel. Groundwater was observed in the boring at 4.0 feet deep. The conditions for liquefaction to occur were not observed at this site during our limited subsurface investigation. The medium dense to dense, clayey, sandy Gravel below the groundwater will not undergo further densification enough to cause liquefaction during a seismic event due to the high gravel and clay content, and also due to the dense condition. Therefore, the possibility of liquefaction that could adversely affect the site is very low.

Therefore, in our professional opinion, the site conditions found in the boring will not result in wide spread liquefaction that will have significant adverse impacts on the structures during a seismic event.

8.0 CONCLUSIONS

In our professional opinion, based on our field investigation, office review and previous work in the area, the soils conditions at the site are suitable for a conventional seismic retrofit. Crushed rock structural fill over the clayey, sandy Gravel will provide adequate support of new foundations, grade beams and/or buttresses. In our opinion, this school site is not subject to large scale liquefaction that will cause a significant adverse impact to the structures.

Additional borings around the structures on this site could possibly find zones of soils that may liquefy. However, these are likely to be moderate to small in size and should not adversely impact the structure.

If a full seismic retrofit geotechnical design report is needed, additional tasks to be accomplished would be as follows:

1. 2 or 3 additional borings.
2. Laboratory testing for expansive index, strength and settlement evaluation.
3. Evaluation of data for developing design parameters.

These could be used to provide a full scale Seismic Retrofit Design Report.

8.1 LIMITATIONS

The analyses, conclusions and recommendations contained in this report are based on site conditions as they existed at the time of the study, and assume soils, rock and groundwater conditions exposed and observed in the boring during our investigation are representative of soils and groundwater conditions throughout the site. If during construction, subsurface conditions or assumed design information is found to be different, we should be advised at once so that we can review this report and reconsider our recommendations in light of the changed conditions. If there is a significant lapse of time (5 years) between submission of this report and the start of work at the site, if the project is changed, or if conditions have changed due to acts of God or construction at or adjacent to the site, it is recommended that this report be reviewed in light of the changed conditions and/or time lapse.

This report was prepared for the use of the School District and their design team for evaluating the need for a full scale Seismic Retrofit evaluation and design report. It should be made available to contractors for information and factual data only. This report should not be used for contractual purposes as a warranty of site subsurface conditions. It should also not be used at other sites or for projects other than the one intended.

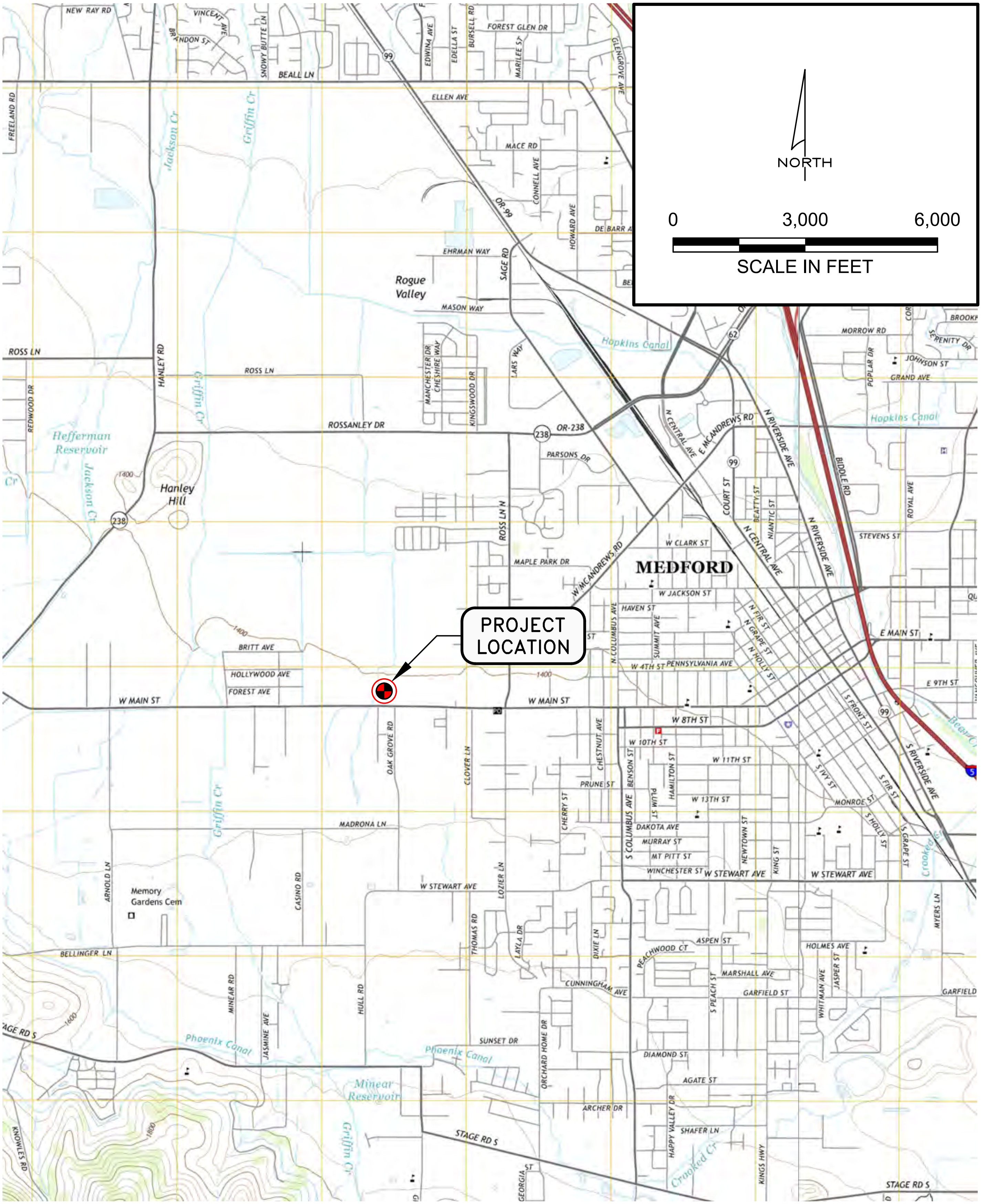
We have performed these services in accordance with generally accepted geotechnical engineering and professional geology practices in southern Oregon, at the time the study was accomplished. No other warranties, either expressed or implied, are provided.

THE GALLI GROUP
GEOTECHNICAL CONSULTING

Dennis Duru, M.S., E.I.T.
Staff Associate

Melvin Galli III, P.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER 84454 OREGON January 12, 2016 MELVIN JOHN GALLI III
EXPIRES: 06/30/21



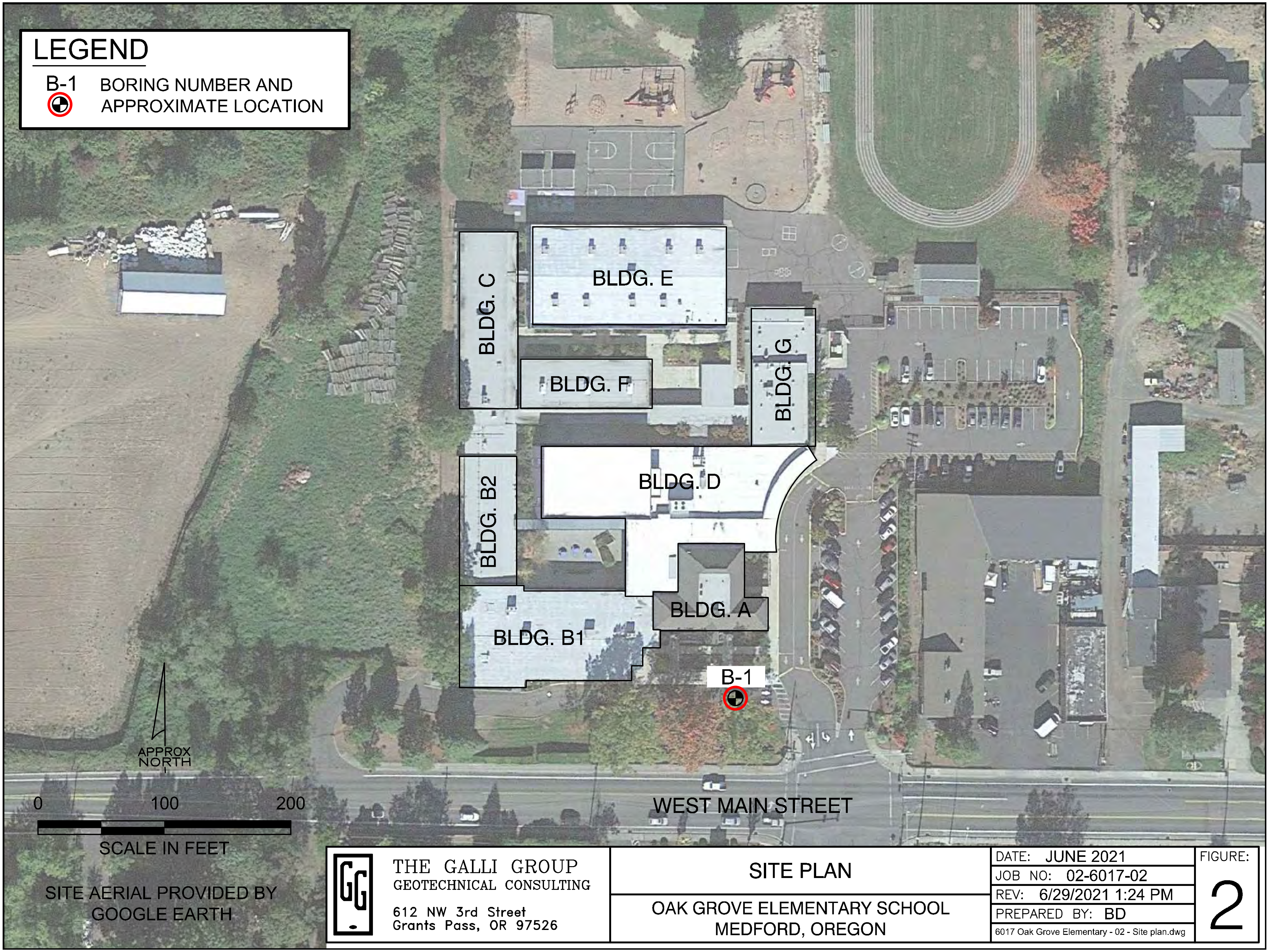
THE GALLI GROUP
GEOTECHNICAL CONSULTING
612 NW 3rd Street
Grants Pass, OR 97526

VICINITY MAP

OAK GROVE ELEMENTARY SCHOOL
MEDFORD, OREGON

DATE:	JUNE 2021
JOB NO:	02-6017-02
REV:	6/29/2021 1:15 PM
PREPARED BY:	BD
6017 Oak Grove Elementary - 01 - Vicinity.dwg	

FIGURE: 1



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612 NW 3rd Street
Grants Pass, OR 97526

SITE PLAN

OAK GROVE ELEMENTARY SCHOOL
MEDFORD, OREGON

DATE:	JUNE 2021
JOB NO:	02-6017-02
REV:	6/29/2021 1:24 PM
PREPARED BY:	BD
6017 Oak Grove Elementary - 02 - Site plan.dwg	

FIGURE: 2

APPENDIX A

BORING LOGS

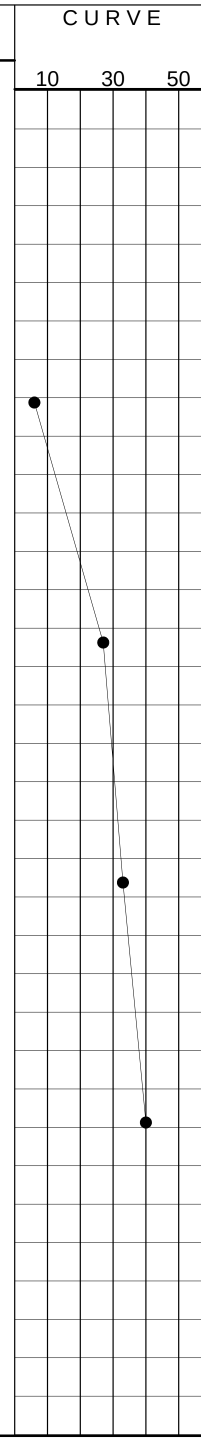
THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

BORING LOG
B-1

Project: Oak Grove Elementary School | Project No.: 02-6017-02
Client: Medford School District | Date: 6/18/2021
Location: Planter area south of Building A (see Site Plan) | Elevation:
Driller: TGG (Ken, Nick) | Logged By: Dennis Duru
Drill Rig: ATV Mounted SSA, 4" Diameter
Depth To Water> Initial: 4.0 | At Completion: 4.0

USCS	Description	Depth
	Grass rootzone.	0.25
CH-MH	Medium stiff, brown, clayey Silt; some gravel moist.	5.0
GC/SC	Medium dense to dense, clayey, sandy Gravel; saturated.	11.0
CL	Hard, brown, cemented, gravelly Clay; damp.	11.5
	Bottom of boring at 11.5 Free groundwater at 4.0	

Sample No. and Type	NMC	Standard Penetration Test N
S-1	9%	6
S-2	17%	27
S-3	17%	33
S-4	17% 13%	40



Legend of Samplers: Grab sample | SPT sample | Shelby tube sample

This information pertains only to this boring and should not be interpreted as being indicative of the site.

THE GALLI GROUP
Geotechnical Consulting

PRELIMINARY SEISMIC RETROFIT STUDY
WASHINGTON ELEMENTARY SCHOOL
MEDFORD, OREGON

For: Ron Havniear
Medford School District
815 South Oakdale Ave
Medford, OR 97501

c/o Stephen L. Chase, Project Manager
ZCS Engineering & Architecture
127 NW D Street
Grants Pass, Oregon 97526

By: **THE GALLI GROUP**
612 NW Third Street
Grants Pass, OR 97526
(541) 955-1611

02-6009-01
June 30, 2021

Table of Contents

PRELIMINARY SEISMIC RETROFIT STUDY
WASHINGTON ELEMENTARY SCHOOL
MEDFORD, OREGON

1.0 INTRODUCTION

This report presents results of our geotechnical and geological evaluation of the Washington Elementary School for a potential seismic retrofit of the school. The subject school is located at 610 S. Peach Street, on the northwest corner of the intersection of South Peach Street and Dakota Avenue in Medford Oregon. Please see Figure 1, Vicinity Map, for a more precise location.

The purpose of our investigation and this report was to accomplish a limited site surface and subsurface evaluation (one boring) and conduct a planning level seismic risk assessment (office studies) in order to provide preliminary geotechnical and geologic information and evaluate the likelihood and consequences of geotechnical/geologic related seismic failures, including liquefaction and landslide potential during the design seismic event, for consideration of the potential seismic retrofit.

2.0 SITE AND PROJECT DESCRIPTION

The site is currently occupied by a functioning elementary school, which consists of 3 to 4 structures directly connected to create a large building complex. The structures are surrounded by play fields, access roads, parking, walkways and open space. The site is relatively flat with undeveloped portions of the site consisting of landscaping, well-maintained lawn and a few trees.

We understand the district is conducting a preliminary review to determine the level of seismic retrofit needed for the structures on this campus. Their review will largely be based on the evaluation of the potential geologic hazards (such as liquefaction) provided in this report, and an evaluation of the potential structural damage to these facilities associated with the design seismic event. The findings will also likely be used to determine if grant funding will be pursued to complete the seismic retrofit work.

3.0 FIELD EXPLORATION

On June 11, 2021, Associate Engineer, Dennis Duru, M.Sc., E.I.T and our drilling crew, visited the site to accomplish the subsurface investigation. One (1) exploratory boring was drilled on the planter area located on the north site of the schools' main building (Building A). A utility locate was completed prior to our investigation and our representative coordinated with school personnel to identify the field exploration location away from the marked and known utility locations. See Figure 2, Site Plan, for a layout of the site and the location of the boring. The boring was drilled with our ATV-mounted, solid-stem auger drill rig and penetrated to a depth of 8.0 feet before encountering the weathered sandstone bedrock. Upon completion, the boring was backfilled with drill spoils.

Standard Penetration Testing (SPT) was accomplished in each boring. This entails driving a 1½-inch diameter steel split spoon sampler by dropping a 140-pound weight for a 30-inch drop. The total number of blows it takes to drive the sampler the last 12 inches of an 18-inch drive is called the SPT N-value. These can be correlated with density and soil strength parameters from testing on thousands of other projects.

Our representative identified the final exploration location, logged subsurface soils and water conditions and obtained soil samples for transport to our laboratory. Visual classifications of the soils were made in the field and are presented in the Boring Logs in Appendix A, at the end of this report.

4.0 LABORATORY TESTING

Moisture content tests were accomplished on soil samples obtained by Standard Penetration Testing. No other tests were accomplished.

5.0 SUBSURFACE CONDITIONS

5.1 SOIL

The subsurface consisted of stiff, brown Clay with some gravel to a depth of 4.0 feet. This was then underlain by dense to very dense, brown, sandy Gravel (top of weathered Sandstone bedrock).

Please see more specific soils information in the Boring Logs in Appendix A. Please note that the soils are shown as distinct layers in the Boring Logs while in nature they may change more gradually. Soils conditions may also change somewhat between the locations investigated.

5.2 GROUNDWATER

Generally, the soils encountered were moist. No groundwater was encountered in the one boring. Review of nearby well log information shows that the static groundwater level is approximately at 14 feet below the ground surface.

6.0 GEOLOGIC OR SEISMIC INDUCED HAZARDS

Summary of Site Geology and Seismicity. The project area is located in the Medford East 30x60 minute USGS topographic quadrangle (see Vicinity Map, Figure 1). Mapped geologic units at the project area consist primarily of Alluvial Fan deposits and bedrock members of the Hornbrook Formation (Wiley et al, 2011). The Marine Sandstone, Siltstone and Conglomerate members are the mapped bedrock unit at the project (Wiley et al, 2011; OGDC-6, 2015). Weathered conglomerate Sandstone was encountered in the auger boring at this site at relatively shallow depths of 4.0 feet below ground surface.

The project site is in proximity to several zones of active seismicity. The region is affected by the Cascadia Subduction Zone (CSZ), an active subduction zone off the Oregon coast considered capable of Magnitude 8.5 or greater earthquakes. Average recurrence intervals for such great earthquakes, as determined by recent investigations, range between 300-600 years. The last "great" earthquake was interpreted to be approximately 300 years ago. The CSZ is the main seismic event for consideration for this seismic retrofit.

The project area has an additional tectonic source from earthquakes occurring along active Basin and Range faults as close as 50 kilometers to the southeast. This region has produced numerous earthquakes, including significant events near Klamath Falls and Warner Valley. Such events occur generally once every one to two decades.

Flood Hazard. The site is not near streams or rivers. Therefore, it is not within a 100-year floodplain.

Landslides/Slope Instability. There are no slopes close to the site. Therefore, possibility of slope failure, rock fall or slide run out damage at the site is very low.

Liquefaction and Lateral Spread. The project is underlain by Clay with Gravel over dense to very dense sandy Gravel in the shallow subsurface. Sandy gravels with densities similar to those logged during our limited exploration have not been known to liquefy in a seismic event. In addition, ground water was not encountered during our limited exploration. <u>Therefore, liquefaction and lateral spread is considered to be a low to very low potential hazard for this site.</u> See more in Section 7.0 of this report.

Expansive Soils. The upper soils within the subsurface consist of Clay with some gravel. The upper portion of the subsurface is mapped as the soil unit 34B (Coleman Loam) according to the NRCS-Web soil survey. This soil consists of Loam (CL), Clay Loam (CL), Clay (CH), gravelly Clay (GC), with liquid limits between 30 and 60. Given the liquid limit range, and based on our experience with soils with similar properties, the upper soils would likely have moderate expansion potential.

These expansive soils can have adverse impacts on foundations and all manner of concrete flatwork if the building design does not account for such soils. We recommend that each proposed building site (with clayey soils present) have the soil subgrade examined and tested (Expansion Index) prior to final design and construction. In that way, locations that have expansive soils will have the retrofit structure(s) and drainage designed accordingly.

Construction over expansive soils generally requires embedment of footings to 3 to 4 feet (final depth to be verified after EI testing) below the exterior grade and placement of floor slabs over at least 24 inches or more of compacted rock fill in order to mitigate expansion potential of the underlying soil subgrades. Maintaining the moisture content in the soil to keep it in a moist and fully swelled condition prior to being covered is also critical to proper performance of the structures.

Note: The geotechnical engineer must provide site specific laboratory testing and remedial design recommendations on projects that have potentially expansive, gravelly Clay, clayey Silt or Clay soils present.

Ground Rupture. No Quaternary faults were identified at the project site. Therefore, the risk of damage at the site due to ground rupture is considered very low.

Ground Shaking. Project structures including foundations and retaining walls must be designed for the potential for very strong ground shaking during the anticipated seismic event. The peak site modified horizontal acceleration (PGA_M), is 0.358g. This is based on the Site Class C designation, determined for the project from subsurface drilling and evaluation of SPT data. The PGA_M value can be used with an appropriate seismic coefficient in pseudo static analysis for design of the seismic upgrades.

Seismic Ground Amplification or Resonance. No unusually hazardous amplification or resonance effects on seismic waves have been associated with soil/bedrock subsurface conditions in the project area.

Tsunami and Seiche. The site is approximately 80 miles inland from the coast, and not subject to tsunami hazard. The site is not located adjacent to a large lake or body of water, and therefore, not subject to seiche hazard.

7.0 LIQUEFACTION EVALUATION

The liquefaction phenomenon occurs in saturated, loose (low density, uncompacted or poorly compacted), cohesionless soils. When loose, cohesionless soils are saturated, which is the case when the soil is below the water table, then water fills the soil pores. In response to compression when a load is applied to the soil, the water increases in pressure and attempts to migrate towards zones of low pressure. However, if the applied load is rapid and large enough, or if it is repeated many times (cyclic loading) like in an earthquake, such that there is not enough time for the water to dissipate before the next cycle of loading is applied, then the water pressure may build up to a degree where they become greater than the grain-to-grain contact stresses of the soil. The grain-to-grain contact stresses are the source of the shear strength that support structure foundations and overburden soils in these soil structures. This buildup of excess pore water pressure results in total or partial loss of the soil strength, and the soil may be observed to flow like a liquid, hence "liquefaction". At this point, the soil will lose all its shear strength, be deformed, and will not be able to support structures.

The site is underlain by dense to very dense sandy Gravel. No groundwater was observed in the boring to the depth drilled. The conditions for liquefaction to occur were not observed at this site during our limited subsurface investigation. Such soils will not undergo further densification enough to cause liquefaction during a seismic event. Also, water was not observed in our boring and could be below 15 feet deep. Therefore, the possibility of liquefaction that could adversely affect the site is very low.

Therefore, in our professional opinion, the site conditions found in the boring will not result in wide spread liquefaction that will have significant adverse impacts on the structures during a seismic event.

8.0 CONCLUSIONS

In our professional opinion, based on our field investigation, office review and previous work in the area, the soils conditions at the site are suitable for a conventional seismic retrofit. Crushed rock structural fill over the sandy Gravel will provide adequate support of new foundations, grade beams and/or buttresses. In our opinion, this school site is not subject to large scale liquefaction that will cause a significant adverse impact to the school structure.

Additional borings around the structures on this site could possibly find zones of soils that may liquefy. However, these are likely to be moderate to small in size and should not adversely impact the structure.

If a full seismic retrofit geotechnical design report is needed, additional tasks to be accomplished would be as followed:

1. 2 or 3 additional borings.
2. Laboratory testing for determining expansion index and strength and settlement characteristics of the site soils.
3. Evaluation of data for developing design parameters.

These could be used to provide a full scale Seismic Retrofit Design Report.

8.1 LIMITATIONS

The analyses, conclusions and recommendations contained in this report are based on site conditions as they existed at the time of the study, and assume soils, rock and groundwater conditions exposed and observed in the boring during our investigation are representative of soils and groundwater conditions throughout the site. If during construction, subsurface conditions or assumed design information is found to be different, we should be advised at once so that we can review this report and reconsider our recommendations in light of the changed conditions. If there is a significant lapse of time (5 years) between submission of this report and the start of work at the site, if the project is changed, or if conditions have changed due to acts of God or construction at or adjacent to the site, it is recommended that this report be reviewed in light of the changed conditions and/or time lapse.

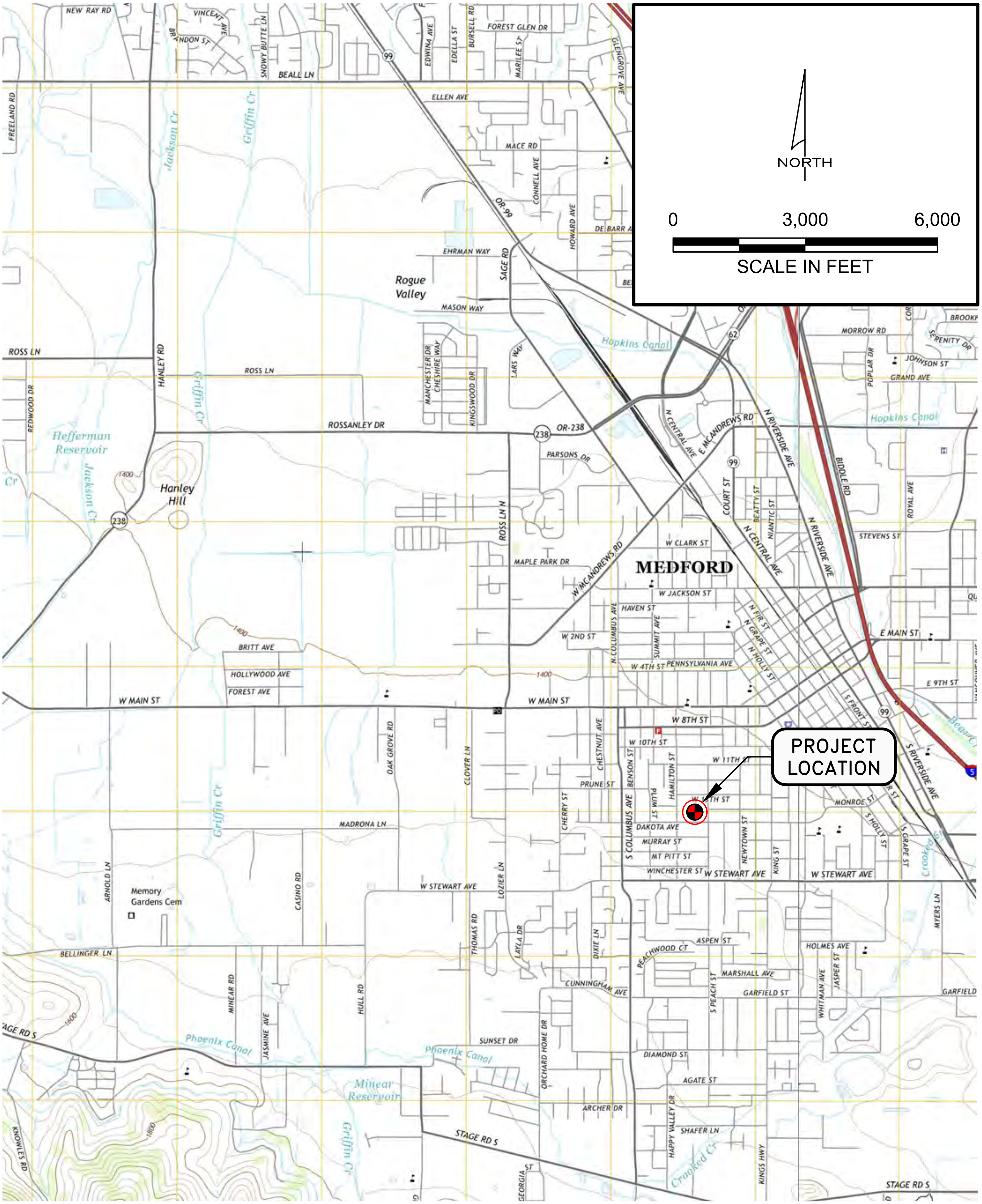
This report was prepared for the use of the School District and their design team for evaluating the need for a full scale Seismic Retrofit evaluation and report. It should be made available to contractors for information and factual data only. This report should not be used for contractual purposes as a warranty of site subsurface conditions. It should also not be used at other sites or for projects other than the one intended.

We have performed these services in accordance with generally accepted geotechnical engineering and professional geology practices in southern Oregon, at the time the study was accomplished. No other warranties, either expressed or implied, are provided.

THE GALLI GROUP
GEOTECHNICAL CONSULTING

Dennis Duru, M.S., E.I.T.
Staff Associate

Melvin Galli III, P.E.
Principal Engineer



THE GALLI GROUP
GEOTECHNICAL CONSULTING

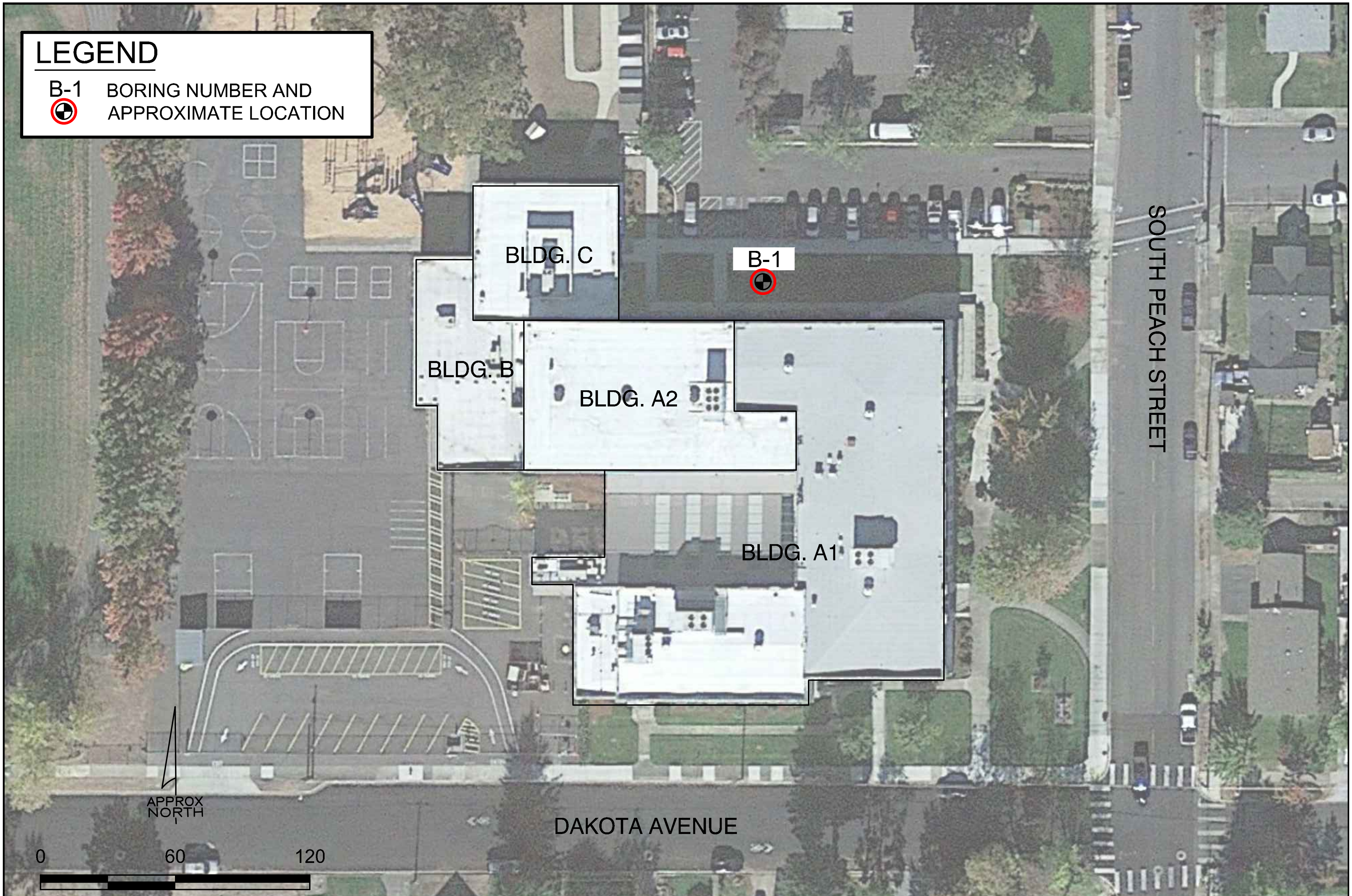
612 NW 3rd Street
Grants Pass, OR 97526

VICINITY MAP

WASHINGTON ELEMENTARY SCHOOL
MEDFORD, OREGON

DATE:	JUNE 2021
JOB NO:	02-6009-02
REV:	6/28/2021 3:22 PM
PREPARED BY:	BD
6009 Washington Elementary - 01 - Vicinity.dwg	

FIGURE:
1



THE GALLI GROUP
GEOTECHNICAL CONSULTING
612 NW 3rd Street
Grants Pass, OR 97526

SITE PLAN

WASHINGTON ELEMENTARY SCHOOL
MEDFORD, OREGON

DATE: JUNE 2021
JOB NO: 02-6009-02
REV: 6/28/2021 3:47 PM
PREPARED BY: BD
6009 Washington Elementary - 02 - Site plan.dwg

FIGURE:
2

APPENDIX A

BORING LOGS

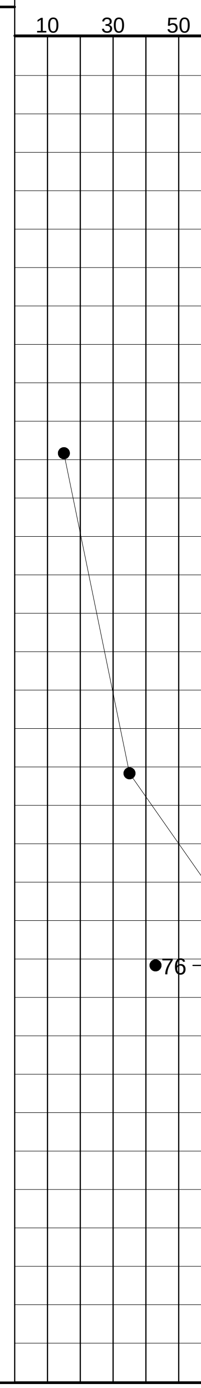
THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

BORING LOG
B-1

Project: Washington Elementary School
Client: Medford School District
Location: Planter area north of main Building A1 & A2 (see Site Plan)
Driller: TGG (Ken, Nick)
Drill Rig: ATV Mounted RIG, 4" diameter SSA
Depth To Water> Initial : NONE

Project No.: 02-6009-01
Date: 6/11/2021
Elevation:
Logged By: Dennis Duru
At Completion : NONE

Graphic Log	USCS	Description	Depth	Sample No. and Type	NMC	Standard Penetration Test N
	OL	Grass rootzone. (0.25)	0			
	CH	Stiff, brown, Clay; some gravel, moist. (4.0)	1.5, 3	S-1	21%	15
	GW-GM	Dense to very dense, brown, sandy Gravel; moist. (8.0)	4.5, 6	S-2	9%	35
			7.5	S-3	15%	76
		Bottom of boring at 8.0 feet. No groundwater encountered.	9, 10.5			



Legend of Samplers: Grab sample SPT sample Shelby tube sample

This information pertains only to this boring and should not be interpreted as being indicative of the site.

THE GALLI GROUP
Geotechnical Consulting

PRELIMINARY SEISMIC RETROFIT STUDY
WILSON ELEMENTARY SCHOOL
MEDFORD, OREGON

For: Medford School District
c/o Stephen L. Chase, Project Manager
ZCS Engineering & Architecture
127 NW D Street
Grants Pass, Oregon 97526

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02-6010-01
June 30, 2021

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PRELIMINARY SEISMIC RETROFIT STUDY
WILSON ELEMENTARY SCHOOL
MEDFORD, OREGON

1.0 INTRODUCTION

This report presents the results of our geotechnical and geological evaluation of the Wilson Elementary School site for a potential seismic retrofit of the school complex. The subject school is located at 1400 Johnson Street on the southwest corner of the intersection of Johnson Street and Corona Avenue in Medford, Oregon. Please see Figure 1, Vicinity Map, for a more precise location.

The purpose of our investigation and this report was to accomplish a limited site surface and subsurface evaluation (one boring) and conduct a planning level seismic risk assessment (office studies) in order to provide preliminary geotechnical and geologic information and evaluate the likelihood and consequences of geotechnical/geologic related seismic failures, including liquefaction and landslide potential during the design seismic event, for consideration of the potential seismic retrofit.

2.0 SITE AND PROJECT DESCRIPTION

The site is currently occupied by a functioning school, which consists of approximately 9 structures, connected via covered walkways or with direct connections. The structures are surrounded by play fields, access roads, parking lots, walkways and open space. The site is relatively flat to mildly sloping with undeveloped portions of the site consisting of well-maintained lawn and a few trees.

We understand the district is conducting a preliminary review to determine the level of seismic retrofit needed for the structures on this campus. Their review will largely be based on the evaluation of the potential geologic hazards (such as liquefaction) provided in this report, and an evaluation of the potential structural damage to these facilities associated with the design seismic event. The findings will also likely be used to determine if grant funding will be pursued to complete the seismic retrofit work.

3.0 FIELD EXPLORATION

On June 11, 2021, Associate Engineer, Dennis Duru, M.Sc., E.I.T and our drilling crew, visited the site to accomplish the subsurface investigation. One (1) exploratory boring was drilled in the grass field on the south side of the southwest building (Building I). A

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utility locate was completed prior to our investigation and our representative coordinated with school personnel to identify the field exploration location away from the marked and known utility locations. See Figure 2, Site Plan, for a layout of the site and the location of the boring. The boring was drilled with our ATV-mounted solid stem auger drill rig and penetrated to depth of 12.75 feet before encountering the very dense, weathered Sandstone bedrock. Upon completion, the boring was backfilled with drill spoils.

Standard Penetration Testing (SPT) was accomplished during drilling, as part of the exploratory boring. This entails driving a 1½-inch diameter steel split spoon sampler by dropping a 140-pound weight for a 30-inch drop. The total number of blows it takes to drive the sampler the last 12 inches of an 18-inch drive is called the SPT N-value. These can be correlated with density and soil strength parameters from testing on thousands of other projects.

Our representative identified the final exploration location, logged subsurface soils and water conditions and obtained soil samples for transport to our laboratory. Visual classifications of the soils were made in the field and are presented in the Boring Log in Appendix A, at the end of this report.

4.0 LABORATORY TESTING

Moisture content tests were accomplished on soil samples obtained by Standard Penetration Testing. No other tests were accomplished.

5.0 SUBSURFACE CONDITIONS

5.1 SOIL

The boring encountered a surficial layer of medium stiff, dark brown to gray Clay to the depth of 4.0 feet. This was underlain by loose, brown Sand to the depth of 12.5 feet. This loose sand layer was then underlain by very dense, sandy Gravels.

Please see more specific soils information in the Boring Log in Appendix A. Please note that the soils are shown as distinct layers in the Boring Logs while in nature they may change more gradually. Soils conditions may also change somewhat between the locations investigated.

5.2 GROUNDWATER

Generally, the soils encountered were moist to saturated. Groundwater was encountered in the boring at 3.7 feet below ground surface.

6.0 GEOLOGIC OR SEISMIC INDUCED HAZARDS

Site Geology and Seismicity. The project area is located in the Medford East 30x60 minute USGS topographic quadrangle (see Vicinity Map, Figure 1). Mapped geologic units at the project area consist primarily of Alluvial Fan deposits and bedrock members of the Hornbrook Formation (Wiley et al, 2011). The Marine Sandstone, Siltstone and Conglomerate members are the mapped bedrock unit at the project (Wiley et al, 2011; OGDC-6, 2015). Weathered conglomerate Sandstone was encountered in the auger boring at this site at relatively shallow depths of 12.5 feet below ground surface.

The project site is in proximity to several zones of active seismicity. The region is affected by the Cascadia Subduction Zone (CSZ), an active subduction zone off the Oregon coast considered capable of Magnitude 8.5 or greater earthquakes. Average recurrence intervals for such great earthquakes, as determined by recent investigations, range between 300-600 years. The last "great" earthquake was interpreted to be approximately 300 years ago. The CSZ is the main seismic event for consideration for this seismic retrofit.

The project area has an additional tectonic source from earthquakes occurring along active Basin and Range faults as close as 50 kilometers to the southeast. This region has produced numerous earthquakes, including significant events near Klamath Falls and Warner Valley. Such events occur generally once every one to two decades.

Flood Hazard. The site is not within a 100-year floodplain of any river or streams according to the FEMA flood mapping.

Landslides/Slope Instability. There are no slopes close to the site. Therefore, possibility of slope failure, rock fall or slide run out damage at the site is low.

Liquefaction and Lateral Spread. The project is underlain by relatively thin layer (approximately 4 feet) of Clay over loose Sand to the depth of 12.5 feet. In addition, static groundwater was at 3.7 feet below ground surface. This means that all the loose sand within the subsurface are below the groundwater level. Loose Sand below the water table is known to liquefy in a seismic event. Therefore, the potential for liquefaction and lateral spread hazard for this site is considered very high. Lateral spread has been recorded within very mild slopes. This site has mild slopes and would likely experience lateral spreading during a seismic event. See more in Section 7.0 of this report.

Expansive Soils. The top 4 feet of soils within the subsurface consist of medium stiff, dark brown, Clay. The upper portion of the subsurface is mapped as the soil unit 127A (Medford silty clay loam) according to the NRCS-Web soil survey. This soil consists of silty clay loam, with liquid limits between 30 and 50. Given the liquid limit range and based on our experience with soils with similar visual properties, the upper soils would likely have moderate expansion potential.

These expansive soils can have adverse impacts on foundations and all manner of concrete flatwork if the building design does not account for such soils. We recommend that each proposed building site (with clayey soils present) have the soil subgrade examined and tested (Expansion Index) prior to final design and construction. In that way, locations that have expansive soils will have the retrofit structure(s) and drainage designed accordingly.

Construction over expansive soils generally requires embedment of footings to 3 to 4 feet (final depth to be verified after EI testing) below the exterior grade and placement of floor slabs over at least 24 inches or more of compacted rock fill in order to mitigate expansion potential of the underlying soil subgrades. Maintaining the moisture content in the soil to keep it in a moist and fully swelled condition prior to being covered is also critical to proper performance of the structures.

Note: The geotechnical engineer must provide site specific laboratory testing and remedial design recommendations on projects that have potentially expansive, gravelly Clay, clayey Silt or Clay soils present.

Ground Rupture. No Quaternary faults were identified at the project site. Therefore, the risk of damage at the site due to ground rupture is considered very low.

Ground Shaking. Project structures including foundations and retaining walls must be designed for the potential for very strong ground shaking during the anticipated seismic event**.** The peak site modified horizontal acceleration (PGA_M), is 0.385g. The PGA_M value can be used with an appropriate seismic coefficient in pseudo static analysis for design of the seismic upgrades.

Seismic Ground Amplification or Resonance. No unusually hazardous amplification or resonance effects on seismic waves have been associated with soil/bedrock subsurface conditions in the project area. Due to the presence of saturated loose Sand in the upper subsurface, the site class for the project is F, and a site-specific ground motion hazard analysis is required to determine the spectral acceleration parameter, which will capture any adverse seismic ground motion effect on the structures.

Tsunami and Seiche. The site is approximately 80 miles inland from the coast, and not subject to tsunami hazard. The site is not located adjacent to a large lake or body of water, and therefore, not subject to seiche hazard.

7.0 LIQUEFACTION EVALUATION

The liquefaction phenomenon occurs in saturated, loose (low density, uncompacted or poorly compacted), cohesionless soils. When loose, cohesionless soils are saturated, which is the case when the soil is below the water table, then water fills the soil pores. In response to compression when a load is applied to the soil, the water increases in pressure and attempts to migrate towards zones of low pressure. However, if the applied load is rapid and large enough, or if it is repeated many times (cyclic loading), like in an earthquake, such that there is not enough time for the water to dissipate before the next cycle of loading is applied, then the water pressure may build up to a degree where it becomes greater than the grain-to-grain contact stresses of the soil. The grain-to-grain contact stresses are the source of the shear strength that supports structure foundations and overburden soils in these soil structures. This buildup of excess pore water pressure results in total or partial loss of the soil strength, and the soil may be observed to flow like a liquid, hence "liquefaction". At this point, the soil will lose all its shear strength, be deformed, and will not be able to support structures.

This site is underlain by loose Sand. Groundwater was observed in the boring at the depth of 3.7 feet. The conditions for liquefaction to occur were present at this site during our limited subsurface investigation. The loose sand will be caused to further densify in a seismic event which will cause liquefaction.

Therefore, in our professional opinion, the site conditions found in the boring will result in wide spread liquefaction and lateral spreading that could have significant adverse impacts on the structures during a seismic event.

Seismic Settlement. During the design seismic event, the upper loose and saturated soils will undergo liquefaction which will induce settlement as well as loss of bearing capacity. Our analyses indicate that settlement between 6.0 and 10.0 inches is anticipated for structures constructed on this material. This analysis considered only the data from our single boring.

Lateral Spread. The site ground is mildly sloping at approximately 2% to the west. During the design seismic event, the upper loose and saturated soils will undergo liquefaction which will lead to lateral spreading of the liquefied soils. Widespread lateral spreading has been observed on sites that are only slightly sloping. Loss of shear strength in even a slightly sloping ground will cause the soil to flow. Our computation shows that lateral movement of the ground of up to 2.5 feet is possible for this site condition. Our computation utilized the data from our single boring and the surface ground slopes intersecting the boring location.

8.0 CONCLUSIONS

In our professional opinion, based on our field investigation, office review and seismic computations, the site will undergo substantial liquefaction which would lead to liquefaction induced settlement and lateral spread during the design seismic event. Detailed investigation would be necessary to evaluate and delineate the effect/extent of such widespread liquefaction on the structure(s) of interest. Retrofit design and construction must incorporate a foundation upgrade that would mitigate the impact of liquefaction at the project site. This will likely require some form of deep foundation support, such as reinforced concrete drilled piers or driven piles.

Additional exploration around the structure(s) on this site will be important to create a more detailed geologic model of the upper subsurface of interest, which will help in delineating the magnitude and extent of liquefaction that could impact the project. In addition, a detailed geologic model of the areas of interest, with the properties of the liquefiable soil fully defined would be very helpful for a Site Response Analysis for the project and in providing effective mitigation recommendations for the structures to reduce the impact of liquefaction.

In a full seismic retrofit geotechnical design report, additional tasks to be accomplished would be as follows:

1. Additional borings to define extent of liquefiable soils and to create a detailed geologic model of the subsurface.
2. Laboratory testing for soil particle analysis, strength and settlement evaluation.
3. Site Response Analysis.
4. Evaluation of data for developing design parameters.

These could be used to provide a full scale Seismic Retrofit Design Report with liquefaction mitigation recommendations.

8.1 LIMITATIONS

The analyses, conclusions and recommendations contained in this report are based on site conditions as they existed at the time of the study, and assume soils, rock and groundwater conditions exposed and observed in the boring during our investigation are representative of soils and groundwater conditions throughout the site. If during construction, subsurface conditions or assumed design information is found to be different, we should be advised at once so that we can review this report and reconsider our recommendations in light of the changed conditions. If there is a significant lapse of time (5 years) between submission of this report and the start of work at the site, if the project is changed, or if conditions have changed due to acts of God or construction at or adjacent to the site, it is recommended that this report be reviewed in light of the changed conditions and/or time lapse.

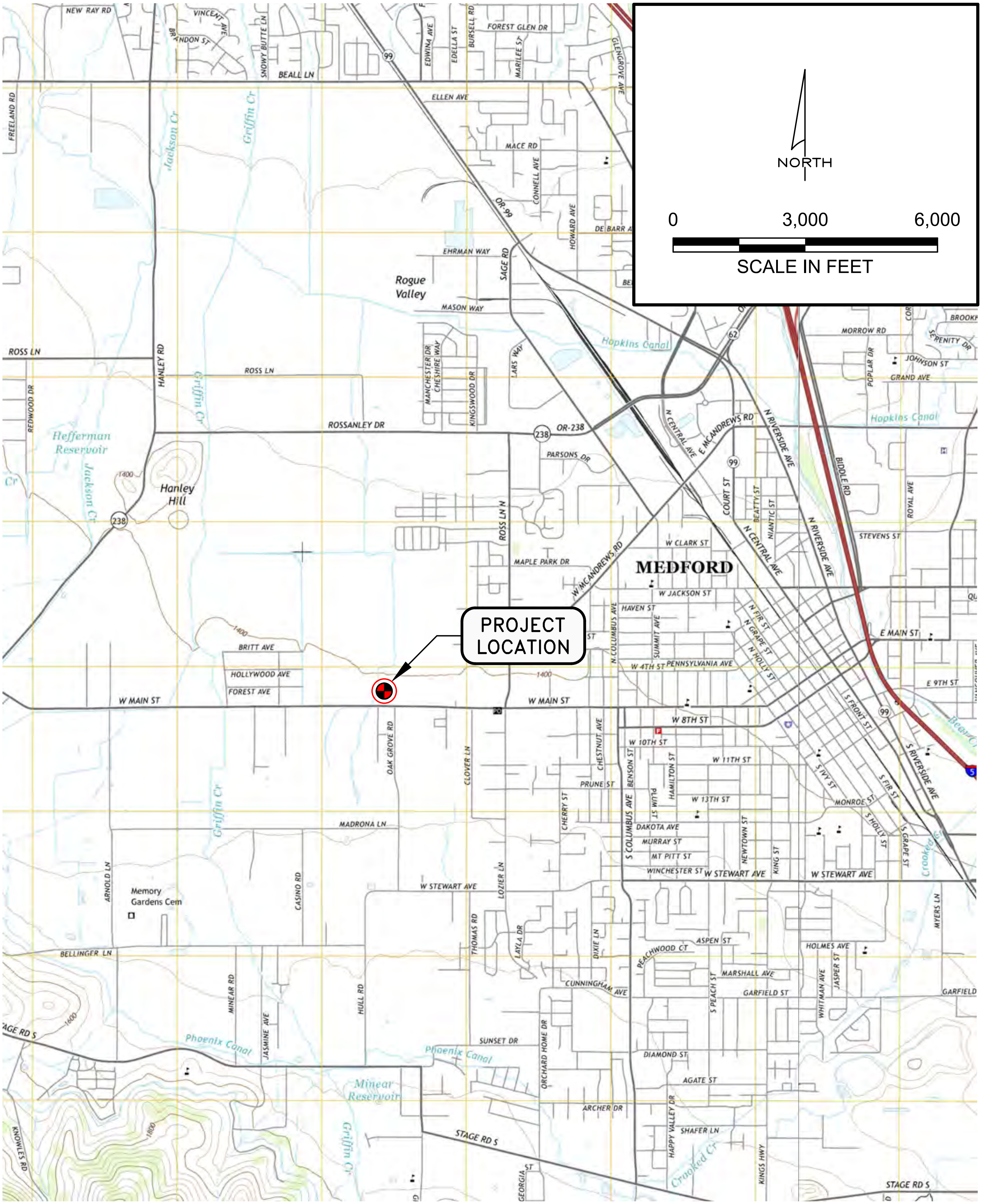
This report was prepared for the use of the School District and their design team for the preliminary assessment of the project and evaluating the need for a full scale Seismic Retrofit evaluation and geotechnical design report. It should be made available to contractors for information and factual data only. This report should not be used for contractual purposes as a warranty of site subsurface conditions. It should also not be used at other sites or for projects other than the one intended.

We have performed these services in accordance with generally accepted geotechnical engineering and professional geology practices in southern Oregon, at the time the study was accomplished. No other warranties, either expressed or implied, are provided.

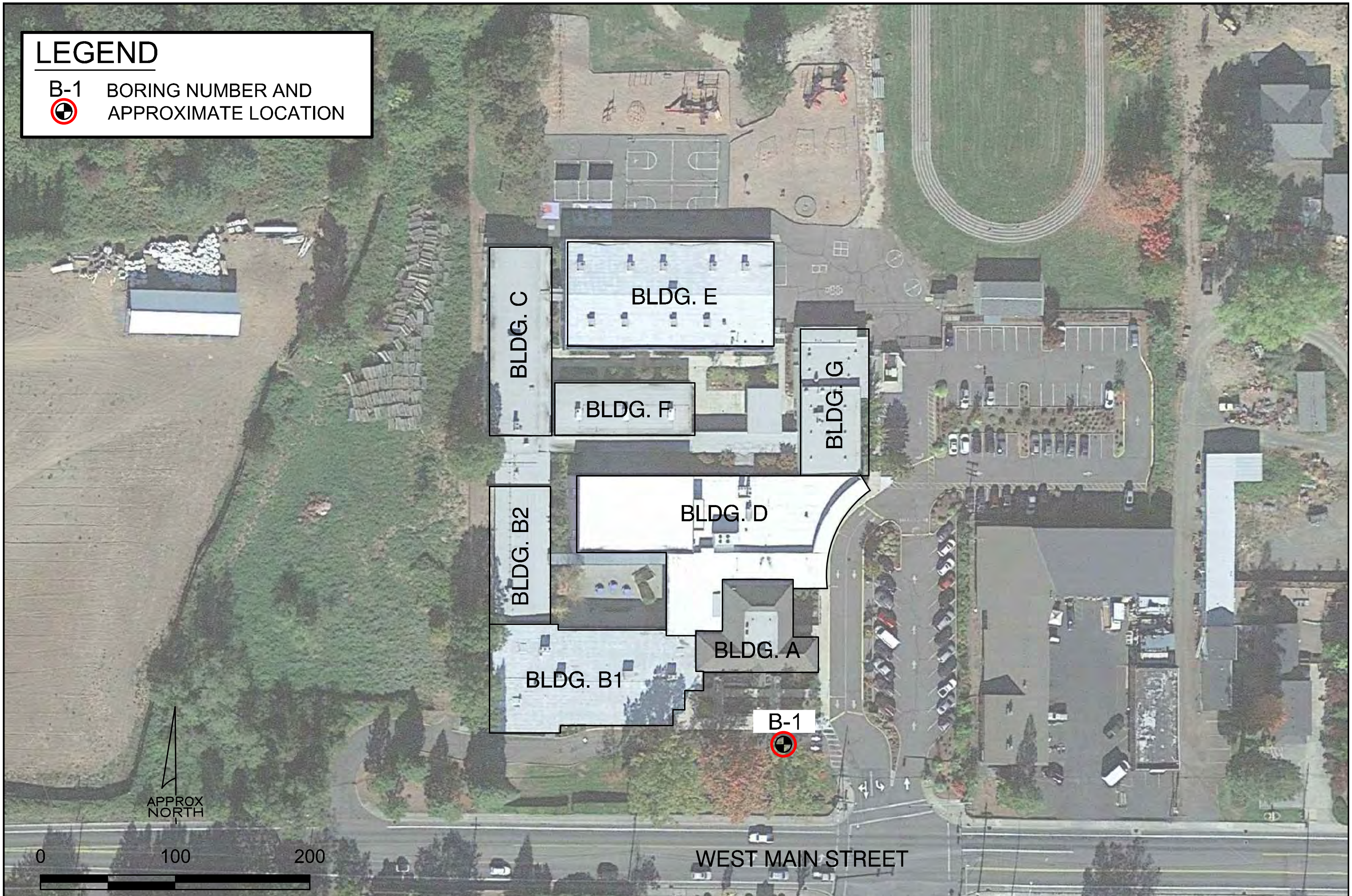
THE GALLI GROUP
GEOTECHNICAL CONSULTING

Dennis Duru, M.S., E.I.T.
Staff Associate

Melvin Galli III, P.E.
Principal Engineer



THE GALLI GROUP GEOTECHNICAL CONSULTING 612 NW 3rd Street Grants Pass, OR 97526	VICINITY MAP OAK GROVE ELEMENTARY SCHOOL MEDFORD, OREGON	DATE: JUNE 2021 JOB NO: 02-6017-02 REV: 6/29/2021 1:15 PM PREPARED BY: BD 6017 Oak Grove Elementary - 01 - Vicinity.dwg	FIGURE: 1



THE GALLI GROUP GEOTECHNICAL CONSULTING 612 NW 3rd Street Grants Pass, OR 97526	SITE PLAN	DATE: JUNE 2021	FIGURE: 2
		JOB NO: 02-6017-02	
	OAK GROVE ELEMENTARY SCHOOL MEDFORD, OREGON	REV: 6/29/2021 1:24 PM	
		PREPARED BY: BD	
		6017 Oak Grove Elementary - 02 - Site plan.dwg	

APPENDIX A

BORING LOGS

THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

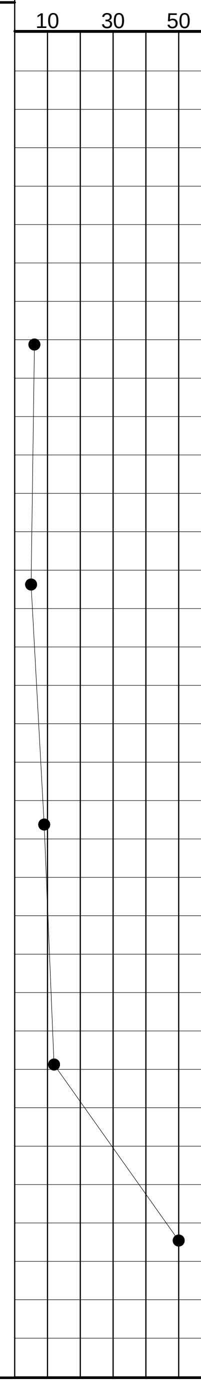
BORING LOG
B-1

Project: Wilson Elementary School
Client: Medford School District
Location: The field on southwest corner of Building I (see Site Plan)
Driller: TGG (Ken, Nick)
Drill Rig: ATV Mounted SSA (4" Diameter)
Depth To Water> Initial : 3.7'

Project No.: 02-6010-01
Date: 6/11/2021
Elevation:
Logged By: Dennis Duru
At Completion : 3.7'

USCS	Description	Depth (ft)	Sample No. and Type	NMC	N
OL	Grass rootzone.	0–0.25			
OH	Medium stiff, dark brown, Clay; moist.	0.25–3.0			
OL/OH	Medium stiff, gray, silty Clay; moist.	3.0–4.0	S-1 (SPT)	29% / 27%	6
SP	Loose to medium dense, brown, Sand; saturated.	4.0–12.5	S-2 (SPT)	31%	5
			S-3 (SPT)	26%	9
			S-4 (SPT)	25%	12
SW/GW	Very dense, brown, Sandy Gravel; saturated.	12.5–12.75	S-5 (SPT)	18%	50
	Bottom of boring at 12.75 feet. Static groundwater at 3.7 feet.				

Standard Penetration Test CURVE



Legend of Samplers: Grab sample SPT sample Shelby tube sample

This information pertains only to this boring and should not be interpreted as being indicative of the site.

THE GALLI GROUP
Geotechnical Consulting

PRELIMINARY SEISMIC RETROFIT STUDY
MCLOUGHLIN MIDDLE SCHOOL
MEDFORD, OREGON

For: Ron Havniear
Medford School District
815 South Oakdale Ave
Medford, OR 97501

c/o Stephen L. Chase, Project Manager
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127 NW D Street
Grants Pass, Oregon 97526

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02-6019-01
June 30, 2021

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PRELIMINARY SEISMIC RETROFIT STUDY
MCLOUGHLIN MIDDLE SCHOOL
MEDFORD, OREGON

1.0 INTRODUCTION

This report presents results of our geotechnical and geological evaluation of the Mcloughlin Middle School for a potential seismic retrofit of the school complex. The subject school is located at 320 W. 2nd Street on the northwest corner of the intersection of West 2nd Street and N. Oakdale Avenue in Medford, Oregon. Please see Figure 1, Vicinity Map, for a more precise location.

The purpose of our investigation and this report was to accomplish a limited site surface and subsurface evaluation (one boring) and conduct a planning level seismic risk assessment (office studies) in order to provide preliminary geotechnical and geologic information and evaluate the likelihood and consequences of geotechnical/geologic related seismic failures, including liquefaction and landslide potential during the design seismic event, for consideration of the potential seismic retrofit.

2.0 SITE AND PROJECT DESCRIPTION

The site is currently occupied by a functioning school, which consists of 7 structures connected via covered walkways or direct connections. The structures are surrounded by play fields, access roads, parking, walkways and open space. The site is relatively flat with undeveloped portions of the site consisting of well-maintained lawn and a few trees.

We understand the district is conducting a preliminary review to determine the level of seismic retrofit needed for the structures on this campus. Their review will largely be based on the evaluation of the potential geologic hazards (such as liquefaction) provided in this report, and an evaluation of the potential structural damage to these facilities associated with the design seismic event. The findings will also likely be used to determine if grant funding will be pursued to complete the seismic retrofit work.

3.0 FIELD EXPLORATION

On June 18, 2021, Associate Staff, Dennis Duru, M.Sc., E.I.T and our drilling crew, visited the site to accomplish the subsurface investigation. One (1) exploratory boring was drilled in the planter area on the east side of Building "A". A utility locate was completed prior to our investigation and our representative coordinated with school personnel to identify the field exploration location away from the marked and known utility locations. See Figure 2, Site Plan, for a layout of the site and the location of the boring. The boring was drilled with our ATV-mounted solid stem auger drill rig and penetrated to depth of 5.5 feet, at which depth drilling operations were stopped due to auger refusal in the dense, silty Gravel. Upon completion, the boring was backfilled with drill spoils.

Standard Penetration Testing (SPT) was accomplished in each boring. This entails driving a 1½-inch diameter steel split spoon sampler by dropping a 140-pound weight for a 30-inch drop. The total number of blows it takes to drive the sampler the last 12 inches of an 18-inch drive is called the SPT N-value. These can be correlated with density and soil strength parameters from testing on thousands of other projects.

Our representative identified the final exploration location, logged subsurface soils and water conditions and obtained soil samples for transport to our laboratory. Visual classifications of the soils were made in the field and are presented in the Boring Logs in Appendix A, at the end of this report.

4.0 LABORATORY TESTING

Moisture content tests were accomplished on soil samples obtained by Standard Penetration Testing. No other tests were accomplished.

5.0 SUBSURFACE CONDITIONS

5.1 SOIL

The subsurface consisted of soft, silty Clay to the depth of 3.0 feet. This was then underlain by medium dense to dense, brown silty Gravel. Review of available deeper boring data showed that the subsurface consists of interbedded layers of Clay and silty Gravel to the depth of approximately 70 feet before encountering Mudstone bedrock which is part of the Hornbrook Formation.

Please see more specific soils information in the Boring Logs in Appendix A. Please note that the soils are shown as distinct layers in the Boring Logs while in nature they may change more gradually. Soils conditions may also change somewhat between the locations investigated.

5.2 GROUNDWATER

Generally, the soils encountered were moist to damp. No groundwater was encountered in the one boring conducted. Review of nearby well log information shows that static groundwater levels typically range from between 6 and 25 feet below ground surface.

6.0 GEOLOGIC OR SEISMIC INDUCED HAZARDS

Summary of Site Geology and Seismicity. The project area is located in the Medford East 30x60 minute USGS topographic quadrangle (see Vicinity Map, Figure 1). Mapped geologic units at the project area consist primarily of Alluvial Fan deposits and bedrock members of the Hornbrook Formation (Wiley et al, 2011). The Marine Sandstone, Siltstone and Conglomerate members are the mapped bedrock unit at the project (Wiley et al, 2011; OGDC-6, 2015). Based on the review of nearby well data, the alluvial deposit extended to the depth of 70 feet on this site before encountering the mudstone bed rock.

The project site is in proximity to several zones of active seismicity. The region is affected by the Cascadia Subduction Zone (CSZ), an active subduction zone off the Oregon coast considered capable of Magnitude 8.5 or greater earthquakes. Average recurrence intervals for such great earthquakes, as determined by recent investigations, range between 300-600 years. The last "great" earthquake was interpreted to be approximately 300 years ago. The CSZ is the main seismic event for consideration for this seismic retrofit.

The project area has an additional tectonic source from earthquakes occurring along active Basin and Range faults as close as 50 kilometers to the southeast. This region has produced numerous earthquakes, including significant events near Klamath Falls and Warner Valley. Such events occur generally once every one to two decades.

Flood Hazard. The site is not near streams or rivers. Therefore, it is not within a 100-year floodplain.

Landslides/Slope Instability. The project site is located within and near the northern edge of a mapped Quaternary landside (Qls). This mapped feature is present on the state landslide database (Statewide Landslide Information Database for Oregon; SLIDO, 2017). Based upon the published mapping, general geomorphology, review of 2-foot contours generated from Lidar datasets (Dogami, 2021) and aerial photos (Google Earth, 2021), as well as subsurface data obtained in this investigation, the mapped landslide in the project area is interpreted to be a relatively deep alluvial fan deposit of material originating upslope.

No recent movement or damage to structures has been associated with this feature in readily available published accounts or our general geotechnical and geologic knowledge of the area. It is therefore assumed this is an inactive “older” deposit. Therefore, in our professional opinion, based on the information from our limited exploration data, the risk of damage due to natural slope instability at this site is considered low. However, any proposed manmade cut or fill slopes should only be made following the recommendations from a detailed geotechnical investigation and report.

Liquefaction and Lateral Spread. The project is underlain by interbedded layer of silty Clay over dense to silty Gravel in the shallow subsurface. Silty Clay and silty

Gravels in a medium dense to dense condition (similar to the ones observed during our limited exploration) have not been known to liquefy in a seismic event. Therefore, liquefaction and lateral spread is considered to be a low to very low potential hazard for this site. See more in Section 7.0 of this report.

Expansive Soils. The upper soils within the subsurface consist of silty Clay. The clays will likely have moderate to high expansion potential based on our experience with soils with similar visual properties.

These expansive soils can have adverse impacts on foundations and all manner of concrete flatwork if the building design does not account for such soils. We recommend that each proposed building site (with clayey soils present) have the soil subgrade examined and tested (Expansion Index) prior to final design and construction. In that way, locations that have expansive soils will have the retrofit structure(s) and drainage designed accordingly.

Construction over expansive soils generally requires embedment of footings to 3 to 4 feet (final depth to be verified after EI testing) below the exterior grade and placement of floor slabs over at least 24 inches or more of compacted rock fill in order to mitigate expansion potential of the underlying soil subgrades. Maintaining the moisture content in the soil to keep it in a moist and fully swelled condition prior to being covered is also critical to proper performance of the structures.

Note: The geotechnical engineer must provide site specific laboratory testing and remedial design recommendations on projects that have potentially expansive, gravelly Clay, clayey Silt or Clay soils present.

Ground Rupture. No Quaternary faults were identified at the project site on review of USGS fault (US Quaternary Faults) maps, and from the one limited exploratory boring. Therefore, the risk of damage at the site due to ground rupture is considered very low.

Ground Shaking: Project structures including foundations and retaining walls must be designed for the potential for very strong ground shaking during the anticipated seismic event. The peak site modified horizontal acceleration (PGA_M), is 0.357g. This is based on the Site Class C designation, determined for the project from subsurface drilling and evaluation of SPT data. The PGA_M value can be used with an appropriate seismic coefficient in pseudo static analysis for design of the seismic upgrades.

Seismic Ground Amplification or Resonance. No unusually hazardous amplification or resonance effects on seismic waves have been associated with soil/bedrock subsurface conditions in the project area.

Tsunami and Seiche. The site is approximately 80 miles inland from the coast, and not subject to tsunami hazard. The site is not located adjacent to a large lake or body of water, and therefore, not subject to seiche hazard.

7.0 LIQUEFACTION EVALUATION

The liquefaction phenomenon occurs in saturated, loose (low density, uncompacted or poorly compacted), cohesionless soils. When loose, cohesionless soils are saturated, which is the case when the soil is below the water table, then water fills the soil pores. In response to compression when a load is applied to the soil, the water increases in pressure and attempts to migrate towards zones of low pressure which is usually upwards towards the ground surface. However, if the applied load is rapid and large enough, or if it is repeated many times (cyclic loading) like in an earthquake, such that there is not enough time for the water to dissipate before the next cycle of loading is applied, then the water pressure may build up to a degree where it becomes greater than the grain-to-grain contact stresses of the soil. The grain-to-grain contact stresses are the source of the shear strength that supports structure foundations and overburden soils in these soil structures. This buildup of excess pore water pressure results in total or partial loss of the soil strength, and the soil may be observed to flow like a liquid, hence "liquefaction". At this point, the soil will lose all its shear strength, be deformed, and will not be able to support structures.

The site is underlain by dense silty Gravel. No groundwater was observed in the boring to the depth drilled. The conditions for liquefaction to occur were not observed at this site during our limited subsurface investigation. Such soils will not undergo further densification enough to cause liquefaction during a seismic event due to the high silt/clay and gravel content, and also due to the dense condition. Therefore, the possibility of liquefaction that could adversely affect the site is very low. However, the upper 3.0 feet of the soil may experience slight settlement during the design seismic event.

Therefore, in our professional opinion, the site conditions found in the boring will not result in wide spread liquefaction that will have significant adverse impacts on the structures during a seismic event.

8.0 CONCLUSIONS

In our professional opinion, based on our field investigation, office review and previous work in the area, the soils conditions at the site are suitable for a conventional seismic retrofit. Crushed rock structural fill over the silty Sand and Gravel will provide adequate support of new foundations, grade beams and/or buttresses. In our opinion, this school site is not subject to large-scale liquefaction that will cause a significant adverse impact to the structures.

Additional borings around the structures on this site could possibly find zones of soils that may liquefy. However, these are likely to be moderate to small in size and should not adversely impact the structure.

If a full seismic retrofit geotechnical design report is needed, additional tasks to be accomplished would be as followed:

1. 2 or 3 additional borings.
2. Laboratory testing for determining strength and settlement characteristics of the site soils.
3. Evaluation of data for developing design parameters.

These could be used to provide a full scale Seismic Retrofit Design Report.

8.1 LIMITATIONS

The analyses, conclusions and recommendations contained in this report are based on site conditions as they existed at the time of the study, and assume soils, rock and groundwater conditions exposed and observed in the boring during our investigation are representative of soils and groundwater conditions throughout the site. If during construction, subsurface conditions or assumed design information is found to be different, we should be advised at once so that we can review this report and reconsider our recommendations in light of the changed conditions. If there is a significant lapse of time (5 years) between submission of this report and the start of work at the site, if the project is changed, or if conditions have changed due to acts of God or construction at or adjacent to the site, it is recommended that this report be reviewed in light of the changed conditions and/or time lapse.

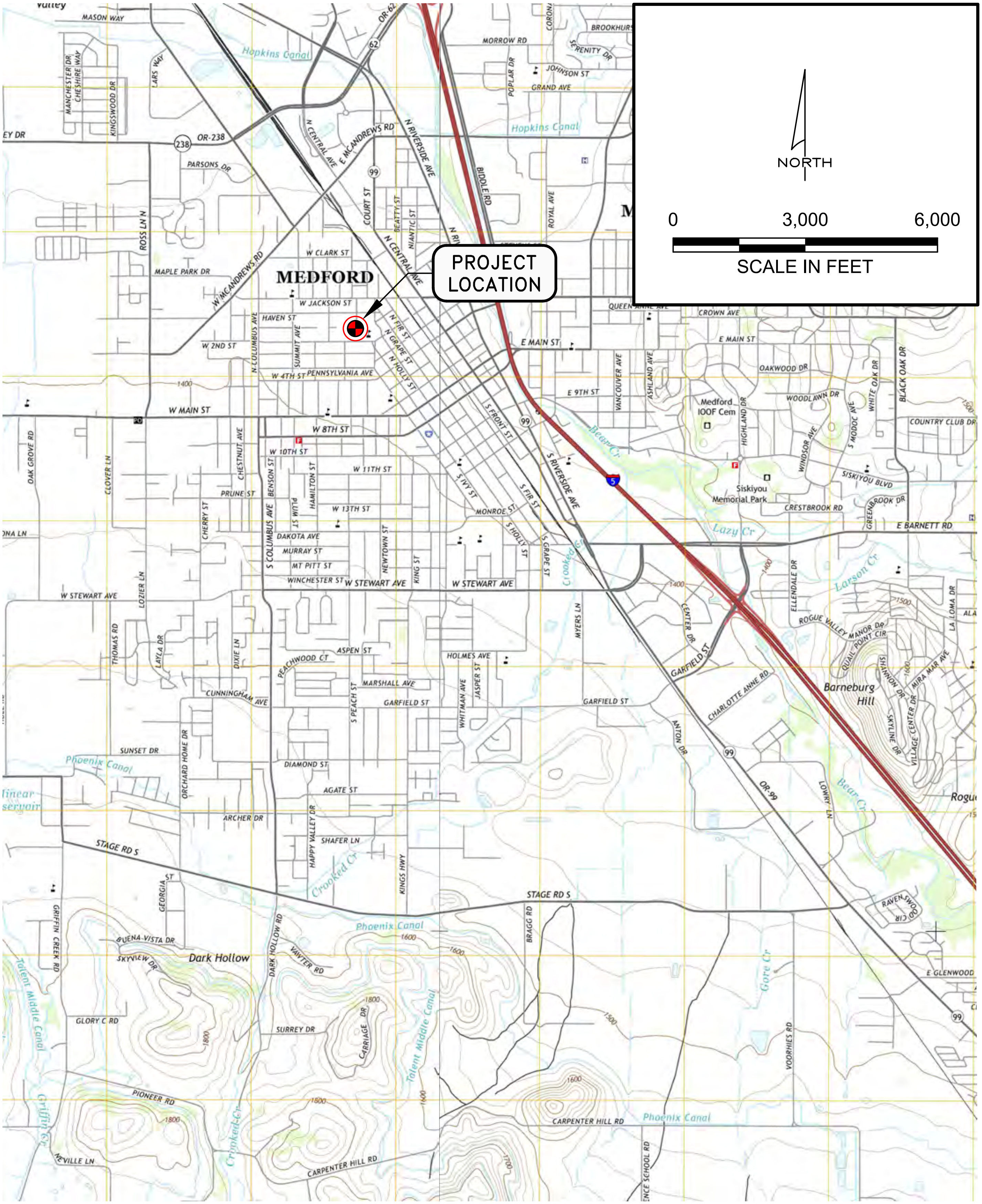
This report was prepared for the use of the School District and their design team for evaluating the need for a full scale Seismic Retrofit evaluation and report. It should be made available to contractors for information and factual data only. This report should not be used for contractual purposes as a warranty of site subsurface conditions. It should also not be used at other sites or for projects other than the one intended.

We have performed these services in accordance with generally accepted geotechnical engineering and professional geology practices in southern Oregon, at the time the study was accomplished. No other warranties, either expressed or implied, are provided.

THE GALLI GROUP
GEOTECHNICAL CONSULTING

Dennis Duru, M.S., E.I.T.
Staff Associate

Melvin Galli III, P.E.
Principal Engineer



THE GALLI GROUP GEOTECHNICAL CONSULTING 612 NW 3rd Street Grants Pass, OR 97526	VICINITY MAP MCLOUGHLIN MIDDLE SCHOOL MEDFORD, OREGON	DATE: JUNE 2021 JOB NO: 02-6019-02 REV: 6/29/2021 1:51 PM PREPARED BY: BD 6019 McLoughlin MS - 01 - Vicinity.dwg

FIGURE: 1

THE GALLI GROUP
GEOTECHNICAL CONSULTING
612 NW 3rd Street
Grants Pass, OR 97526

SITE PLAN

MCLOUGHLIN MIDDLE SCHOOL
MEDFORD, OREGON

DATE: JUNE 2021
JOB NO: 02-6019-02
REV: 6/29/2021 2:01 PM
PREPARED BY: BD
6019 McLoughlin MS - 02 - Site plan.dwg

FIGURE: 2

APPENDIX A

BORING LOGS

THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

BORING LOG
B-1

Project: McLoughlin Middle School
Client: Medford School District
Location: Planter area east of Building A (see Site Plan)
Driller: TGG (Ken, Nick)
Drill Rig: ATV mounted rig
Depth To Water> Initial :

Project No.: 02-6019-01
Date: 6/18/2021
Elevation:
Logged By: Dennis Duru
At Completion :

USCS	Description	Depth	Sample No. and Type	NMC	Standard Penetration Test N
OL	grass rootzone.	0 – 0.25			
CH/MH	Soft, brown, silty Clay; wet to moist.	0.25 – 3.0			
ML	Very stiff, brown, gravelly Silt; damp.	3.0 – 5.0	S-1	15%	46
			S-2	11%	33
GM	Dense, brown, silty Gravel; damp.	5.0 – 5.5			
	Bottom of boring. No free groundwater encountered.	5.5			

Legend of Samplers: Grab sample SPT sample Shelby tube sample

This information pertains only to this boring and should not be interpreted as being indicative of the site.

GEOTECHNICAL & SEISMICITY INVESTIGATION REPORT
SEISMIC RETROFIT DESIGN
MSDEC GYMNASIUM
MEDFORD, OREGON

For: Ron Havniear
Facility Manager
Medford School District
Medford, OR 97501

By: **THE GALLI GROUP**
612 NW Third Street
Grants Pass, OR 97526
(541) 955-1611

02-5760-02
November 25, 2020

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THE GALLI GROUP
Geotechnical Consulting

GEOTECHNICAL & SEISMICITY INVESTIGATION REPORT SEISMIC RETROFIT DESIGN MSDEC GYMNASIUM MEDFORD, OREGON

1.0 INTRODUCTION

This report presents results of our geotechnical and seismicity evaluation of the site for the seismic retrofit to the MSDEC Gymnasium in Medford, Oregon. The subject property is located on the north side of the extension of West Barnett Street, between S. Oakdale Avenue and Kenyon Street. This site is mildly sloping to flat. Please see Figure 1, Vicinity Map, for a more precise site location.

The purpose of this investigation and report was to evaluate the site surface and subsurface conditions with three (3) new borings and one (1) older boring, and conduct office studies in order to determine potential geologic hazards and to provide geotechnical and seismic recommendations for design and construction of the seismic retrofit. This may include improving the foundations, the use of driven piles and/or adding embedded footings/buttresses for lateral and vertical resistance of loads generated in a seismic event.

2.0 SITE AND PROJECT DESCRIPTION

The site is currently occupied by a functioning High School, which apparently contains one main large structure with various wings, a large gymnasium structure, an outdoor play area, a sports stadium, drive lanes and parking lots.

We understand the project to consist of carrying out a Seismic Retrofit to the large gymnasium structure in order to bring it up to current code requirements for public schools. This will likely require structural upgrades, including improved foundations and/or embedded footings/buttresses for resistance of lateral and vertical loads generated in a seismic event.

3.0 FIELD EXPLORATION

On November 17, 2019, Staff Associate, Dennis Duru, M.Sc. E.I.T. and our drilling crew, visited the site to accomplish the subsurface investigation. A total of three (3) new exploratory borings were drilled to depths between 7.5 and 21.5 feet around the structure at the locations shown on Figure 2, Site Plan. **Note:** One additional older boring at the

NE corner of the gymnasium was also used in this report. The drilling was accomplished with our ATV-mounted solid stem auger drill rig. Borings terminated in the dense Sand and Gravel, very stiff Silt or hard, gravelly Silt & Clay. All holes were refilled after drilling with drill spoils. The areas were left clean of most soil debris.

A utility locate was completed prior to our investigation and our representative identified the field exploration locations away from the marked utilities. Borings were advanced with sample collection and testing being accomplished at various depths. Standard Penetration Testing (SPT) was accomplished in each boring. This entails driving a 1½ inch I.D, 2-inch O.D., steel split spoon sampler by dropping a 140-pound weight for a 30-inch drop. The total number of blows it takes to drive the sampler the last 12 inches of an 18-inch drive is called the SPT N-value. These can be correlated with soil strength and density parameters from testing on thousands of other projects.

Our representative identified the final exploration locations away from utilities, logged subsurface soils and water conditions and obtained soil samples for transport to our laboratory. Visual classifications of the soils were made in the field and are presented in the Boring Logs in Appendix A, at the end of this report. Please note that in the logs, soil changes are depicted as distinct layers, while in nature they likely are more gradual.

4.0 LABORATORY TESTING

Soil samples were tested for natural moisture content. One Expansion Index (EI) test was accomplished on a combined bulk sample from shallow depth in B-2 at 2.5 to 4.0 feet. Results were an EI=20. A previously run EI test on a sample from B-1 had a tested EI = 64. This means the upper soils are moderately expansive (change in volume with change in moisture content). The deeper, lighter brown soils are mildly expansive.

5.0 SUBSURFACE CONDITIONS

5.1 SOIL

The borings encountered multiple layers of surficial soil and crushed rock fill underlain by loose to very dense, clayey Sand with gravels, clayey Gravel, soft to stiff Silt and Silt and Clay and gravelly Silt. B-1 terminated at 10.5 feet in very stiff Clay below a 3.3 foot layer of medium dense to dense, silty Sand and Gravel. B-2 terminated in very dense, silty Gravel at 7.5 feet (auger refusal). B-3 terminated in very stiff Silt and Clay at 21.5 feet. B-4 terminated in hard, gravelly silt at a depth of 11.5 feet. These soils were moist to wet or saturated as they became deeper. Please see more specific soils information in the Boring Logs in Appendix A. Please note that the soils are shown as distinct layers in the Boring Logs while in nature they may change more gradually. Soils conditions may also change somewhat between the locations investigated.

5.2 GROUNDWATER

Generally, the upper soils encountered were moist and the deeper soils are wet to saturated. Groundwater was encountered at 4.5 feet in B-3 and 8.0 feet in B-4. Water was not encountered in B-1 and B-2. During wet weather, water will tend to perch on the top of the hard Silt and dense gravels. The surficial silty soils could become unworkable when they become saturated during the wetter months of the year.

6.0 SITE SPECIFIC GROUND MOTION HAZARD ANALYSIS

Site Specific Ground Motion Hazard Analysis was carried out in order to meet the requirements of the new ASCE 7 (2016), as specified in (OSSC, 2019), that a site specific study is required for structures on sites with a Site Class D or E with S_1 greater than or equal to 0.2g and Site Class F. Based on site reconnaissance, desk study and subsurface exploration, the subject site was determined to have a Site Class D in accordance to the Site Classification Procedures for Seismic Design set forth in the ASCE 7-16 Chapter 20. Therefore, a Ground Motion Hazard Analysis is required to determine the design acceleration parameters for the structure.

Probabilistic ground motion hazard analysis was carried out in accordance to ASCE 7-16 section 21.2. Deterministic ground motion spectrum was not calculated because of an **EXCEPTION** to the ASCE 7-16 Section 21.2.2 (See Supplement 1 Page S-6 in ASCE 7-16), that the deterministic ground motion spectrum need not be calculated when the largest spectral response acceleration of the probabilistic ground motion is less than $1.2F_a$. Careful consideration was given to the requirements of Section 21.3 in choosing the final Design Response Spectrum.

6.1 PROBABILISTIC GROUND MOTION HAZARD ANALYSIS

The probabilistic ground motion hazard analysis was accomplished consistent with Method 1 in the ASCE 7-16 section 21.2.1.1. We utilized the US Geologic Survey (USGS, 2014) Unified Hazard Tool to compute the Uniform Hazard Response Spectrum that has a 2% probability of exceedance within a 50 year period for the site. The site Risk Coefficients, C_{Rs} and C_{R1} was obtained from Figure 22-18 and 22-19 of the ASCE 7-16 respectively. The C_R was then calculated as described in the section 21.2.1.1. A scale factor recommended in Section 21.2 was introduced to scale the response spectrum to the maximum response. The probabilistic site specific MCE_R at any period, was determined as the product of the Uniform Hazard Response Spectrum, the C_R and the scale factor at that period. The site specific MCE_R spectral accelerations from the probabilistic ground motion hazard analysis is given in Figure 1.

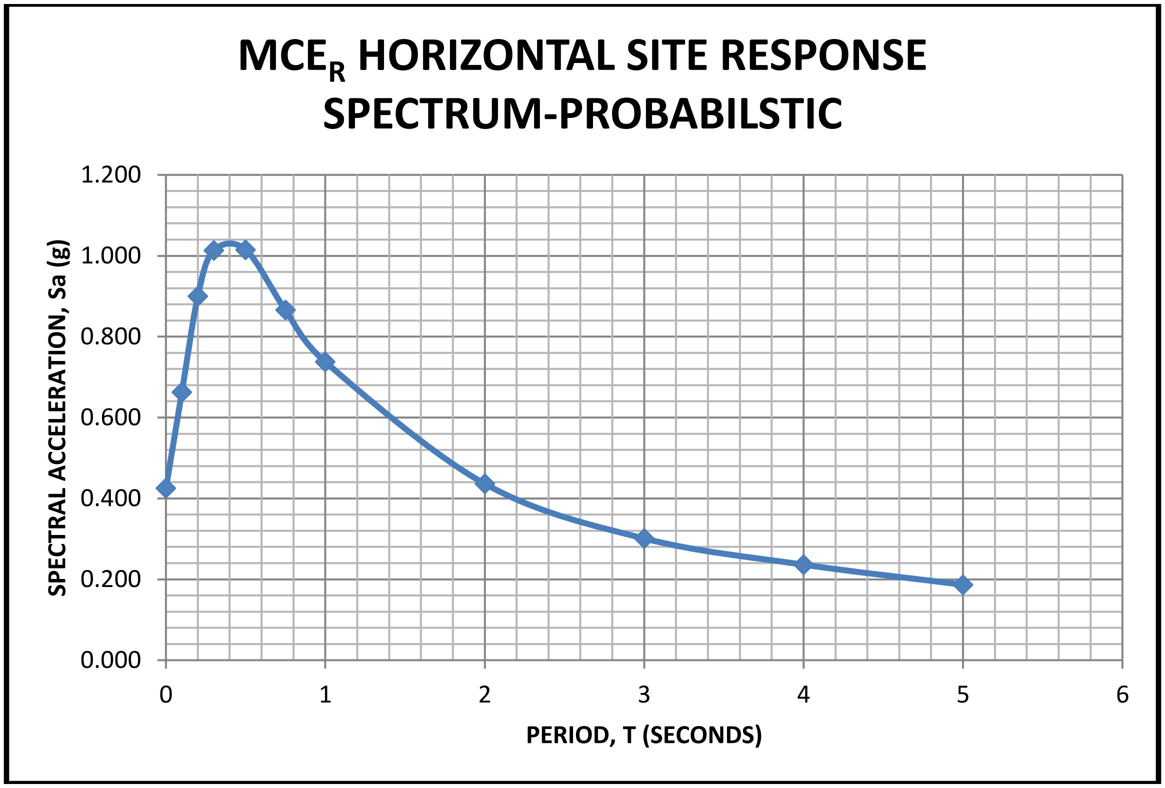

Figure 1. Site specific MCE_R spectral accelerations from the probabilistic ground motion hazard analysis.

6.2 DETERMINISTIC GROUND MOTION HAZARD ANALYSIS

There is an **EXCEPTION** according to the ASCE 7-16 Section 21.2.2 (See Supplement 1 Page S-6 in ASCE 7-16), that the deterministic ground motion spectrum need not be calculated when the largest spectral response acceleration of the probabilistic ground motion is less than $1.2F_a$. The short period site coefficient F_a for the project site is 1.297, and the largest spectral response acceleration determined through the probabilistic ground motion analysis is 1.015g. Therefore 1.015g<1.56 and the Deterministic Ground Motion Spectrum does <u>not</u> need to be calculated.

6.3 SITE SPECIFIC MCE_R

The Site Specific MCE_R spectral response acceleration at any period, S_{am}, is taken as the lesser of the spectral response accelerations from the probabilistic ground motion hazard analysis and the deterministic ground motion hazard analysis in accordance to section 21.2.3 of the ASCE 7-16. The Recommended site specific MCE_R is given in Figure 2.

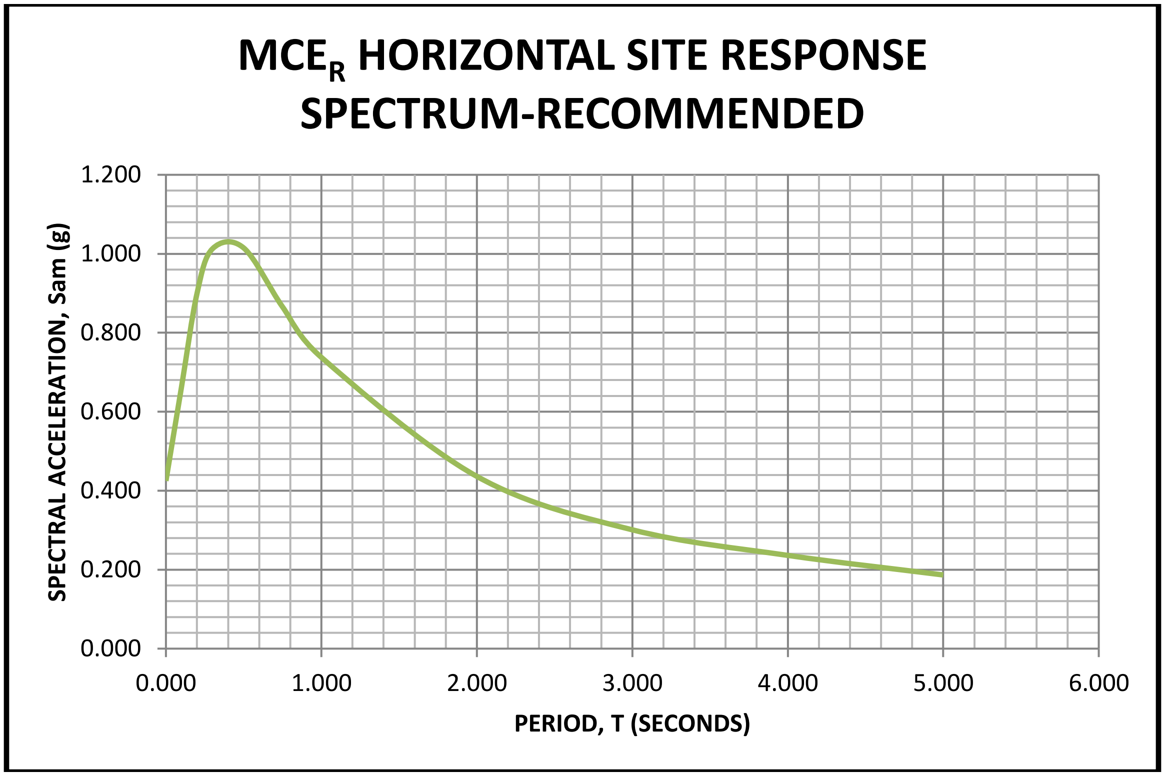

Figure 2. Recommended MCE$_R$ Response Spectrum from the Site-Specific Ground Motion Hazard Analysis

6.4 RECOMMENDED DESIGN ACCELERATION PARAMETERS

The design earthquake parameters for the project area are based upon the methodology set forth in 2019 Oregon Structural Specialty Code and ASCE 7-16 Section 21.3 and 21.4, and on the result of the ground motion hazard analysis. Recommended design acceleration parameters are provided in Tables 2 below.

Table 1. Recommended Design Acceleration Parameters

SITE SPECIFIC GROUND MOTION HAZARD ANALYSIS: MSDEC GYMNASIUM (02-5760-02)	
Project Area: Medford, Oregon	Latitude: 42.316052
	Longitude: -122.871748
Risk Category	III
Site Class	D
Ss=MCER Ground Motion (period=0.2s)	0.628 g
S1=MCER Ground Motion (period = 1.0s	0.361 g
Design Spectral Acceleration **SDS**, Short Period (ASCE 7-16 Section 21.4)	0.609 g
Design Spectral Acceleration **SD1**, 1 sec Period (ASCE 7-16 Section 21.4)	0.567 g
Spectral Response Acceleration, **SMS**, Short Period SDS*1.5 (ASCE 7-16 Section 21.4)	0.913 g

Spectral Response Acceleration, **SM1**, 1 sec Period (ASCE 7-16 Section 21.4)	0.850 g
MCEG adjusted for site class effects, PGA_M	0.386 g
Seismic Design Category (Table 11.6-1 and 11.6-2 ASCE 7-16)	**D**
Per the requirements of Section 11.6 of the ASCE 7-16 code, the is SDS>0.5 and SD1>0.2, therefore **SEISMIC DESIGN CATEGORY** is **D**	

6.5 GEOLOGIC OR SEISMIC INDUCED HAZARDS

Landslides/Slope Instability. There are no slopes in the immediate vicinity of the site. Therefore, there is no possibility of slope failure, rock fall or slide run out damage.

Expansive Soils. The upper soils encountered are mildly to moderately expansive. The Expansion Index tests yielded EI=20 and EI = 64.

Liquefaction and Lateral Spread. The upper parts of the project's subsurface is underlain by silty, clayey Sand and Gravel and combinations of silt and clay. These were medium dense to very dense and very stiff. These soils are not subject to liquefaction. Therefore, mitigation for potential liquefaction is not required.

Ground Rupture. No Quaternary faults were identified at the project site. Therefore, the risk of damage at the site due to ground rupture is considered very low.

Ground Shaking. A Site Class of D should be used for the project, based on a field reconnaissance, subsurface SPT data obtained at the project and our Ground Motion Analysis. Seismic design recommendations are provided in Table 1. The PGA_m for the site is 0.386g.

Seismic Ground Amplification or Resonance. No hazardous amplification or resonance effects from seismic waves have been associated with the soil subsurface conditions in the project area. Some amplification is possible. However, the design parameters in the Seismic Design table should account for such added loading. The risk of damage at the site from unexpectedly severe shaking due to seismic wave amplification is low.

Tsunami and Seiche. The site is approximately 80 miles inland from the coast, and not subject to tsunami hazard. The site is not located adjacent to a large lake or body of water, and, therefore, not subject to seiche hazard. Therefore, risk of damage at the site due to catastrophic flooding is near zero.

Summary. Other than moderately hard shaking during a seismic event and a moderately expansive clay soil matrix of the clayey gravels, there are no substantial geologic hazards that would cause damage to the structure.

7.0 CONCLUSIONS

In our professional opinion, based on our field investigation, and office review, and previous work in the area, the soils conditions at the site are suitable for the proposed seismic retrofit, provided the recommendations of our report are incorporated in the design and construction of the project. Crushed rock backfill over the medium dense silt/clay sand and gravel mixtures or hard Silt and very stiff Clay, on driven pipe piles will provide good foundation and buttress support.

8.0 GEOTECHNICAL RECOMMENDATIONS

The subject site has reasonably acceptable soils for support of the structure. The following sections provide methods and data for proper design of the seismic retrofit items.

8.1 SITE PREPARATION AND GRADING

This structure has areas with exterior walkways and landscaping that may require structural and foundation upgrades for the seismic upgrade. Therefore, normal methods of demolition, debris removal, clearing, grubbing, stripping for organic removal and subgrade soil overexcavation and preparation will apply in areas of the work.

8.1.1 Manmade Fill & Debris Considerations

All old fill and debris encountered during construction must be removed. Soil that is clear of debris could be used in landscape berms. All other debris or debris laden soil must be wasted off site. The full extent of any waste fill removal (if any) will be determined during site stripping and excavation operations.

8.1.2 Clearing, Grubbing and Stripping

The site area that will be used for the work must be stripped and cleared of all vegetation, sod and organic topsoil. Additional stripping (or excavations) will most likely be required to remove root balls from bushes (very little of this is expected) and overexcavation for crushed rock backfill. The stripped materials removed shall be hauled from the site or stockpiled for use in landscape areas only (such as landscape mounds). This material must <u>not</u> be used as structural fill.

Holes or depressions resulting from the removal of underground obstructions or old fill and debris that extend below the finish subgrade and will be beneath support items shall be cleared of all loose material and dished to provide access for compaction equipment. These areas shall then be backfilled and compacted to grade with structural fill, as described later in this report.

It is recommended that grubbing and stripping of the site, old fill removal and compaction of depressions below finish subgrade, be observed and/or decided by the geotechnical engineer or his representative from The Galli Group.

8.2 STRUCTURAL FILL PLACEMENT AND COMPACTION

8.2.1 Beneath Structures

Structural fill is defined as any fill placed and compacted to specified densities and used in areas that will be under foundations, slab-on-grade floors or structure support items. It appears that new footings and buttresses will have structural fill below, beside and/or above them. The subgrade needs to be prepared properly and the fill must be placed and compacted correctly for proper long-term performance.

Structural Fill Materials. Ideally, and particularly for wet weather construction, structural fill must consist of a free-draining granular material (non-expansive) with a maximum particle size of six inches. The material shall be reasonably well-graded with less than 5 percent fines (silt and clay size passing the No. 200 mesh sieve). During dry weather, any organic-free, non-expansive, will less than 12% passing the No. 200 sieve, compactable granular material, meeting the maximum size criteria, is typically acceptable for this purpose. Locally available crushed rock, jaw-run crushed "shale" (low-grade rock) and good quality Sandy Decomposed Granite [DG] (dry weather only) have performed adequately for most applications of structural fill. See Section 8.6 for import fill specifications (aggregate base, aggregate subbase, sandy Gravels and Decomposed Granite).

Note: It should be noted that some areas to be overexcavated and backfilled with Structural Fill will have limited access for equipment. Therefore, use of smaller crushed rock (3/4" or 1" minus) would allow use of smaller hand-held compaction equipment on thin lifts to attain proper compaction. Use of 2 or 3 sack sand slurry or cement treated crushed rock would also work well in these "tight" spaces. The reduction in labor could offset the higher cost of these cement-laden fill materials and "compaction" is usually easy.

Structural Fill Placement. All structural fill shall be placed in horizontal lifts not exceeding 8 inches loose thickness (less, if necessary to obtain proper compaction) for heavy compaction equipment and three to four inches for light and hand-operated equipment. Each lift must be compacted by mechanical means to a minimum of 98 percent of the maximum dry density, as determined by ASTM Test Method D-698 (Standard Proctor).

Beneath Footings. Structural fill placed beneath footings or other structural elements must extend beyond all sides of such elements a distance equal to at least ½ the total depth of the structural fill beneath the structural element in question for vertical support (i.e. for 2 feet of structural fill beneath footings, extend the fill at least 1 foot past all edges of the footing).

To facilitate the earthwork and compaction process, the earthwork contractor shall place and compact fill materials at or slightly above their optimum moisture content. If fill soils are too high on the wet side of optimum, they can be dried by continuous windrowing and aeration or by intermixing lime or Portland Cement to absorb excess moisture and improve soil properties. If soils become dry during the summer months, a water truck should be available to help keep the moisture content at or near optimum during compaction operations.

Fill Placement Observation and Testing Methods. The required construction monitoring of the structural fill utilizing standard nuclear density gauge testing and standard laboratory compaction curves (ASTM D-698 specified) is applicable to materials 2-inch size and smaller. Larger (2½" or above) jaw-run "shale", crushed rock or larger broken decomposed granite (DG) do not yield consistent results with this type of testing. The high percentage of rock particles greater than ¾'s of an inch in these materials causes laboratory and field density test results to be erratic and does not provide an adequate representation of the density achieved. Therefore, construction specifications for this type of material typically specify method of placement and compaction coupled with visual observation during the placement and compaction operations of lifts, instead of nuclear density testing.

Observation of Fill Placement. For the larger rock materials, we recommend the 8-inch lift (after being "worked in") be compacted by a minimum of 3 passes with a heavy vibratory roller or large equipment-mounted vibratory plate. One "pass" is defined as the roller moving across an area once in both directions. **Note:** Much of the work on this seismic upgrade project will likely consist of narrower (2 to 4 feet wide) strip footings. Therefore, a hoe pack or hand operated compactors will likely be needed. Therefore, use of smaller rock materials will be helpful. The placement and compaction must be observed by our representative. After compaction, as specified above, is completed the entire area shall be proof tested with vibratory compactors while being observed by the geotechnical representative. *Observing the hoepak work the top of the lifts with heavy down pressure will be sufficient for density verification.*

Nuclear Density Testing of Fill. Field density testing by nuclear density gage would be adequate for verifying compaction of 2-inch to ¾-inch minus crushed base rock, expansive clay and silt soils, Decomposed Granite and other materials 2 inches or smaller in size. Therefore, typical % compaction specifications would suffice. Testing must be accomplished in a systematic manner on all lifts as they are placed. Testing only the upper lifts is not adequate.

Cement Treated Fill Materials. When either 2-sack sand slurry or a compacted cement treated base rock is used, density verification is relatively easy. Observation of placement and casting cylinders for strength testing of the slurry is acceptable. The slurry should have a break strength at 28 days of at least 500 psi. Cement treated (7% by weight) crushed rock shall be compacted in 4" lifts with hand equipment can be observed during compaction. Samples shall be taken for strength testing. Test values at 14 days shall be at least 250 to 300 psi.

8.2.2 Non-Structural Fill

Any waste soil, organic strippings or other deleterious soil (such as wet or dried out expansive clay) would be considered non-structural fill. These materials may make reasonable landscape soils and lawn topsoil material. This material may be placed in landscape areas and waste soil areas such as berms with slopes at 3.0H:1.0V or flatter. It must not be placed under footings or other structural members. It is recommended that when these soils are used they be given a moderate level of compaction (92 to 94 percent) to help seal them from surface water.

8.3 FOUNDATION SUPPORT

During the site investigation, borings encountered somewhat expansive, clayey silt with abundant gravels and dense, clayey and silty sand and gravel. One boring had soft clayey silt soils that will provide poor footing support and must be removed beneath all foundations in that area. Conventional spread footings on compacted crushed rock over the medium dense silty/clayey sand and gravel soils are adequate for the proposed retrofits. Therefore, foundations must be founded on at least 24 inches of compacted structural rock fill which extends to a depth of at least 4 feet below the surface (but must also have a firm, stable subgrade).

8.3.1 Footings on Compacted Crushed Rock

1. Excavate footings a minimum of 24 inches below bottom of footing, penetrating into firm, stable subgrade soils (below any soft soil zones).
2. All areas over-excavated beneath footings must extend beyond all edges of the footing a distance equal to at least ½ the depth of the structural fill to be placed (i.e., 2 feet deep over-ex and rock fill requires the width to be 1-foot wider on all sides of the footing).
3. Redensify the footing subgrade disturbed during excavation.
4. Place a woven geotextile support fabric (ACF 180 or equivalent) on the compacted subgrade; pull tight.
5. Backfill with at least 24 inches of compacted crushed rock structural fill, compacted to at least 98% of ASTM D-698 (initial 12" may be compacted to 95% of ASTM D-698).
6. Footings placed on the crushed rock as listed above may be designed for a bearing pressure of 2,000 pounds per square foot. This may be increased by 33% for <u>transitory</u> wind and seismic loading.
7. The base of spread footings shall be buried a minimum of 16-inches below finish grade in order to provide lateral support and frost protection.
8. We recommend minimum lateral dimensions of 18 inches for continuous load bearing footings and 30 inches for isolated piers constructed in this manner.

Anticipated Settlements. For properly constructed foundations founded on redensified and structural fill covered native soils, we anticipate maximum total and differential settlement to be less than approximately 7/8-inch and 1/2-inch, respectively.

Foundation Drains. We typically recommend all footings be installed with a footing drain to intercept groundwater seepage. Footing drains consisting of a rigid, smooth-wall perforated pipe surrounded by drain rock (one side and above), all wrapped in a non-woven geotextile fabric and should be placed adjacent to the footings. This is addressed more fully later in this report (Section 8.5). Please see Figure 3.

8.3.2 Driven Small Diameter Pipe Piles

New footings placed for this upgrade, if subjected to long-term loading, will undergo some amount of settlement. If damage from settlement is unacceptable these new footings should be supported on driven, small diameter, steel pipe piles or embedded into the dense or hard underlying soils.

The small diameter pipe piles have turned out to be the most cost-effective method for support of moderately heavy footings in such soil conditions. These shall be designed and installed as listed below.

Pipe Pile Design

- Driven 3” or 4” diameter galvanized steel pipe piles.
- Standard wall thickness (Sch 40; 0.237" wall thickness).
- Drive closed ended; anticipated depth is 15 to 25 feet or less.
- Utilize vibratory driver sized for 3” and 4” pipe piles (1,100 pound class).
- Final set criteria; drive until less than 1 inch of advancement in 8 seconds or more of continuous driving.
- Pile Top; new construction cap of 3/8”$_t$ x 4" x 4" steel plate for each pile (or per Structural Engineer).
- Use sleeved friction couplers; piles are for vertical compression load only (no Uplift Load Capacity).
- Pile capacity is 15 kips (3”) and 20 kips to 24 kips (4”), depending on load test with Factor of Safety of 2.0+; pile load tests (3) should be accomplished at time of production driving; this is strongly recommended.
- Typical Spacing; 4 to 6 feet along strip footings; depending upon the loads above.
- Minimum pile spacing must be at least 2 feet.
- Multiple piles may be used beneath larger spread footings depending upon the load and stability needed.

Embed top of pile with reaction plate at distance up into footing or grade beam as recommended by project structural engineer (usually 6" to 8" depending upon footing/grade beam thickness).

Settlement. We anticipate total and differential settlement of the structure when using the pipe piles would be less than 1/2 inch and 1/4 inch, respectively.

8.4 LATERAL LOAD RESISTANCE

8.4.1 Foundation Members

Lateral loads exerted upon these members can be resisted by passive pressure acting on buried portions of the foundations and other buried structures, by friction between the bottom of structural elements and the underlying soil and by batter piles or pile top resistance against the soil.

We recommend the use of passive equivalent fluid pressures of the following values for portions of the structure and foundations embedded into the native soils.

- Native Soils (ignore upper 18”) 200 pcf
- Dense Compacted Crushed Rock (below 18”) 400 pcf

A coefficient of friction of 0.55 can be used for elements poured neat against angular crushed rock structural fill. These should be reduced to 0.20 for areas over a vapor barrier or 0.35 over native soils.

8.4.2 Buttress or Thrust Block

Concrete buttresses or thrust blocks may also be used for lateral resistance. These consist of an embedded concrete deadman block of reinforced concrete that can resist loads in all directions. These can be designed utilizing the lateral bearing capacities provided below.

Lateral Bearing Capacity

Depth	Capacity (ksf)
2 to > 4 feet	0.5 up to 10 kips
4 to > 6 feet	1.25 up to 20 kips
6 to > 8 feet	2.0 up to 30 kips

(1) These must be poured “neat” against the dense clean native excavation sidewall.
(2) These may also be formed and poured and then backfilled around with compacted crushed rock Structural Fill.

8.4.3 Pile lateral Resistance

Lateral loads may be resisted by the pipe piles in two different ways.

Pile/Soil Interaction. The first is by direct resistance of the top of the pile due to soil resistance. Based on lateral resistance tests, we have found the pipe piles will have the following resistance with no more than 1 1/2 inch lateral displacement at the top (pile top fixed in the pile cap/footing/grade beam).

Pile Size	Lateral (Kips)
3” pile	1.0
4” pile	1.4

These load resistance values will be in addition to the passive pressure acting on sides of the footing/grade beam given above.

Batter Piles. The second method would be to use batter piles. These shall be installed and designed as listed below:

- Install at up to 30° off the vertical.
- Drive pile to embedment into the dense sand and gravels or hard Clay and Silt.
- Lateral load resistance for pile with top fixed into the pile cap/footing shall be the computed lateral component of the axial load capacity based on angle off the vertical. Use an additional Factor of Safety of 1.5.
- Example: 3” piles installed at 20° off the vertical would provide lateral resistance of 15 kips x Sin 20° x 1/1.5 Safety Factor. This would be 3.4 kips of lateral resistance. This pile would also provide Cos 20° x 15 kips = 14.1 kips of vertical capacity.

We recommend we have the opportunity to review the pile layout and verify batter pile capacity if these are used. **Note:** For batter piles to perform properly the pile cap/grade beam on top must be subjected to a vertical compression load as well as the lateral load to be resisted. The vertical load must be greater than the horizontal load by a factor of 1.5.

8.5 FOUNDATION DRAINS

All exterior foundations and embedded structures should have proper drainage.

Footing Drains. Foundation drainage should consist of a rigid smooth wall perforated pipe surrounded by at least 6 inches of drain rock on top and on one side, all wrapped in a non-woven geotextile, designed as a filter fabric (such as Mirafi 140N or equivalent). The perforated pipe should be located on the footing next to the stem wall (or beside the footing), provided this is at least 12 inches below underslab drain rock. See Figure 3.

All drains shall be tightlined and positively sloped to an approved stormwater disposal location into the public storm drain system or detention pond. **Note:** In no case shall water be collected and/or directed or discharged close to the foundations. Such improper water discharge can cause added water related problems.

We strongly recommend against connecting roof drains or surface area drains to foundation drains unless to a discharge line away from the structure. Foundation drains should consist of rigid smooth-wall perforated pipe. The rigid smooth-wall pipe can be cleaned out by means of a “roto-rooter” type system should it become plugged with sediment or fine roots. We recommend cleanouts be placed periodically by the designer to facilitate cleaning and maintenance of the drains.

8.6 MATERIAL SPECIFICATIONS

The following materials specifications shall apply to the materials as used on this project.

Aggregate Base Rock (Acceptable for Structural Fill)

- Angular Crushed Rock (¾ or 1" Minus); R=85 or greater; Well Graded (No Gaps and at least 60% retained on the No. 4 sieve)
- Exceeds the fracture, durability and sand equivalent requirements outlined in Section 00641 of the Oregon Standard Specifications for Construction
- Maximum passing the No. 200 sieve ≤ 5%
- Compacted to 98% of the maximum dry density as determined by ASTM D698 or AASHTO T-99

Aggregate Subbase Rock (Acceptable for Structural Fill)

- Angular Clean Crushed (jaw run) hard "Shale" (4" Minus Jaw-Run) or Crushed Rock (2" to 4" Minus); R=50 or greater; Angular and Reasonably Well Graded
- At Least 60% retained on the No. 4 Sieve.
- Exceeds the fracture, durability and sand equivalent requirements outlined in Section 00641 of the Oregon Standard Specifications for Construction
- Maximum passing the No. 200 sieve ≤ 10% Total; ≤ 3% Clay Size
- <u>During wet weather; passing No. 200 sieve ≤ 5%.</u>
- Compacted to 98% of the maximum dry density as determined by ASTM D698 or AASHTO T-99; initial lift may not attain 95% due to soft subgrade; Engineer to decide in the field.
- Care must be taken to avoid very silty subbase that will not support construction loads, especially when wet (will not meet specifications).

Decomposed Granite

- Sandy DG with little to no clay.

Embankment Fill (Acceptable for Structural Fill During <u>Dry</u> Weather)

- Reasonably well graded (not open work)
- Has at least 60% retained on the No. 4 sieve.
- Has no more than 30% passing No. 200 sieve.
- Passing No. 200 sieve must have less than 20% clay size.

On-Site Soil Fill

- None available.

Clean Sand

- Clean washed sand or sand and gravel, less than 1% passing No. 200.
- Gravel to be rounded or subrounded (no fracture faces), 1" or less.
- Must have less than 30% gravel by weight.

Note: Some fill materials will be difficult to nearly impossible to compact during wet weather. *The contractor must select the type of structural fill that will be able to be placed and compacted to specified conditions during the weather conditions that can take place during the construction schedule.*

Drain Rock (For drainage sections)

- Clean washed rounded or angular openwork drain rock.
- Gradation to be 1/4" and greater, sized to not move into and through perforations in the pipe.
- 1/4" to 3/4" clean crushed, 3/4" to 1" clean rounded rock, 1" to 2" clean angular rock are all acceptable.
- Clean means washed rock with NO coating of silt, clay or sand.

Note: All types may be used in all applications of drain rock that are not beneath Asphaltic Concrete paved areas. In all AC areas angular clean drain rock must be used for AC support.

Note: Drainage layer drain rock that is beneath the floor slab must be the angular clean drain rock.

Geotextile Filter Fabric

- Non-woven geotextile filter fabric for wrapping drainage sections and separation of openwork rock from sands or soils fines.
- Meet specifications as per Mirafi 140N or equivalent.
- Overlap all edges at least 24 inches (12" for drainage section envelope).
- Secure in place such that overlaps will not move during covering operation.

Geotextile Support Fabric

- Woven geotextile support fabric designed for separation of crushed rock and subgrade soil and for rock section support.
- Meet specifications as per ACF180 woven support fabric.
- Overlap edges at least 2 feet and ends at least 5 feet.
- Align roll lengthwise with direction of traffic in all drive lanes.
- Pull tight full length and keep tight during placement of crushed rock above fabric.
- Do not drive on the fabric until it is covered with rock.

Perforated Pipe

- 3", 4" or 6" rigid wall, smooth interior perforated pipe.
- Secure all joints with solvent weld glue. DO NOT use only compression push together fittings.
- Slope to drain per specifications in report or on plan sheets.
- Align perforations in the downward direction.
- Must always be placed within filter fabric wrap unless specifically specified otherwise.
- Protect from construction traffic until buried at least 2 times pipe diameter (minimum 8 inches) of angular rock fill.

Wall Sheet Drain

- Polymer sheet drain with filter fabric attached 1 or 2 sides, designed for drainage of vertical embedded foundation walls.
- For walls up to 10 feet tall. Must meet specifications as for American Wick Drain's AMERDRAIN 200 or 220.
- Install and splice and patch per manufacturer's recommendations.
- Install with fabric side towards the backfill.
- Attach to wall per manufacturer's recommendations.
- Extend down wall all the way to bottom of drainage section around perforated pipe.
- Protect from damage when backfilling with crushed rock larger than 2-inch minus.
- Repair all damaged areas prior to final backfill.

These materials shall be used on this project as specified in this report and on project plans or specifications.

NOTE: DEVIATIONS FROM SPECIFIED MATERIALS MUST BE APPROVED IN WRITING BY THE GEOTECHNICAL ENGINEER, OWNER AND OWNER'S OTHER CONSULTANTS/DESIGN ENGINEERS PRIOR TO USE AT THE SITE.

9.0 ADDITIONAL SERVICES AND LIMITATIONS

9.1 ADDITIONAL SERVICES

We should review construction plans and specifications for this project as they are being developed. In addition, The Galli Group 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. Additionally, to observe compliance with the intent of our recommendations, design concepts, and the plans and specifications, all construction operations dealing with earthwork, foundations and rock placement and compaction should be observed by a representative from The Galli Group.

For this project, we anticipate additional services could include the following:

- Review of final construction plans and specifications for compliance with geotechnical recommendations.
- Possible project team meetings to clarify issues and proceed smoothly into and through the construction process.
- Observation of on-site excavations to verify stability is acceptable.
- Observation and/or testing of overexcavation, subgrade preparation, structural fill placement and compaction, subdrains and site drainage.
- Pile installation and test loading.

- Periodic construction field reports and final report, as requested by the client and required by the building department.

We would provide these additional services on a time-and-expense basis in accordance with our current Standard Fee Schedule and General Conditions at the time of construction. If we are not retained to provide these services, we cannot be held responsible for the decisions by others or for geotechnical related issues in the constructed product that we have not verified.

9.2 LIMITATIONS

The analyses, conclusions and recommendations contained in this report are based on site conditions and assumed development plans as they existed at the time of the study, and assume soils and groundwater conditions exposed and observed in the borings during our investigation are representative of soils and groundwater conditions throughout the site. If during construction, subsurface conditions or assumed design information is found to be different, we should be advised at once so that we can review this report and reconsider our recommendations in light of the changed conditions. If there is a significant lapse of time (5 years) between submission of this report and the start of work at the site, if the project is changed, or if conditions have changed due to acts of God or construction at or adjacent to the site, it is recommended that this report be reviewed in light of the changed conditions and/or time lapse.

This report was prepared for the use of the School District and their design and construction team for the design and construction of the project. It should be made available to contractors for information and factual data only. This report should not be used for contractual purposes as a warranty of site subsurface conditions. It should also not be used at other sites or for projects other than the one intended.

We have performed these services in accordance with generally accepted geotechnical engineering practices in southern Oregon, at the time the study was accomplished. No other warranties, either expressed or implied, are provided.

THE GALLI GROUP
GEOTECHNICAL CONSULTING

Dennis Duru, M.Sc., E.I.T.
Staff Associate

William F. Galli, P.E., G.E.
Senior Principal Engineer

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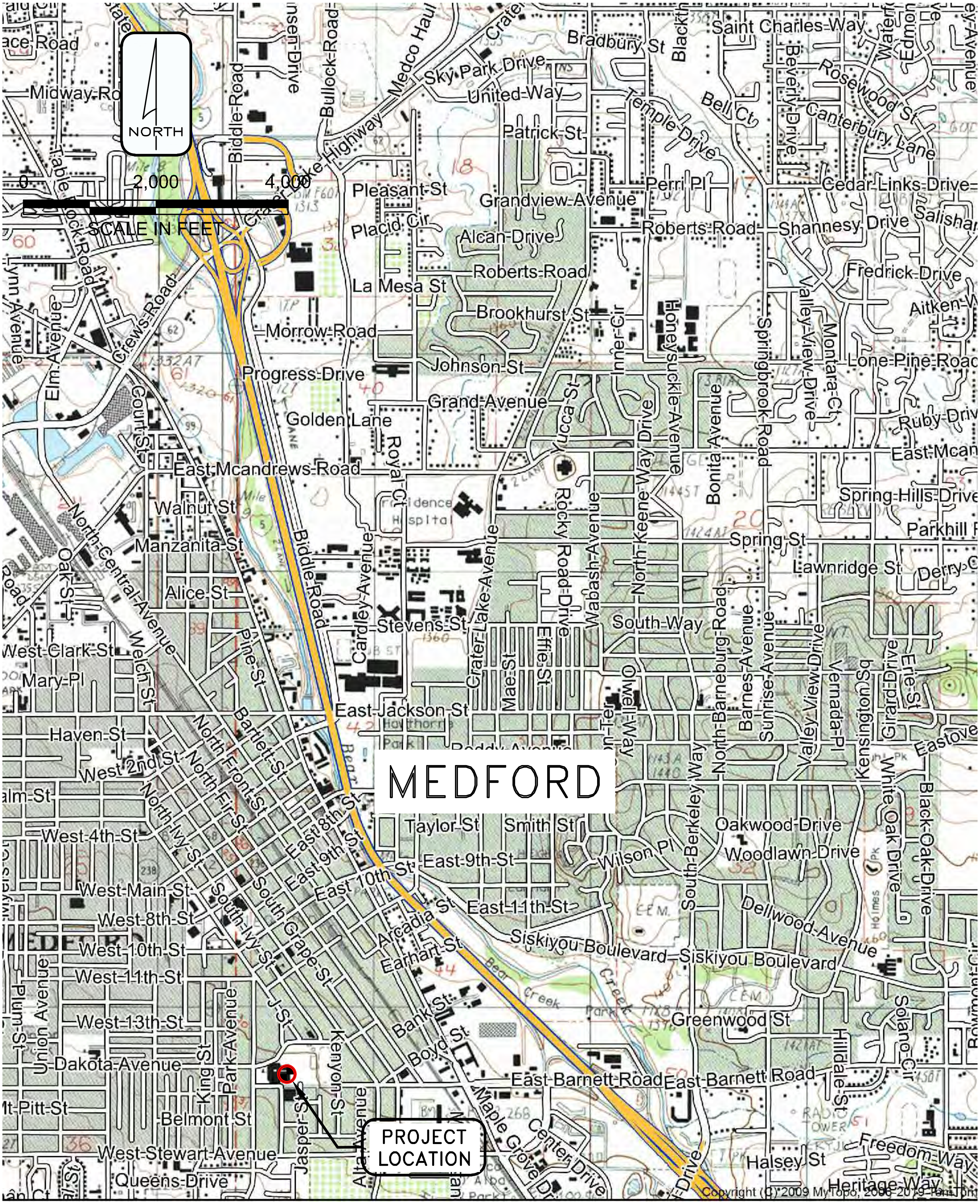
OSSC. (2019). *Oregon Structural Specialty Code.*

USGS. (2014). *National Seismic Hazard Mapping*. Retrieved from https://earthquake.usgs.gov/hazards/interactive/



THE GALLI GROUP GEOTECHNICAL CONSULTING 612 NW 3rd Street Grants Pass, OR 97526	VICINITY MAP MSDEC GYMNASIUM MEDFORD, OREGON	DATE: NOVEMBER 2020 JOB NO: 02-5760-02 REV: 11/25/2020 11:10 AM PREPARED BY: JDT/BD 5760-2 Medford Central Hi Sch -1-Vicinity.dwg	FIGURE: 1

LEGEND

B-1 BORING NUMBER AND APPROXIMATE LOCATION

APPROX NORTH

0 100 200

SCALE IN FEET

SITE PLAN PROVIDED BY GOOGLE EARTH

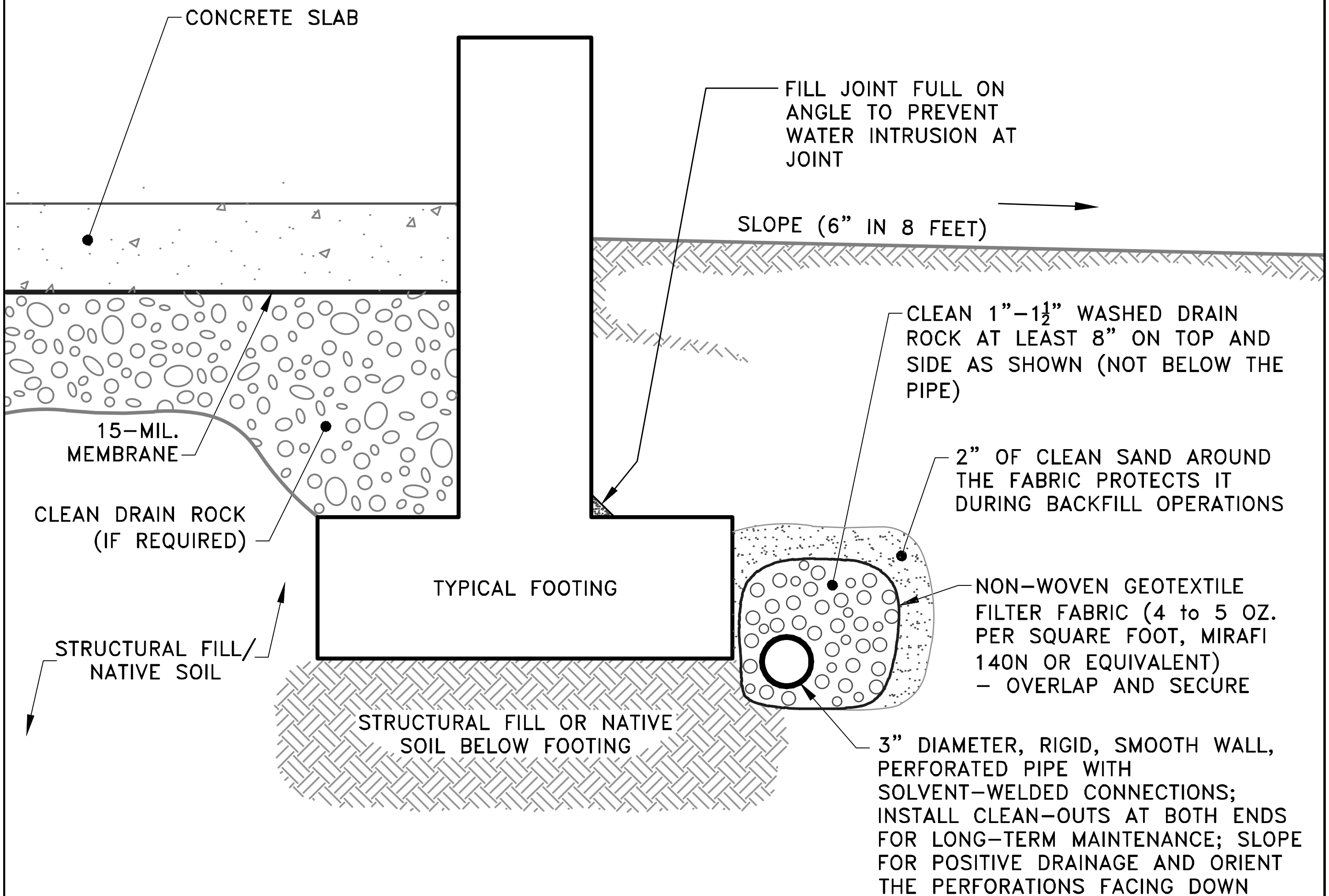
THE GALLI GROUP
GEOTECHNICAL CONSULTING
612 NW 3rd Street
Grants Pass, OR 97526

SITE PLAN WITH BORING LOCATIONS

MSDEC GYMNASIUM
MEDFORD, OREGON

DATE:	NOVEMBER 2020
JOB NO:	02-5760-02
REV:	11/25/2020 3:00 PM
PREPARED BY:	JDT/BD
5760-2 Medford Central Hi Sch -2- Site Plan.dwg	

FIGURE: 2



NOTES:

(1) VAPOR BARRIER TO BE STEGO INDUSTRIES 15mil STEGO WRAP OR EQUIVALENT. OWNER MAY CHOOSE TO USE 6mil VISQUENE, UNDERSTANDING IT WILL NOT WORK AS WELL.

(2) CAPILLARY BREAK ROCK BELOW VAPOR BARRIER TO BE 1/4" TO 3/4" CLEAN CRUSHED ROCK OR EQUIVALENT.

FOR ILLUSTRATION PURPOSES ONLY – NOT FOR CONSTRUCTION
NOT TO SCALE

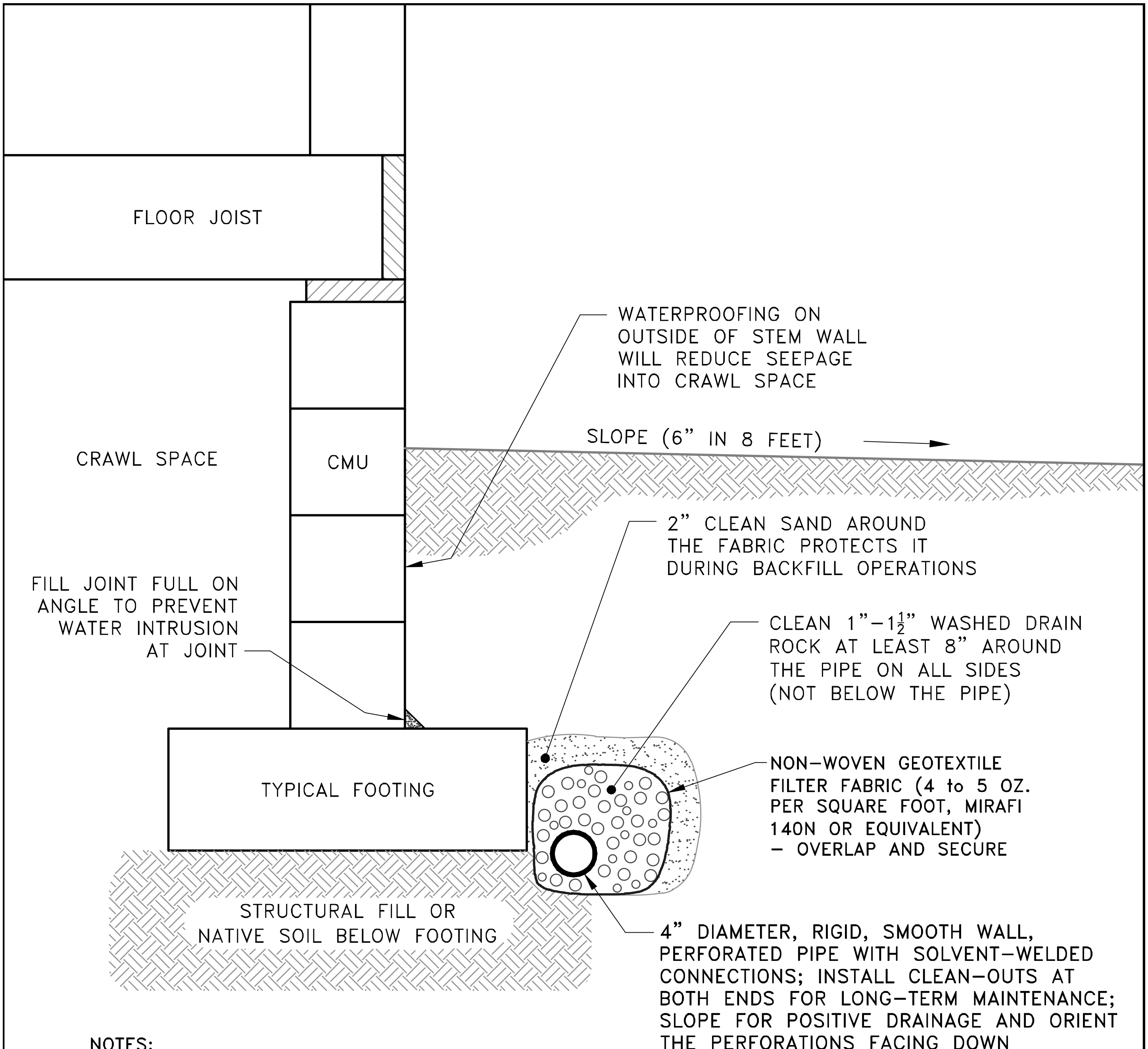
THE GALLI GROUP
GEOTECHNICAL CONSULTING
612 NW 3rd Street
Grants Pass, OR 97526

TYPICAL FOUNDATION DRAIN
SLAB ON GRADE FLOOR

MSDEC GYMNASIUM
MEDFORD, OREGON

DATE: NOVEMBER 2020
JOB NO: 02-5760-02
REV: 11/25/2020 11:05 AM
PREPARED BY: JDT/BD
5760-2 Medford Central Hi Sch -3- found drain-slab.dwg

FIGURE: 3



NOTES:

(1) VAPOR BARRIER TO BE STEGO INDUSTRIES 15mil STEGO WRAP OR EQUIVALENT. OWNER MY CHOOSE TO USE 6mil VISQUENE, UNDERSTANDING IT WILL NOT WORK AS WELL.

(2) CAPILLARY BREAK ROCK BELOW VAPOR BARRIER TO BE 1/4" TO 3/4" CLEAN CRUSHED ROCK OR EQUIVALENT.

FOR ILLUSTRATION PURPOSES ONLY – NOT FOR CONSTRUCTION
NOT TO SCALE

THE GALLI GROUP GEOTECHNICAL CONSULTING 612 NW 3rd Street Grants Pass, OR 97526	TYPICAL FOUNDATION DRAIN WITH CRAWL SPACE MSDEC GYMNASIUM MEDFORD, OREGON	DATE: NOVEMBER 2020 JOB NO: 02-5760-02 REV: 11/25/2020 11:10 AM PREPARED BY: BD 5760-2 Medford Central Hi Sch -4-crawl.dwg	FIGURE: 4

APPENDIX A

BORING LOGS

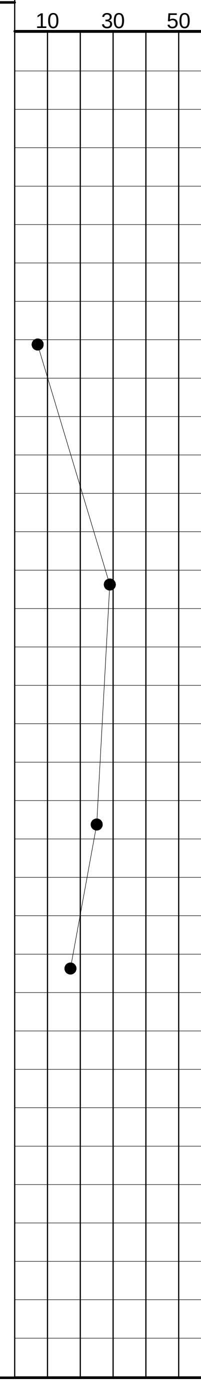
THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

BORING LOG
B-1

Project: CENTRAL MEDFORD HIGH SCHOOL
Project No.: 02-5760-01
Client: MEDFORD SCHOOL DISTRICT
Date: 11/08/2019
Location:
Elevation:
Driller: BLAKE, KEN
Logged By: DENNIS DURU
Drill Rig: ATV MOUNTED, 4" DIA. SSA
Depth To Water> Initial :   At Completion :

USCS	Description	Depth (ft)	Sample No. and Type	NMC	SPT N
OL	Grass Rootzone.	0.25			
OH	Soft, dark brown, Clay; moist.	1.0			
GC	Medium dense, dark brown, clayey Gravel; moist.	2.0			
SC	Medium dense, dark gray, silty Sand; moist.	2.5			
GP-GC	Medium dense, brown, gravelly, silty, Clay; moist.	5.5	S-1		7
GM/SM	Medium dense, brown, silty Sand and Gravel; moist.	8.8	S-2		29
			S-3		25
CL/CH	Very stiff, brown, mottled streak, Clay; moist.	10.5	S-4		17
	Bottom of Boring at 10.5 Feet. No Free Groundwater Encountered.				

Standard Penetration Test CURVE



Legend of Samplers: Grab sample   SPT sample   Shelby tube sample

This information pertains only to this boring and should not be interpreted as being indicative of the site.

THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

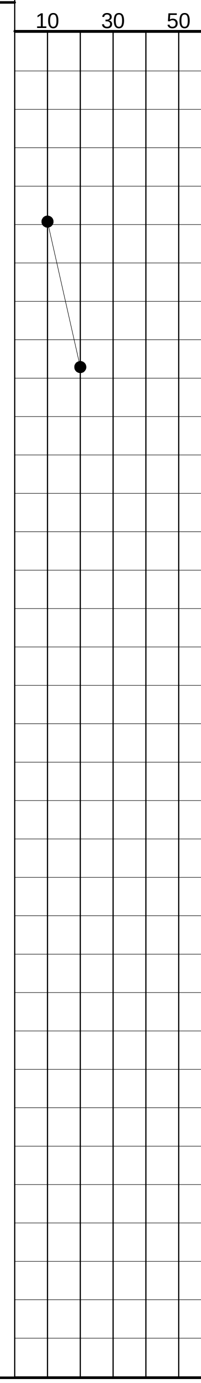
BORING LOG
B-2

Project: MSDEC Gymnasium
Client: Medford School District
Location: See figure 2, Site Plan with Boring Locations
Driller: TGG (Aaron and Ken)
Drill Rig: ATV Mounted 4" SSA
Depth To Water> Initial ▽ :

Project No.: 5760-02
Date: 11/17/20
Elevation:
Logged By: Dennis Duru
At Completion ▼ : None

Graphic Log	USCS	Description	Depth	Sample No. and Type	NMC	Standard Penetration Test N
	SM	Loose to medium dense, light brown, silty Sand with some gravels; moist.	0			
			3.3	S-1	10%	10
				S-2	7%	20
	GP/GW	(6.5) Very dense, brown, Gravels and Crushed Rock; moist.	6.6			
		(7.5) Bottom of boring at 7.5 feet; augar refusal. No freewater encountered.				
			9.9			
			13.2			
			16.5			
			19.8			
			23.1			

Standard Penetration Test CURVE



Legend of Samplers: Grab sample | SPT sample | Shelby tube sample

This information pertains only to this boring and should not be interpreted as being indicative of the site.

THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

BORING LOG
B-3

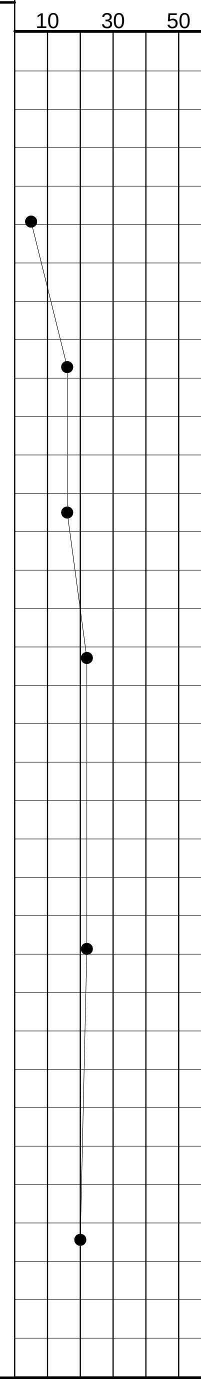
Project: MSDEC Gymnasium
Client: Medford School District
Location: See figure 2, Site Plan with Boring Locations
Driller: TGG (Aaron and Ken)
Drill Rig: ATV Mounted 4" SSA
Depth To Water> Initial : 4.5'

Project No.: 5760-02
Date: 11/17/20
Elevation:
Logged By: Dennis Duru
At Completion : 4.5'

Layer Depth (ft)	USCS	Description	Sample No. and Type	NMC	N
0 – 0.25	OL/OH	Rootzone			
0.25 – 2.0	GC	Loose, brown, clayey Gravel; crushed rock fill, moist.			
2.0 – 5.0	MH/CH	Soft to medium stiff, light brown, clayey Silt; moist.	S-1 (SPT)	21%	5
5.0 – 7.5	SC/GC	Medium dense, brown, clayey Sand and Gravel; moist to saturated.	S-2 (SPT)	15%	16
7.5 – 16.0	CH/MH	Very Stiff, light brown, Silt and Clay; with some gravels, saturated.	S-3 (SPT)	29%	16
			S-4 (SPT)	31%	22
16.0 – 20.0	GC	Medium dense, brown, clayey Gravel; saturated.	S-5 (SPT)	20%	22
20.0 – 21.5	CH/MH	Very stiff, light brown, Silt and Clay; wet.	S-6 (SPT)	29%	20
		Bottom of boring at 21.5 feet. Free water encountered at 4.5 feet.			

Depth scale: 0, 3.3, 6.6, 9.9, 13.2, 16.5, 19.8, 23.1

Standard Penetration Test CURVE



Legend of Samplers: Grab sample | SPT sample | Shelby tube sample

This information pertains only to this boring and should not be interpreted as being indicative of the site.

THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

BORING LOG
B-4

Project: MSDEC Gymnasium
Client: Medford School District
Location: See figure 2, Site Plan with Boring Locations
Driller: TGG (Aaron and Ken)
Drill Rig: ATV Mounted 4" SSA
Depth To Water> Initial : 8.0' At Completion : 8.0'

Project No.: 5760-02
Date: 11/17/20
Elevation:
Logged By: Dennis Duru

USCS	Description	Layer Depth	Depth	Sample No. and Type	NMC	N
OL/OH	Rootzone	0.25	0			
GM	Loose to medium dense, brown, silty Gravel; moist.	5.0	3.3	S-1	7%	15
SC/GC	Medium dense, brown, clayey Sand and Gravel; moist to saturated.	7.5	6.6	S-2	8%	20
CH/MH	Very Stiff, light brown, Silt and Clay; with some gravels, saturated.	10.0		S-3	18%	29
CH/MH	Hard, brown, gravelly Silt and Clay; saturated.	11.5	9.9	S-4	12%	35
	Bottom of boring at 11.5 feet. Free water encountered at 8.0 feet.		13.2			
			16.5			
			19.8			
			23.1			

Standard Penetration Test CURVE: 10, 30, 50

Legend of Samplers: Grab sample; SPT sample; Shelby tube sample

This information pertains only to this boring and should not be interpreted as being indicative of the site.

APPENDIX B

LABORATORY TEST RESULTS

Expansion Index Worksheet (ASTM D-4829)

THE GALLI GROUP
Geotechnical Consulting

Client:	Medford School District
Project	MSDEC Gymnasium
Job No:	02-5760-02
Test Date:	11/24/2020
Sample Location:	B-1,S-1 2.5' to 4.0'
Sample Date:	11/17/2020
Description of Soil:	Medium brown, clayey Sand and Gravel

Expansion Index measured (EIm):

$EI_m = \Delta H / H_{orig} * 1000$

begin dial :	0.0584
end dial:	0.0803
EI_m:	**22**

Weight of ring (g):	365.3
Wt. Wet sample in ring(g):	774.5
Sample Wet Weight (g):	409.28
Sample Length (in.):	1
Sample Diameter (in.):	4.01
Volume of sample (ft^3):	0.007309
Sample Unit Wt. (PCF):	**123.3**
Sample Dry Unit Wt. (PCF):	**114.1**

Saturation (S):

$S=(SG)(w)\gamma d)/(SG)*62.4)-\gamma d$

SG:	2.7
γd:	114.1
%*w* :	8.1
S=	**46**

As prepared for testing:

can no.	D-4
wet weight of soil + can (g)	271.21
dry weight of soil + can (g)	260.44
weight of can (g)	127.51
weight of dry soil (g)	132.93
weight of water (g)	10.77
moisture content (% of dry weight)	8.1

EI_{50} Calculation:

$EI_{50=Elm - (50-Sm)*[(65+Elm)/(220-Sm)]}$

EI_M	22
S	46
EI_{50} =	**20**

After testing:

can no.	556
wet weight of soil + can (g)	620.49
dry weight of soil + can (g)	558.60
weight of can (g)	179.55
weight of dry soil (g)	379.05
weight of water (g)	61.89
moisture content (% of dry weight)	16.3

#4 + (dry wt.)	133.79
#4 - (dry wt.)	519.10
% Passing #4 Sieve =	**79.5**

Tested By: Lyn Chand

Expansion Index Worksheet (ASTM D-4829)

THE GALLI GROUP
Geotechnical Consulting

Client:	Medford School District
Project	MSD Education Center
Job No:	02-5764-01
Test Date:	11/12/2019
Sample Location:	B-1/S-1 @ 0.25' to 1.0'
Sample Date:	11/8/2019
Description of Soil:	Brown, silty Clay

Expansion Index measured (EIm):

$EI_m = \Delta H / H_{orig} * 1000$

begin dial :	0.0430
end dial:	0.1040
EI_m:	**61**

Weight of ring (g):	365.2
Wt. Wet sample in ring(g):	755.2
Sample Wet Weight (g):	389.95
Sample Length (in.):	1
Sample Diameter (in.):	4.01
Volume of sample (ft^3):	0.007309
Sample Unit Wt. (PCF):	**117.5**
Sample Dry Unit Wt. (PCF):	**103.8**

Saturation (S):

$S=(SG)(w)\gamma d)/(SG)*62.4)-\gamma d$

SG:	2.7
γd:	103.8
%*w* :	13.2
S=	**57**

As prepared for testing:

can no.	26
wet weight of soil + can (g)	520.74
dry weight of soil + can (g)	468.45
weight of can (g)	71.91
weight of dry soil (g)	396.54
weight of water (g)	52.29
moisture content (% of dry weight)	13.2

EI_{50} Calculation:

$EI_{50} = EIm - (50-Sm)*[(65+EIm)/(220-Sm)]$

EI_M	61
S	57
EI_{50} =	**67**

After testing:

can no.	AD-2
wet weight of soil + can (g)	622.90
dry weight of soil + can (g)	519.76
weight of can (g)	179.70
weight of dry soil (g)	340.06
weight of water (g)	103.14
moisture content (% of dry weight)	30.3

#4 + (dry wt.)	47.6
#4 - (dry wt.)	1568.7
% Passing #4 Sieve =	**97.1**

Tested By: Lyn Chand

THE GALLI GROUP
Geotechnical Consulting

PRELIMINARY SEISMIC RETROFIT STUDY
NORTH MEDFORD HIGH SCHOOL
MEDFORD, OREGON

For: Ron Havniear
Medford School District
815 South Oakdale Ave
Medford, OR 97501

c/o Stephen L. Chase, Project Manager
ZCS Engineering & Architecture
127 NW D Street
Grants Pass, Oregon 97526

By: **THE GALLI GROUP**
612 NW Third Street
Grants Pass, OR 97526
(541) 955-1611

02-6020-01
June 30, 2021

Table of Contents

PRELIMINARY SEISMIC RETROFIT STUDY
NORTH MEDFORD HIGH SCHOOL
MEDFORD, OREGON

1.0 INTRODUCTION

This report presents results of our geotechnical and geological evaluation of the North Medford High School for a potential seismic retrofit of the school structures. The subject school site is located at 1900 N. Keene Way Drive, on the east side of the intersection of N. Keene Way Drive and Brookhurst Street, in Medford Oregon. Please see Figure 1, Vicinity Map, for a more precise location.

The purpose of our investigation and this report was to accomplish a limited site surface and subsurface evaluation (one boring) and conduct a planning level seismic risk assessment (office studies) in order to provide preliminary geotechnical and geologic information and evaluate the likelihood and consequences of geotechnical/geologic related seismic failures, including liquefaction and landslide potential during the design seismic event, for consideration of the potential seismic retrofit.

2.0 SITE AND PROJECT DESCRIPTION

The site is currently occupied by a functioning high school, which consists of 12 structures. The majority of the structures are connected via covered walkways. Some of the structures are surrounded by play fields, access roads, parking, walkways and open space. The site is relatively flat to mildly sloping with undeveloped portions of the site consisting of landscape areas, well-maintained lawn/fields and a few trees.

We understand the district is conducting a preliminary review to determine the level of seismic retrofit needed for the structures on this campus. Their review will largely be based on the evaluation of the potential geologic hazards (such as liquefaction) provided in this report, and an evaluation of the potential structural damage to these facilities associated with the design seismic event. The findings will also likely be used to determine if grant funding will be pursued to complete the seismic retrofit work.

3.0 FIELD EXPLORATION

On June 18, 2021, Associate Staff, Dennis Duru, M.Sc., E.I.T and our drilling crew, visited the site to accomplish the subsurface investigation. One (1) exploratory boring was drilled in the grass field east of school building "H". A utility locate was completed

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prior to our investigation and our representative coordinated with school personnel to identify the field exploration location away from the marked and known utility locations. See Figure 2, Site Plan, for a layout of the site and the location of the boring. The boring was drilled with our ATV-mounted, solid-stem auger drill rig and penetrated to a depth of 3.0 feet before encountering the weathered sandstone bedrock. Upon completion, the boring was backfilled with drill spoils.

Standard Penetration Testing (SPT) was accomplished in each boring. This entails driving a 1½-inch diameter steel split spoon sampler by dropping a 140-pound weight for a 30-inch drop. The total number of blows it takes to drive the sampler the last 12 inches of an 18-inch drive is called the SPT N-value. These can be correlated with density and soil strength parameters from testing on thousands of other projects.

Our representative identified the final exploration location, logged subsurface soils and water conditions and obtained soil samples for transport to our laboratory. Visual classifications of the soils were made in the field and are presented in the Boring Logs in Appendix A, at the end of this report.

4.0 LABORATORY TESTING

Moisture content tests were accomplished on soil samples obtained by Standard Penetration Testing. No other tests were accomplished.

5.0 SUBSURFACE CONDITIONS

5.1 SOIL

The subsurface consisted of stiff, dark brown Clay to a depth of 1.5 feet. This was then underlain by very dense, brown Sand (weathered Sandstone of the Hornbrook Formation).

Please see more specific soils information in the Boring Logs in Appendix A. Please note that the soils are shown as distinct layers in the Boring Logs while in nature they may change more gradually. Soils conditions may also change somewhat between the locations investigated.

5.2 GROUNDWATER

Generally, the soils encountered were damp to moist. No groundwater was encountered in the one boring conducted. Review of nearby well log information shows that static groundwater levels are over 60 feet below the ground surface.

6.0 GEOLOGIC OR SEISMIC INDUCED HAZARDS

Summary of Site Geology and Seismicity. The project area is located in the Medford East 30x60 minute USGS topographic quadrangle (see Vicinity Map, Figure 1). Mapped geologic units at the project area consist primarily of Alluvial Fan deposits and bedrock members of the Hornbrook Formation (Wiley et al, 2011). The Marine Sandstone, Siltstone and Conglomerate members are the mapped bedrock unit at the project (Wiley et al, 2011; OGDC-6, 2015). The top of the weathered Sandstone was encountered in the augered boring at this site at a shallow depth of 3 feet below the ground surface.

The project site is in proximity to several zones of active seismicity. The region is affected by the Cascadia Subduction Zone (CSZ), an active subduction zone off the Oregon coast considered capable of Magnitude 8.5 or greater earthquakes. Average recurrence intervals for such great earthquakes, as determined by recent investigations, range between 300-600 years. The last "great" earthquake was interpreted to be approximately 300 years ago. The CSZ is the main seismic event for consideration for this seismic retrofit.

The project area has an additional tectonic source from earthquakes occurring along active Basin and Range faults as close as 50 kilometers to the southeast. This region has produced numerous earthquakes, including significant events near Klamath Falls and Warner Valley. Such events occur generally once every one to two decades.

Flood Hazard. The site is not near streams or rivers. Therefore, it is not within a 100-year floodplain.

Landslides/Slope Instability. There are no slopes close to the site. Therefore, possibility of slope failure, rock fall or slide run out damage at the site is low.

Liquefaction and Lateral Spread. The project is underlain by stiff Clay over very dense Sand in the shallow subsurface. The very dense Sand encountered in our boring is the top of the weathered Sandstone bedrock of the Hornbrook Formation underlying the project site. Very dense Sand soils/weathered Sandstone will not liquefy in a seismic event. Based on our review of available data (well logs and NRCS web soil survey) as well as from the results of our subsurface exploration, it appears the depth to bedrock across the site is at approximately 4 feet or less. <u>Therefore, liquefaction and lateral spread is not considered to be a potential hazard for this site</u>. See more in Section 7.0 of this report.

Expansive Soils. The upper soils within the subsurface consist of dark brown Clay. The clays will likely have low to moderate expansion potential based on our experience with soils with similar visual properties.

These expansive soils can have adverse impacts on foundations and all manner of concrete flatwork if the building design does not account for such soils. We recommend that each proposed building site (with clayey soils present) have the soil subgrade examined and tested (Expansion Index) prior to final design and construction. In that way, locations that have expansive soils will have the retrofit structure(s) and drainage designed accordingly.

Construction over expansive soils generally requires embedment of footings to 3 to 4 feet (final depth to be verified after EI testing) below the exterior grade and placement of floor slabs over at least 24 inches or more of compacted rock fill in order to mitigate expansion potential of the underlying soil subgrades. Maintaining the moisture content in the soil to keep it in a moist and fully swelled condition prior to being covered is also critical to proper performance of the structures.

Note: The geotechnical engineer must provide site specific laboratory testing and remedial design recommendations on projects that have potentially expansive, gravelly Clay, clayey Silt or Clay soils present.

Ground Rupture. No Quaternary faults were identified at the project site based on our review of USGS fault (US Quaternary Faults) maps, and from the single limited exploratory boring. Therefore, the risk of damage at the site due to ground rupture is considered very low.

Ground Shaking. Project structures including foundations and retaining walls must be designed for the potential for very strong ground shaking during the anticipated seismic event. The peak site modified horizontal acceleration (PGA_M), is 0.35g. This is based on the Site Class C designation, determined for the project from subsurface drilling and evaluation of SPT data. The PGA_M value can be used with an appropriate seismic coefficient in pseudo static analysis for design of the seismic upgrades.

Seismic Ground Amplification or Resonance. No unusually hazardous amplification or resonance effects on seismic waves have been associated with soil/bedrock subsurface conditions in the project area.

Tsunami and Seiche. The site is approximately 80 miles inland from the coast, and not subject to tsunami hazard. The site is not located adjacent to a large lake or body of water, and therefore, not subject to seiche hazard.

7.0 LIQUEFACTION EVALUATION

The liquefaction phenomenon occurs in saturated, loose (low density, uncompacted or poorly compacted), cohesionless soils. When loose, cohesionless soils are saturated which is the case when the soil is below the water table, then water fills the soil pores. In response to compression when a load is applied to the soil, the water increases in pressure and attempts to migrate towards zones of low pressure which is usually upwards towards the ground surface. However, if the applied load is rapid and large enough, or if it is repeated many times (cyclic loading) like in an earthquake, such that there is not enough time for the water to dissipate before the next cycle of loading is applied, then the water pressure may build up to a degree where they become greater than the grain-to-grain contact stresses of the soil. The grain-to-grain contact stresses are the source of the shear strength that supports structure foundations and overburden soils in these soil structures. This buildup of excess pore water pressure results in total or partial loss of the soil strength, and the soil may be observed to flow like a liquid, hence "liquefaction". At this point, the soil will lose all its shear strength, be deformed, and will not be able to support structures.

<u>The conditions for liquefaction to occur was not observed at this site during our limited subsurface investigation</u>. The site is underlain by very dense Sand (weathered Sandstone). No groundwater was observed in the boring to the depth drilled and could be below 60 feet deep. Therefore, liquefaction cannot take place.

<u>Therefore, in our professional opinion, the site conditions found in the boring will not result in wide spread liquefaction that will have significant adverse impacts on the structures during a seismic event.</u>

8.0 CONCLUSIONS

In our professional opinion, based on our field investigation, office review and previous work in the area, the soils conditions at the site are suitable for a conventional seismic retrofit. Crushed rock structural fill over the very dense Sand will provide adequate support of new foundations, grade beams and/or buttresses. In our opinion, this school site is not subject to large scale liquefaction that will cause a significant adverse impact to the school structures.

Additional borings around the structures on this site could possibly find zones of soils that may liquefy. However, these are likely to be moderate to small in size and should not adversely impact the structure.

If a full seismic retrofit geotechnical design report is needed, additional tasks to be accomplished would be as follows:

1. 2 or 3 additional borings.
2. Laboratory testing for determining strength and settlement characteristics of the site soils.
3. Evaluation of data for developing design parameters.

These could be used to provide a full scale Seismic Retrofit Design Report.

8.1 LIMITATIONS

The analyses, conclusions and recommendations contained in this report are based on site conditions as they existed at the time of the study, and assume soils, rock and groundwater conditions exposed and observed in the boring during our investigation are representative of soils and groundwater conditions throughout the site. If during construction, subsurface conditions or assumed design information is found to be different, we should be advised at once so that we can review this report and reconsider our recommendations in light of the changed conditions. If there is a significant lapse of time (5 years) between submission of this report and the start of work at the site, if the project is changed, or if conditions have changed due to acts of God or construction at or adjacent to the site, it is recommended that this report be reviewed in light of the changed conditions and/or time lapse.

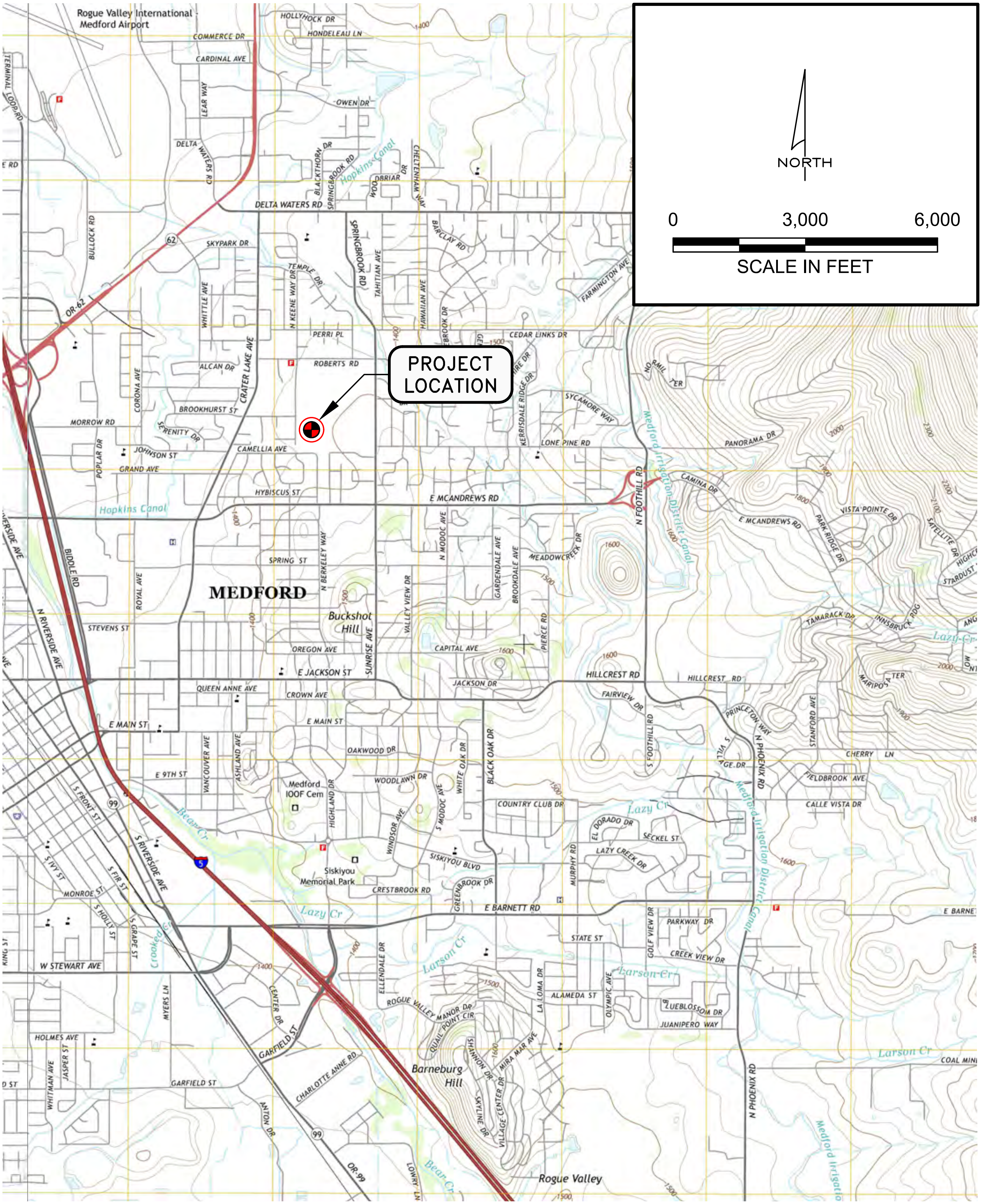
This report was prepared for the use of the School District and their design team for evaluating the need for a full scale Seismic Retrofit evaluation and report. It should be made available to contractors for information and factual data only. This report should not be used for contractual purposes as a warranty of site subsurface conditions. It should also not be used at other sites or for projects other than the one intended.

We have performed these services in accordance with generally accepted geotechnical engineering and professional geology practices in southern Oregon, at the time the study was accomplished. No other warranties, either expressed or implied, are provided.

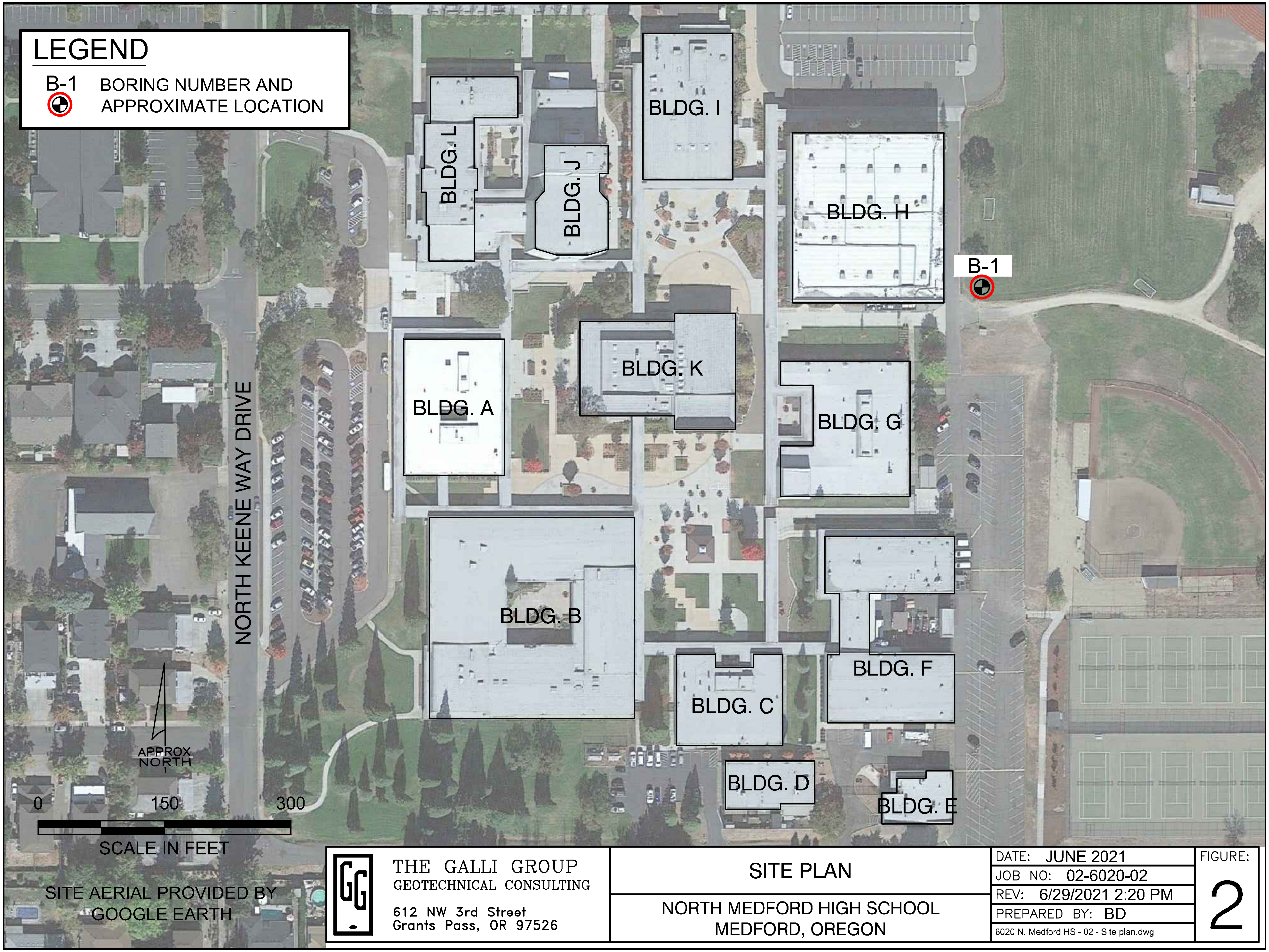
THE GALLI GROUP
GEOTECHNICAL CONSULTING

Dennis Duru, M.S., E.I.T.
Staff Associate

Melvin Galli III, P.E.
Principal Engineer



THE GALLI GROUP GEOTECHNICAL CONSULTING 612 NW 3rd Street Grants Pass, OR 97526	VICINITY MAP NORTH MEDFORD HIGH SCHOOL MEDFORD, OREGON	DATE: JUNE 2021 JOB NO: 02-6020-02 REV: 6/29/2021 2:04 PM PREPARED BY: BD 6020 N. Medford HS - 01 - Vicinity.dwg	FIGURE: 1



THE GALLI GROUP
GEOTECHNICAL CONSULTING
612 NW 3rd Street
Grants Pass, OR 97526

SITE PLAN

NORTH MEDFORD HIGH SCHOOL
MEDFORD, OREGON

DATE: JUNE 2021
JOB NO: 02-6020-02
REV: 6/29/2021 2:20 PM
PREPARED BY: BD
6020 N. Medford HS - 02 - Site plan.dwg

FIGURE: 2

APPENDIX A

BORING LOGS

THE GALLI GROUP
GEOTECHNICAL CONSULTANTS

BORING LOG
B-1

Project: North Medford High School
Client: Medford Middle School
Location: Grass field on the east side of Building H (see Site Plan)
Driller: TGG (Ken, Nick)
Drill Rig: ATV Mounted Rig, 4" diameter SSA
Depth To Water> Initial :

Project No.: 02-6020-01
Date: 6/18/2021
Elevation:
Logged By: Dennis Duru
At Completion :

Graphic Log	USCS	Description	Depth	Sample No. and Type	NMC	Standard Penetration Test N	CURVE (10 30 50)
	OL	Grass rootzone. (0.25)	0				
	OH	Stiff, dark brown, Clay; moist (1.5)					
	SW	Very dense, brown, Sand; damp. (Top of weathered Sandstone) (3.0)		S-1 (SPT)		53	●
		Bottom of boring at 3.0 No free groundwater encountered	5				
			10				
			15				
			20				
			25				
			30				
			35				

Legend of Samplers: Grab sample SPT sample Shelby tube sample

This information pertains only to this boring and should not be interpreted as being indicative of the site.