Development Services

From Concept to Construction

Phone: 503-823-7300 Email: bds@portlandoregon.gov 1900 SW 4th Ave, Portland, OR 97201
More Contact Info (http://www.portlandoregon.gov//bds/article/519984)

APPEAL SUMMARY

Status: Decision Rendered

Appeal ID: 14920	**Project Address:** 4732 N Albina Ave
Hearing Date: 4/12/17	**Appellant Name:** Martha Williams
Case No.: P-002	**Appellant Phone:** 503-946-6694
Appeal Type: Plumbing	**Plans Examiner/Inspector:** to be determined
Project Type: commercial	**Stories:** 4 **Occupancy:** not provided **Construction Type:** not provided
Building/Business Name:	**Fire Sprinklers:** Yes - not provided
Appeal Involves: Erection of a new structure	**LUR or Permit Application No.:** 17-112866-CO
Plan Submitted Option: pdf [File 1] [File 2] [File 3]	**Proposed use:** Multi-Family Apartments

APPEAL INFORMATION SHEET

Appeal item 1

Code Section	Oregon Plumbing Specialty Code - Storm Drainage 1101 .5.3.2
Requires	2016 Portland Stormwater Management Manual - Chapter 2.3.4.15, Drywells, Design Requirements, Setbacks Pg 2-118.
Proposed Design	The applicant is proposing the use of a drywell system to be installed in the open air parking lot adjacent to the new building. One of the drywells in the system will be located within 10 feet of the structure in order to provide an escape route in the event that a large storm event occurs, see 'Reason for Alternate' section for more information. The proposed drywell design has been reviewed by the Geotechnical Engineer per attached memorandum. The drywell system proposed for the building was sized to infiltrate the 10-year storm and was designed with an escape route to safely drain stormwater away from structures during large storm events. The sizing of the drywell systems was done using HydroCAD®. A design infiltration rate of 2 in/hr was used for calculations, see attached Stormwater Report. The drywell system will be tested at the time of installation to verify infiltration capacity. Feasibility of on-site infiltration: The feasibility of the drywell system location is based on infiltration testing, maintenance, structural design and strength of soils. Infiltration testing was performed by GeoDesign, Inc. and documented in the "Report of Geotechnical Engineering SeNices" dated July 22, 2016. This report shows measured infiltration rates of 4 in/hr onsite at a depth of 15.5 feet. The drywells are deep and will be discharging stormwater 7-22 feet below the bottom of the adjacent footings, which equates to 10-25 feet below grade. The infiltration rate of the deep soils will prevent saturation of the shallow soils directly underneath the buildings. See attachments for supporting data on the effectiveness of infiltration for the site.
Reason for alternative	The applicant proposes that one drywell in the system be installed approximately 7 feet from adjacent footings in order to achieve safe overland flow in the event of a major storm event. As a

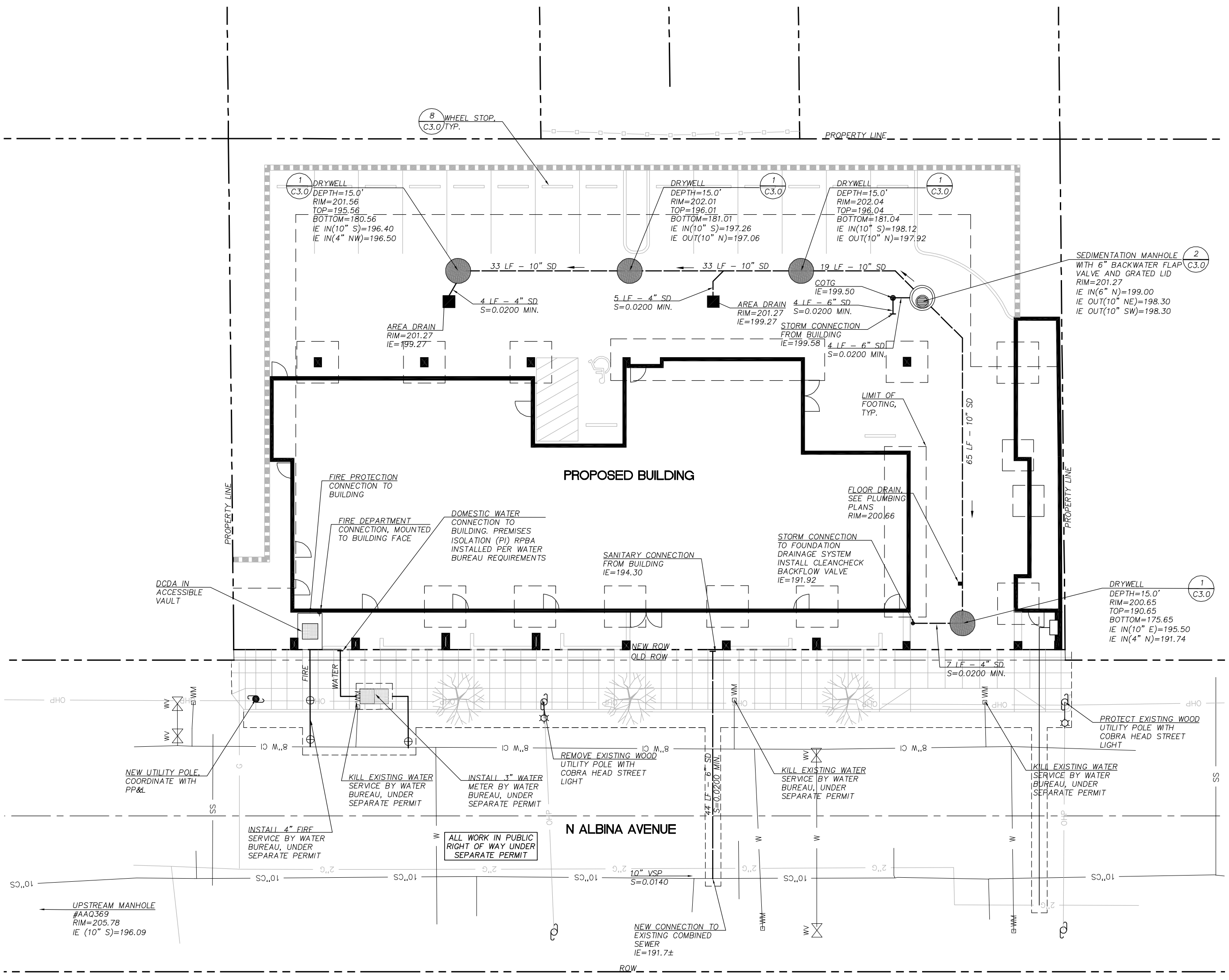
result, the drywell will not be in accordance with the OPSC.
Mitigation of Maintenance and Overflow Concerns:
The drywells will have an accessible, bolt down manhole rim located in open vehicle drive aisles to allow for maintenance as required by Oregon Department of Environmental Quality (ODEQ). The applicant has confirmed with a local company (River City Environmental Inc.) that a vacuum truck can reach lengths up to 300 feet for drywell maintenance, which allows for all of the drywells in the proposed system to be maintained. The drywells will be maintained by a professional management company who will follow the county recorded Operations and Maintenance Plan for the drywells.
Mitigation of Soil Bearing Concerns:
The bearing capacity of the footings will not significantly be affected by the by the proposed drywell, as explained in the attached memo from GeoDesign Inc, dated March 17, 2017.

APPEAL DECISION

Reduced setbacks for drywells: Denied.

Appellant may contact Joe Blanco (503) 823-2059 for more information.

Pursuant to City Code Chapter 25.07, you may appeal this decision to the Plumbing Code Board of Appeal within 180 calendar days of the date this decision is published. For information on the appeals process and costs, including forms, appeal fee, payment methods and fee waivers, go to www.portlandoregon.gov/bds/appealsinfo, call (503) 823-7300 or come in to the Development Services Center.



UTILITY PLAN

SCALE: 1"=10'

SHEET NOTES

1. ALL DOMESTIC WATER AND FIRE PROTECTION WORK IN THE PUBLIC RIGHT OF WAY BY PORTLAND WATER BUREAU AT OWNER'S EXPENSE. CONTRACTOR TO COORDINATE WORK WITH PORTLAND WATER BUREAU.
2. INSTALL 4" PERFORATED FOUNDATION DRAINAGE PIPE AROUND PERIMETER OF BUILDING INSTALLED PER DETAIL 3/C3.0. CONNECT PERFORATED PIPE TO SOLID PIPE WITH CLEANCHECK BACKFLOW PREVENTOR.
3. RETAINING WALL WITH GUARDRAIL BY STRUCTURAL.

SHEET LEGEND

●	DRYWELL	1 / C3.0
◎	SEDIMENTATION MANHOLE WITH GRATED LID	2 / C3.0
■	AREA DRAIN	7 / C3.0
▪	FLOOR DRAIN	
•	COTG	2 / C3.1

GRAPHIC SCALE
0 10 20
1 inch =10 ft.

NOTE: INFORMATION IN THESE DOCUMENTS IS NOT APPROVED FOR CONSTRUCTION UNTIL A BUILDING PERMIT HAS BEEN ISSUED.

PERMIT

EMERICK ARCHITECTS
321 SOUTHWEST FOURTH AVE SUITE 200 PORTLAND OR 97204
P 503.235.9400 W EMERICK-ARCHITECTS.COM

Humber
Design
Group, Inc.
Portland, OR • 503.946.6690 • hdgpdx.com

REGISTERED PROFESSIONAL ENGINEER
76,071
OREGON
JULY 12, 2005
MARTHA S. WILLIAMS
EXPIRES 6-30-2018

REV. #: DATE: DESCRIPTION

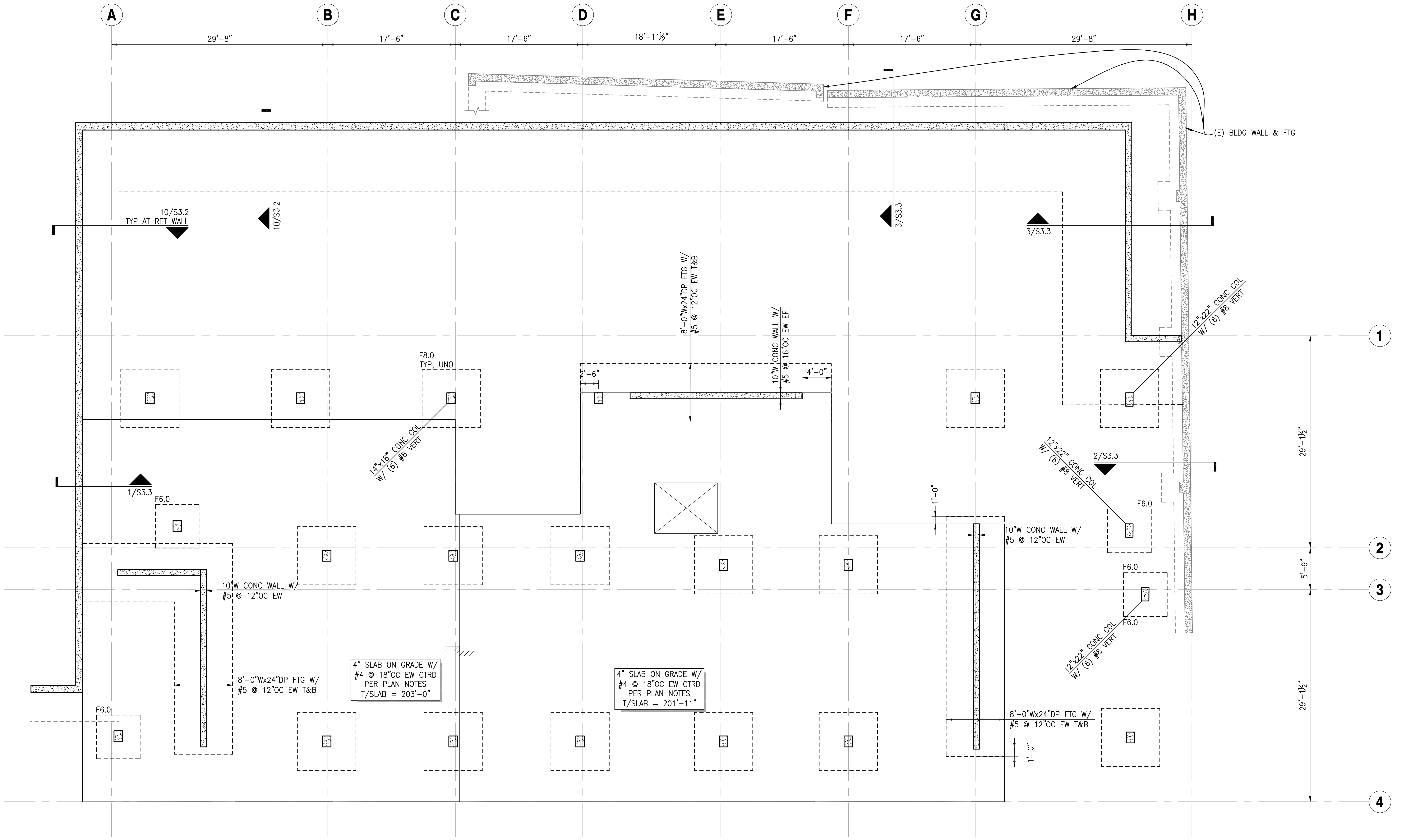
ALBINA APARTMENTS

4734 N. ALBINA AVE.
PORTLAND, OR 97217

UTILITY PLAN

PROJECT NO: 1531
DRWN BY: PLM CHK'D BY: MSW
DATE: 01.30.17

C2.0

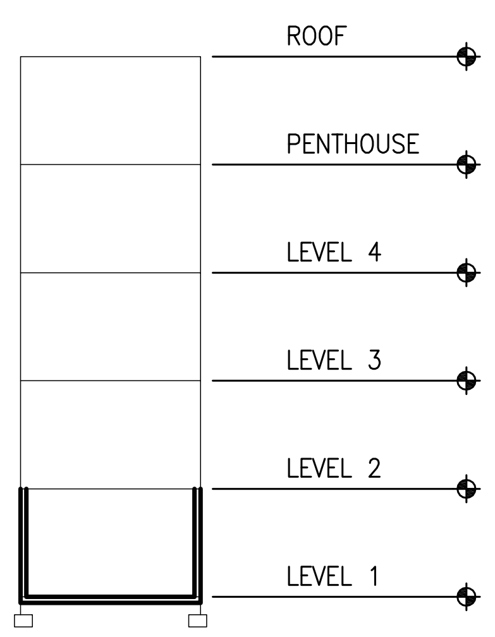


01410	SPREAD FOOTING SCHEDULE				
TYPE	SIZE			REINFORCING	COMMENTS
	LENGTH	WIDTH	DEPTH		
F6.0	6'-0"	6'-0"	14"	(7) #5B EW	----
F8.0	8'-0"	8'-0"	18"	(9) #6B EW	----

FOUNDATION PLAN NOTES:

1. STRUCTURAL GENERAL NOTES, DESIGN CRITERIA, ABBREVIATIONS AND LEGEND PER S1.1 AND S1.2.
2. VERIFY ALL DIMENSIONS AND ELEVATIONS WITH THE ARCHITECTURAL DRAWINGS. ALL EXISTING DIMENSIONS SHALL BE FIELD VERIFIED.
3. CONTRACTOR SHALL LOCATE AND VERIFY THE FOLLOWING WITH OTHERS PRIOR TO POURING CONCRETE: ALL DOOR OPENINGS IN FOUNDATION WALLS; DRAINS AND SLOPES; BLOCKOUTS FOR [POOLS, SPAS, FREEZERS, COOLERS, PLUMBING, SPRINKLERS AND HVAC]. ALL DUCTS, CHASES AND PIPES PER MECHANICAL, PLUMBING, ELECTRICAL AND SPRINKLER DRAWINGS. STAIR DETAILS AND GUARDRAILS PER ARCHITECTURAL DRAWINGS. CONCRETE CURBS AND LOCATIONS PER ARCHITECTURAL DRAWINGS.
4. TOP OF SLAB (T/SLAB) ELEVATION ASSUMED 0'-0". FOR ACTUAL T/SLAB ELEVATION REFER TO CIVIL AND ARCHITECTURAL DRAWINGS. PROVIDE 6 MIL VAPOR BARRIER BELOW SLAB AT INTERIOR SPACES. PROVIDE FREE-DRAINING GRANULAR FILL PER GEOTECH REPORT.
5. TYPICAL TOP OF INTERIOR (T/INTERIOR) FOOTING ELEVATION = -0'-8", UNO. TYPICAL TOP OF EXTERIOR (T/EXTERIOR) FOOTING ELEVATIONS = -1'-0", UNO.
6. ALL FOOTINGS AND SLABS TO BEAR ON COMPETENT NATIVE SOIL AND/OR STRUCTURAL FILL. SUBGRADE PREPARATION, STRUCTURAL FILL, FOOTING DRAINS, AND OTHER REQUIREMENTS PER GEOTECH REPORT AS NOTED IN THE STRUCTURAL GENERAL NOTES.
7. CJ INDICATES CONTROL JOINT PER PLAN.
8. CONTRACTOR TO VERIFY TOP OF CONCRETE (T/CONC) WALL ELEVATIONS ON ALL PARTIAL HEIGHT RETAINING WALLS. MAINTAIN T/WALL ELEVATION A MINIMUM OF 6" ABOVE FINISH GRADE PER 10/S3.2.
9. MOISTURE PROOF ALL CONCRETE STEM AND BASEMENT WALLS PER ARCHITECT.
10. STEEL STAIRS SHALL BE BIDDER-DESIGNED, UNO. APPLICABLE DESIGN REQUIREMENTS PER STRUCTURAL GENERAL NOTES.
11. TYPICAL DETAILS PER:

4/S3.2	TYPICAL COLUMN SECTIONS
5/S3.1	TYPICAL LAP SPLICE SCHEDULE
9/S3.1	STANDARD HOOKS AND BAR BENDS
2/S3.2	TYPICAL STEPPED FOOTING
5/S3.2	TYPICAL STAIR ON GRADE
4/S3.2	TYPICAL DEPRESSED SLAB DETAIL
7/S3.2	TYPICAL STEP AT SLAB ON GRADE
11/S3.2	TYPICAL CONCRETE WALL OPENING REINFORCEMENT



NOTE:
DARKENED LINES DESIGNATE AREA OF WORK.

LEVEL 1 FOUNDATION & MILD REINFORCING PLAN

SCALE: 1/8"=1'-0"

N

EMERICK ARCHITECTS
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P 503.235.9400 W EMERICK-ARCHITECTS.COM

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400 SW 6TH AVENUE • SUITE 605
PORTLAND, OREGON 97204
PHONE: (503) 242-2448 • FAX: (503) 242-2449
WEBSITE: www.dci-engineers.com
CIVIL / STRUCTURAL

FOR PERMIT

These drawings are sufficiently complete for submission to the jurisdiction having authority for permit. The Contractor shall not use these drawings for construction until Contractor receives written approval for use in construction by the jurisdiction having authority and DCI Engineers.

STRUCTURAL
REGISTERED PROFESSIONAL ENGINEER
74858PE
Shirley Chalupa
OREGON
MARCH 23, 2011
SHIRLEY CHALUPA
EXPIRES: 12-31-17

REV. #:	DATE:	DESCRIPTION

ALBINA
APARTMENTS
4732 N ALBINA AVE
PORTLAND, OR 97217

LEVEL 1
FOUNDATION &
MILD REINFORCING
PLAN

PROJECT NO: 1531
DCI PROJECT NO: 16031-0104
DRWN BY: IAK
CHK'D BY: SC
DATE: 12.20.16

S2.1

GEODESIGN INC

Memorandum

To:	Andy Schreck	From:	Reed S. Kistler, P.E. and Brett A. Shipton, P.E., G.E.
Company:		Date:	March 17, 2017
Address:	17154 Lowenberg Terrace Lake Oswego, OR 97034		
cc:	Paige Miller, Humber Design Group, Inc. (via email only) Marissa Brown, Emerick Architects (via email only)		
GDI Project:	SchreckA-1-01		
RE:	Dry Well Review Albina Apartments 4734 N Albina Avenue Portland, Oregon		

This memorandum documents our review of the proposed location and design of dry wells for the proposed Albina Apartments located at 4734 North Albina Avenue in Portland, Oregon. We prepared a geotechnical report[1] for this project.

We reviewed a utility plan prepared by Humber Design Group, Inc. that shows the locations of the proposed dry wells. We understand that four dry wells and one sedimentation manhole will be constructed on site. We understand that each dry well will have a minimum perforated length of 15 feet. In addition, we understand that one dry well will be located approximately 8 feet from a spread footing. For dry wells located within 10 feet of spread footings, we recommend that the top of the dry well perforated section be located a minimum of 5 feet below the elevation of adjacent footing subgrade.

Based on our review of the information provided, the proposed dry wells will not significantly affect bearing capacity of the footings. We recommend that the project structural engineer also review the proposed dry well design to verify they have sufficient structural capacity to resist surcharge loads from the adjacent footings.

RSK:BAS:rc
One copy submitted (via email only)
Document ID: SchreckA-1-01-031717-geom.docx

56207
July 11, 2000
BRETT A. SHIPTON
EXPIRES: 6/30/18

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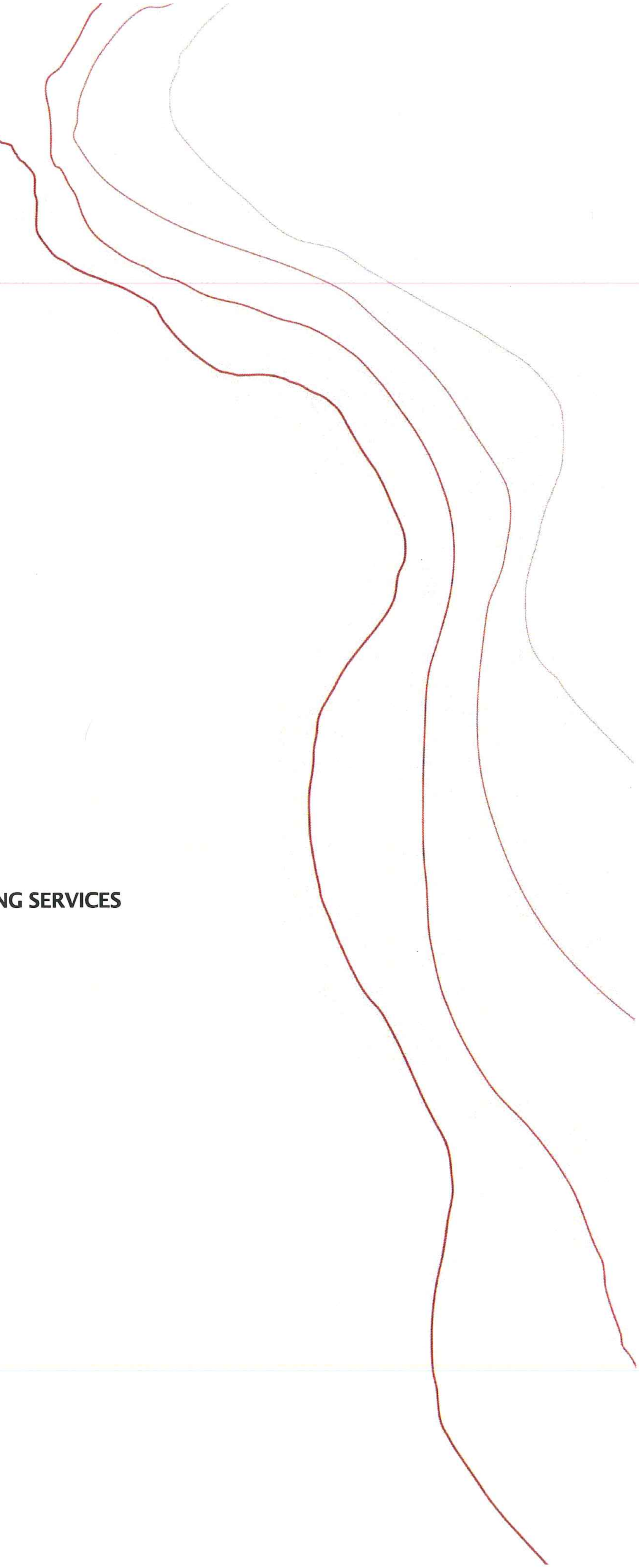
[1] GeoDesign, Inc. *Report of Geotechnical Engineering Services; Proposed Albina Apartments; 4732 N Albina Avenue; Portland, Oregon,* dated July 22, 2016. GeoDesign Project: SchreckA-1-01

GEODESIGN INC



REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Proposed Albina Apartments
4732 N Albina Avenue
Portland, Oregon

For
Andy Schreck
July 22, 2016

GeoDesign Project: SchreckA-1-01

GEODESIGN INC

July 22, 2016

Andy Schreck
17154 Lowenberg Terrace
Lake Oswego, OR 97034

Report of Geotechnical Engineering Services
Proposed Albina Apartments
4732 N Albina Avenue
Portland, Oregon
GeoDesign Project: SchreckA-1-01

GeoDesign, Inc. is pleased to submit our report of geotechnical engineering services for the proposed Albina Apartments development located at 4732 N Albina Avenue in Portland, Oregon. Our services for this project were conducted in accordance with our proposal dated June 20, 2016.

We appreciate the opportunity to be of service to you. Please call if you have questions regarding this report.

Sincerely,

GeoDesign, Inc.

Brett A. Shipton, P.E., G.E.
Principal Engineer

VCL:BAS:kt
Attachments
One copy submitted (via email only)
Document ID: SchreckA-1-01-072216-geor.docx

EXECUTIVE SUMMARY

The following is a summary of our findings and recommendations for design and construction of the proposed development. We recommend that the main report be referenced for a more thorough description of the subsurface conditions and geotechnical recommendations for the project.

- The proposed structure can be supported on shallow foundations bearing on firm native soil. Approximately 3 to 6 feet of fill was encountered in our explorations. Any fill material encountered at footing subgrade be removed and replaced with structural fill.

- We recommend that in all building slab areas where the cut is less than 3 feet, and fill is present at the subgrade elevation, the surface foot of material should either be removed and replaced with structural fill or the subgrade scarified and compacted as structural fill to a depth of 1 foot.

- If portions of the building will be embedded below ground surface, excavation sidewalls will require shoring if they are adjacent to existing settlement-sensitive structures or to stay within property boundaries.

- The fine-grained soil at the site can be sensitive to small changes in moisture content and difficult to adequately compact during wet weather or when the moisture content of the soil is more than a couple of percent above the optimum required for compaction. If the moisture content of the soil is currently above optimum, drying will be required if used as structural fill.

- The site will require demolition of existing buildings, concrete slabs, and other site features. In particular, wet, sensitive subgrade should be anticipated beneath the pavement areas.

TABLE OF CONTENTS

1.0 INTRODUCTION

GeoDesign, Inc. is pleased to submit this geotechnical engineering report for the proposed Albina Apartments development located at 4732 N Albina Avenue in Portland, Oregon. Figure 1 shows the site relative to existing physical features. Figure 2 shows the current site layout and our approximate exploration locations. Acronyms and abbreviations used herein are defined at the end of this document.

The property is currently occupied by two single-story warehouse structures; a paved parking area; and a fenced, grass-covered area on the north portion of the site. We understand that the existing structures will be demolished.

Plans are preliminary at the time of this report. Based on our review of a conceptual site plan, the development will consist of a new four-story apartment building on the west portion of the property along N Albina Avenue, with surface parking at the rear (east) of the property. A basement is not planned at this time. However, based on existing site grades, portions of the building (north and south) would possibly be embedded below the ground surface. Structural loads were not available at the time of this report. We have assumed that the column loads will be between 300 and 400 kips and wall loads will be less than 4 kips per foot. Floor slab loads are assumed to be less than 150 psf. Finish floor grades are not known; however, based on the existing topography, cuts of up to approximately 12 feet could be required to develop the site.

2.0 SCOPE OF SERVICES

The purpose of our geotechnical engineering services was to provide geotechnical engineering recommendations for use in design and construction of the proposed development. Our scope of work included the following:

- Reviewed readily available published geologic data and our in-house files for existing information on subsurface conditions in the site vicinity.
- Completed a subsurface exploration program consisting of three borings (B-1 through B-3) to depths ranging from approximately 26.5 to 41.5 feet BGS. Two borings (INF-1 and INF-2) were drilled to depths of 10.0 to 17.0 feet BGS for infiltration testing. Infiltration testing was conducted at depths of 15.5 and 8.5 feet BGS in borings INF-1 and INF-2, respectively.
- Maintained continuous logs of the explorations and collected samples at representative intervals.
- Performed the following laboratory tests:
  - Fifteen moisture content determinations in accordance with ASTM D 2216
  - Six fines content determinations in accordance with ASTM D 1140
- Provided recommendations for site preparation and grading, including demolition, temporary and permanent slopes, fill placement criteria, suitability of on-site soil for fill, subgrade preparation, and recommendations for wet weather construction.
- Provided foundation support recommendations for the proposed structure. Our recommendations include allowable bearing capacity and lateral resistance parameters.
- Provided recommendations for use in design of conventional retaining walls, including backfill and drainage requirements and lateral earth pressures.

- Evaluated groundwater conditions at the site, and provided general recommendations for dewatering during construction and subsurface drainage, if required.
- Provided recommendations for construction of asphalt pavements for on-site access roads and parking areas, including subbase, base course, and AC paving thickness.
- Provided seismic design recommendations in accordance with the procedures outlined in the 2012 IBC and 2014 SOSSC.
- Prepared this geotechnical engineering report that presents our findings, conclusions, and recommendations.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The site is located on the east side of N Albina Avenue in Portland, Oregon, and is surrounded by a mixt of residential and commercial properties. The site is currently occupied by two single-story buildings with a paved parking lot to the west of the buildings. The north approximately one-third of the site consists of a fenced grass area. Since the adjacent properties to the north, east, and south are located at higher elevations, the existing buildings are partially embedded on the east and south sides. There is a concrete retaining wall at the south boundary of the site that extends from the south building towards N Albina Avenue. The parking area on the west side of the buildings is relatively flat with ground surface elevations ranging from approximately 199 to 200 feet. The vacant grass area is sloped with an approximate ground surface elevation of 200 feet on the west side near N Albina Avenue, increasing up to an elevation of approximately 212 feet at the east side of the grass area.

3.2 SUBSURFACE CONDITIONS

3.2.1 General

We explored subsurface conditions at the site by drilling three borings (B-1 through B-3) to depths ranging from 26.5 to 41.5 feet BGS. Two borings (INF-1 and INF-2) were drilled adjacent to boring B-2 to conduct infiltration testing. The approximate exploration locations are shown on Figure 2. The exploration logs and laboratory test results are presented in the Appendix.

Our explorations generally encountered variable undocumented fill underlain by sand. At the location of boring B-1, which was drilled in the parking area, the surface consists of approximately 5 inches of concrete underlain by 4 inches of aggregate base. The following sections summarize the subsurface units encountered.

3.2.2 Undocumented Fill

Fill was encountered to depths of approximately 3 and 6 feet BGS in borings B-1 and B-3, respectively. Boring B-3 was drilled in the grass area in the north portion of the site. Fill was not encountered in boring B-2. The fill consists of medium dense, silty sand and medium stiff to stiff silt with varying amounts of sand. In boring B-3, the fill includes trace organics consisting of charcoal and rootlets. Laboratory testing on a selected sample of the fill indicates the moisture content was approximately 24 percent at the time of our explorations.

3.2.3 Sand

The fill is underlain by silty sand and sand with silt. The sand is generally medium dense to dense. Laboratory testing on selected samples of the sand indicates the moisture contents varied from approximately 13 to 27 percent at the time of our explorations.

3.2.4 Groundwater

We did not observe groundwater in our explorations. Based on our review of water well logs on file with the Oregon Water Resources Department and projects completed in the site vicinity, groundwater is generally anticipated at a depth greater than 50 feet BGS. The depth to groundwater may fluctuate in response to seasonal changes, prolonged rainfall, changes in surface topography, and other factors not observed in this study.

3.3 INFILTRATION TESTING

Infiltration testing was completed to assist in the evaluation of stormwater infiltration facilities for the project. The infiltration testing was conducted in general accordance with the recommendations for the "Encased Falling Head" method included in the 2014 City of Portland Stormwater Management Manual. We performed the falling-head infiltration tests in the borings within a 10-inch-diameter casing. The infiltration rate was measured under low-head conditions of approximately 12 inches of water or less after saturated conditions had been achieved.

A representative sample was collected below the infiltration test depths for grain-size analysis. Table 1 summarizes the infiltration test results and fines content determinations. The exploration logs, a description of the infiltration test procedures, and the laboratory test results are presented in the Appendix.

Table 1. Infiltration Rates

Location	Depth (feet BGS)	Material	Infiltration Rate (inches/hour)	Fines Content[1] (percent)
INF-1	15.5	Sand with Silt	4	11
INF-2	8.5	Sand with Silt	4	7

1. Fines content: material passing a U.S. Standard No. 200 sieve

The infiltration rates provided in Table 1 are measured rates and are unfactored. Correction factors should be applied to the measured infiltration rates by the civil engineer during design to account for the degree of long-term maintenance and influent/pre-treatment control, as well as the potential for long-term clogging due to siltation and bio-buildup, depending on the proposed length, location, and type of infiltration facility. In addition, correction factors to be applied to the test results are provided in Exhibit F.2-1 of the 2014 City of Portland Stormwater Management Manual.

4.0 DESIGN RECOMMENDATIONS

4.1 GENERAL

The following sections provide our design recommendations for the project. All site preparation and structural fill should be prepared as recommended in the "Construction" section of this report.

4.2 SHALLOW FOUNDATIONS

4.2.1 General

The proposed structure can be supported on conventional spread footings bearing on undisturbed, firm native soil. Footings should not be directly supported on soft, loose soil or undocumented fill. We anticipate that most of the undocumented fill will be removed with site grading and excavation for the footings, and any remaining fill in the footing subgrade after cutting should be removed and replaced with structural fill.

4.2.2 Bearing Capacity

We recommend that spread footings bearing on the sand be sized based on an allowable bearing pressure of 3,000 psf. This is a net bearing pressure; the weight of the footing and overlying backfill can be ignored in calculating footing sizes. The recommended allowable bearing pressures apply to the total of dead and long-term live loads and may be increased by one-third for short-term loads, such as those resulting from wind or seismic forces.

We recommend that isolated column and continuous wall footings have minimum widths of 24 and 18 inches, respectively. The bottom of exterior footings should be founded at least 18 inches below the lowest adjacent grade. Interior footings should be founded at least 12 inches below the base of the floor slab.

4.2.3 Lateral Resistance

Lateral loads on footings can be resisted by passive earth pressure on the sides of the structure and by friction on the base of the footings. Our analysis indicates that the available passive earth pressure for footings confined by native soil and structural fill is 350 pcf, modeled as an equivalent fluid pressure. Adjacent floor slabs, pavements, or the upper 12-inch depth of adjacent unpaved areas should not be considered when calculating passive resistance.

A coefficient of friction equal to 0.30 may be used when calculating resistance to sliding for footings in direct contact with the sand. Footings in contact with crushed rock should be designed using a coefficient of friction of 0.40.

4.2.4 Settlement

We anticipate that total post-construction settlement will be less than 1 inch for spread foundations designed in accordance with the recommendations provided above. Differential settlement between similarly loaded footings is expected to be less than ½ inch.

4.2.5 Subgrade Observation

All footing and floor subgrades should be evaluated by a representative of GeoDesign to evaluate the bearing conditions. Observations should also confirm that all loose or soft material, organics, unsuitable fill, and softened subgrades (if present) have been removed. Localized deepening of footing excavations may be required to penetrate any deleterious material.

4.3 FLOOR SLABS

To help reduce moisture transmission and to provide uniform support, we recommend a minimum 6-inch-thick layer of floor slab base rock be placed and compacted over prepared subgrade. The floor slab base rock should meet the requirements in the "Materials" section of this report and compacted to at least 95 percent of ASTM D 1557.

Vapor barriers are often required by flooring manufacturers to protect flooring and adhesives. Many flooring manufacturers will warrant their products only if a vapor barrier is installed according to their recommendations. Selection and design of the appropriate vapor barrier (if needed) should be based on discussions among members of the design team. We can provide additional information to assist you with your decision.

Slabs should be reinforced according to their proposed use and per the structural engineer's recommendations. Load-bearing concrete slabs may be designed assuming a modulus of subgrade reaction, k, of 120 psi per inch.

4.4 RETAINING STRUCTURES

As indicated above, the north and south ends of the proposed building will likely be embedded below the ground surface. In addition, retaining walls will be required at the north, east, and south sides of the site, assuming the finish grade of the surface parking will be similar to that of N Albina Avenue. If shoring is required to protect the adjacent buildings or other settlement-sensitive structures during excavation for the site, our recommendations are described in the "Shoring" section of this report. Walls should be designed to resist the earth pressures developed by the shoring system, unless the shoring is designed as a permanent installation. Permanent retaining structures not in contact with temporary shoring should be designed as recommended below.

Our recommendations for permanent retaining walls are based on the following assumptions: (1) the walls are not in contact with temporary shoring, (2) the walls consist of conventional, cantilevered retaining walls or embedded building walls, (3) the walls are less than 15 feet in height, (4) the retained soil is level, and (5) drainage is provided behind the walls to prevent hydrostatic pressures for developing. Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project vary from these assumptions.

Walls not restrained from rotation should be designed using an equivalent fluid pressure of 35 pcf. An equivalent fluid pressure of 55 pcf should be used for design of walls restrained from rotation. These values do not consider hydrostatic pressures. Permanent basement walls with more than one level of bracing should be designed to resist lateral earth pressures presented on Figure 3.

Seismic earth pressures on embedded walls should be designed using a dynamic force of $7H^2$ pounds per linear foot of wall, where H is the wall height. This seismic force should be applied as a distributed load throughout the excavated depth of the retaining wall, with the centroid located at a distance of 0.6H from the base of the wall.

4.5 PAVEMENT

New pavement should be installed on competent subgrade or new engineered fill prepared in conformance with the "Site Preparation" and "Materials" sections of this report. Given the building proposed, our pavement recommendations are based on the assumption that the standard-duty traffic section will be subject to passenger cars and occasional maintenance and delivery truck trucks. We do not have specific information on the frequency and types of vehicles that will use the area; however, we have assumed that standard traffic conditions will consist of a maximum of 2 trucks per day and a maximum of 200 cars per day. We recommend the heavy-duty pavement section be constructed in areas that will be subject to higher traffic volumes (such as entrances and areas subject to repeated delivery vehicles). The heavy-duty section assumes traffic will consist of up to ten trucks per day.

We calculated pavement sections using the above-referenced traffic conditions using a design life of 10 and 20 years and AASHTO design methods. The design of the recommended pavement section is based on an assumed resilient modulus of 4,000 psi and the assumption that construction will be completed during an extended period of dry weather. Wet weather construction may require an increased thickness of aggregate base to support the rock trucks and compaction equipment. Table 2 summarizes the recommended pavement sections.

Table 2. Pavement Section Thickness

Design Life (years)	**Standard-Duty Section**		**Heavy-Duty Section**	
	AC Thickness (inches)	**Aggregate Base Thickness (inches)**	**AC Thickness (inches)**	**Aggregate Base Thickness (inches)**
10	2.5	7.0	3.0	10.0
20	2.5	8.0	3.5	10.0

The AC and aggregate base should meet the specifications for ACP and aggregate base rock provided in the "Materials" section of this report.

Construction traffic should be limited to non-building, unpaved portions of the site or haul roads. Construction traffic should not be allowed on new pavements. If construction traffic is to be allowed on newly constructed road sections, an allowance for this additional traffic will need to be made in the design pavement section.

4.6 SEISMIC DESIGN CONSIDERATIONS

4.6.1 IBC Parameters

Based on our explorations, the following design parameters can be applied if the building is designed using the applicable provisions of the 2012 IBC and 2014 SOSSC. The parameters in Table 3 should be used to compute seismic base shear forces.

Table 3. IBC Seismic Design Parameters

Seismic Design Parameter	Short Period (T_s = 0.2 second)	1 Second Period (T_1 = 1.0 second)
MCE Spectral Acceleration, S	S_s = 0.97 g	S_1 = 0.42 g
Site Class	D	
Site Coefficient, F	F_a = 1.11	F_v = 1.58
Adjusted Spectral Acceleration, S_M	S_{MS} = 1.08 g	S_{M1} = 0.66 g
Design Spectral Response Acceleration Parameters, S_D	S_{DS} = 0.72 g	S_{D1} = 0.44 g
Design Spectral PGA	0.29 g	

4.6.2 Liquefaction

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. The excessive buildup of pore water pressure results in the sudden loss of shear strength in a soil. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water. In general, loose, saturated sand soil with low silt and clay content is the most susceptible to liquefaction. Low plasticity, silty sand may be moderately susceptible to liquefaction under relatively higher levels of ground shaking. Liquefaction is not considered a site hazard.

5.0 CONSTRUCTION

5.1 SITE PREPARATION

5.1.1 Demolition

Demolition includes the complete removal of the existing structures, concrete footings, pavement, utilities, and various other former site improvements that may be encountered during construction. We recommend that all abandoned underground vaults, underground storage tanks, septic tanks, manholes, utility lines, foundation elements, and other subsurface structures that are beneath new structural components be entirely removed.

Voids resulting from the removal of improvements should be backfilled with compacted structural fill, as discussed in the "Structural Fill" section of this report. Utility lines abandoned under new structural components should be completely removed and backfilled with structural fill. Firm subgrade should be exposed at the bottom of the excavations before backfilling, and the sides of the temporary excavations should be sloped at a minimum of 1.5H:1V.

Demolished material should be transported off site for disposal. Soft soil encountered during site preparation should be replaced with structural fill.

5.1.2 Clearing

There are some grass areas and trees at the site that will need to be removed. In addition, stumps and root balls should be grubbed out to the depth of the roots, which could exceed 3 feet BGS. Depending on the methods used to remove the root balls, considerable disturbance and loosening of the subgrade could occur during site grubbing. We recommend that soil disturbed during grubbing operations be removed to expose firm, undisturbed subgrade. The resulting excavations should be backfilled with structural fill.

Where present, the existing topsoil zone should be stripped and removed from all fill areas. The average depth of stripping for vegetated areas will be approximately 1 to 2 inches, although greater stripping depths may be required to remove localized zones of loose or organic soil. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal or used in landscaped areas. Stripping should extend at least 5 feet beyond the limits of proposed structural areas.

5.1.3 Fill Improvement

Fill generally consisting of medium dense, silty sand and medium stiff to stiff silt with sand and trace organics was encountered in our explorations to depths of approximately 3 to 6 feet BGS. The thicker fill was encountered in boring B-3, which was drilled in the grass mound at the north end of the site.

Within all proposed structural fill, pavement, at-grade floor slabs, and improvement areas; for a 5-foot margin beyond such areas; and where less than 3 feet of cut is required, if fill is observed at the subgrade elevation, we recommend that the surface foot of the stripped subgrade be removed and replaced with structural fill or the subgrade scarified and compacted as structural fill to a depth of 1 foot.

The exposed subgrade should be closely evaluated by a geotechnical engineer during the process. Considerable soil processing, including moisture conditioning and the removal of roots or other deleterious material from the soil, may be required to use the excavated material as structural fill. Because of the moisture-sensitive nature of the on-site soil, scarification and compaction of the subgrade should be completed during the summer dry period. Compaction should be performed as described in the "Materials" section of this report.

5.1.4 Subgrade Evaluation

Upon completion of demolition, clearing, and subgrade stabilization, and prior to the placement of fill, structures, or pavement improvements, the exposed subgrade should be evaluated by proof rolling. Based on the results of our explorations, our experience with the local soil conditions, and experience with subgrade under prior structures (especially building slabs), we anticipate that relatively easily disturbed soil will be encountered under the existing buildings. The silty sand can be easily damaged during demolition and construction activities. Methods to protect the subgrade from disturbance are provided in the "Construction Considerations" section of this report.

A member of our geotechnical staff should observe the exposed subgrade after demolition, site cutting, and fill removal have been completed to determine if there are additional areas of unsuitable or unstable soil. Our representative should observe a proof roll with a fully loaded dump truck or similar heavy, rubber-tired construction equipment to identify soft, loose, or unsuitable areas. Areas that appear to be too wet and soft to support proof rolling equipment should be evaluated by probing and prepared in accordance with the recommendations for wet weather construction presented in the "Construction Considerations" section of this report.

5.2 CONSTRUCTION CONSIDERATIONS

The fine-grained soil present on this site is easily disturbed. If not carefully executed, site preparation, utility trench work, and excavations can create extensive soft areas and significant repair costs can result. Earthwork planning, regardless of the time of year, should include considerations for minimizing subgrade disturbance.

If construction occurs during or extends into the wet season, or if the moisture content of the surficial soil is more than a couple percentage points above optimum, site stripping and cutting may need to be accomplished using track-mounted equipment. Likewise, the use of granular haul roads and staging areas will be necessary for support of construction traffic during the rainy season or when the moisture content of the surficial soil is more than a few percentage points above optimum. The base rock thickness for pavement areas is intended to support post-construction design traffic loads. This design base rock thickness may not support construction traffic or pavement construction when the subgrade soil is wet. Accordingly, if construction is planned for periods when the subgrade soil is wet, staging and haul roads with increased thicknesses of base rock will be required. The amount of staging and haul road areas, as well as the required thickness of granular material, will vary with the contractor's sequencing of a project and type/frequency of construction equipment. Based on our experience, between 12 and 18 inches of imported granular material is generally required in staging areas and between 18 and 24 inches in haul roads areas. Stabilization material may be used as a substitute provided the top 4 inches of material consists of imported granular material. The actual thickness will depend on the contractor's means and methods and, accordingly, should be the contractor's responsibility. In addition, a geotextile fabric should be placed as a barrier between the subgrade and imported granular material in areas of repeated construction traffic. The imported granular material, stabilization material, and geotextile fabric should meet the specifications in the "Materials" section of this report.

5.3 EXCAVATION

5.3.1 General

Conventional heavy earthmoving equipment in proper working condition should be capable of making necessary excavations of the on-site soil for site cuts and utilities. Soil with more sand content may be prone to raveling, and shoring will be required to maintain vertical excavation walls and protect adjacent facilities.

5.3.2 Temporary Slopes

Where construction slopes are possible, temporary slopes of 1.5H:1V for excavation of the basement may be used to vertical depths of 15 feet or less, provided groundwater seepage is not encountered. At this inclination, the slopes will likely ravel and require some ongoing repair. If seepage is encountered, the slopes should be flattened to protect the surface from raveling. All cut slopes should be protected from erosion by covering them with plastic sheeting during the rainy season. If sloughing or instability is observed, the slope might need to be flattened or the cut supported by shoring.

Excavations should not undermine adjacent utilities, foundations, walkways, streets, or other hardscapes unless special shoring or underpinned support is provided. We recommend a minimum horizontal distance of 5 feet from the edge of the existing improvements to the top of the temporary slope. Unsupported excavations should not be conducted within a downward and outward projection of a 1H:1V line from 2 feet outside the edge of an adjacent structural feature.

5.3.3 Utility Trench Excavation

Trench cuts should stand vertical to a depth of approximately 4 feet in competent soil provided groundwater seepage does not occur in the trench walls. As discussed in the "Temporary Slopes" section of this report, open excavation techniques may be used to excavate trenches with depths up to 10 feet, provided the walls of the excavation are cut at a slope of 1H:1V, groundwater seepage is not present, and surcharge loads are not present within 10 feet of the top of the slope. The walls of the trench should be flattened or braced for stability and a dewatering system installed if seepage is encountered or excessive sloughing and caving occurs. Use of a trench box or other approved temporary shoring is recommended for cuts below the water table. If shoring is used, we recommend that the type and design of the shoring system be the responsibility of the contractor who is in the best position to choose a system that fits the overall plan of operation.

5.3.4 Excavation Dewatering

Excavation dewatering might be required to maintain dry working conditions in excavations depending on the time of year and the severity of rainfall during construction. Based on the results of previous studies at the site, groundwater is anticipated to be relatively deep, at a depth greater than 50 feet BGS. However, perched or static groundwater could be present at shallower depths after prolonged wet periods. Excavation dewatering will be necessary if groundwater is encountered. Dewatering systems are best designed by the contractor; however, assuming that excavations will not exceed more than approximately 6 to 8 feet BGS, it is our opinion that it should be possible to remove groundwater encountered by pumping from a sump in trenches. More intense use of pumps may be required at certain times of the year and where more intense seepage occurs. Removed water should be routed to a suitable discharge point.

If groundwater is present at the base of utility excavations, we recommend placing up to 12 inches of stabilization material at the base of the excavation. Specifications for stabilization material are provided in the "Materials" section of this report.

5.3.5 Safety

All excavations should be made in accordance with applicable OSHA and state regulations. While we have described certain approaches to utility trench excavations in the foregoing discussion, the contractor should be responsible for selecting the excavation and dewatering methods, monitoring the trench excavations for safety, and providing shoring as required to protect personnel and adjacent improvements.

5.4 SHORING

5.4.1 General

If excavations for site development are within the influence zone of the footings of the adjacent structures, shoring will be required to protect the adjacent structures. The influence zone of the existing footings generally extends downwards at a 1H:1V slope from the bottom corner of the footings. We recommend the locations and depths of the existing footings be checked in the field to verify these assumptions. We have provided recommendations below for shoring design.

5.4.2 Lateral Earth Pressures

Shoring should be designed using the values on Figure 4. The recommended design parameters for cantilevered shoring and anchored shoring are shown on Figure 4. Shoring with one level of anchors or bracing should be designed using a triangular pressure distribution as shown for a cantilevered wall on Figure 4 (left). Shoring with more than one level of anchors or bracing should be designed using the earth pressure diagram provided on Figure 4 (right). The above equivalent fluid pressures do not include effects from surcharge loads. The values on Figure 5 can be used to compute surcharge-induced lateral earth pressures.

5.4.3 Soldier Piles

Structural design of the soldier piles should consider the lateral earth pressures discussed above. In addition to lateral earth pressures, the soldier piles will be subject to compressive forces as a result of the downward component of the tieback anchor loads. We recommend a minimum soldier pile embedment of 10 feet below the base of the excavation. We recommend an allowable end bearing capacity of 4 ksf for piles embedded in the sand. An allowable skin friction of 0.5 ksf between the grout and surrounding soil is recommended. In addition, we recommend the grout at the tip of the pile have sufficient strength to withstand the imposed loads. These values should be verified by the structural engineer designing the shoring. Grout should be placed using tremie pipe methods.

We anticipate that lagging will consist of pressure-treated lumber. To maintain the integrity of the excavation, prompt and careful installation of lagging, particularly in areas of seepage and loose soil, is recommended. All voids behind the lagging should be backfilled promptly. To minimize the risk of hydrostatic pressures from developing behind the wall, lean concrete or other low-permeability material should not be used as backfill.

5.4.4 Tieback Anchors

We have provided recommendations for anchored or braced shoring if necessary. The bonded zone for the tieback anchors should be maintained outside of the "unbonded zone" shown on Figure 4. We anticipate the tieback anchors will be capable of achieving allowable bond strengths of between 3 and 5 kips per foot in the sand, depending on the method of construction. A variety of methods are available for construction of tieback anchors. Therefore, we recommend the contractor be responsible for selecting the appropriate bonded length and installation methods to achieve the required anchor capacity. Tieback anchors should be locked off at 100 percent of the design load.

Prior to installing production anchors, we recommend performance tests be conducted on a minimum of two anchors. The purpose of these tests is to verify the installation procedure selected by the contractor before a large number of anchors are installed. Performance tests should be performed to 150 percent of the design load and in accordance with the guidelines provided in *Recommendations for Prestressed Rock and Soil Anchors* (Post Tensioning Institute, 2014).

We recommend proof tests be conducted on all production anchors in accordance with the guidelines presented in *Recommendations for Prestressed Rock and Soil Anchors*. The anchors should be proof tested to at least 133 percent of the design load.

5.4.5 Shoring Dewatering

We do not anticipate dewatering will be required for shoring systems; however, the contractor should be prepared to employ dewatering techniques if necessary. The selection, design, and construction of the temporary dewatering system should be the responsibility of the contractor who is in the best position to modify or adapt the system to changing groundwater conditions and construction sequencing and requirements. The construction dewatering system should be adaptable to varying flow and conditions and capable of lowering the level of the groundwater to a minimum of 2 feet below the base of the excavation.

5.5 DRAINAGE

Where possible, the finished ground surface around the building should be sloped away from the structure at a minimum 2 percent gradient for a distance of at least 5 feet. Downspouts or roof scuppers should discharge into a storm drain system that carries the collected water to an appropriate stormwater system. Trapped planter areas should not be created adjacent to the building without providing means for positive drainage (e.g., swales or catch basins).

5.6 PERMANENT SLOPES

Permanent cut and fill slopes should not exceed 2H:1V. Access roads and pavements should be located at least 5 feet from the top of cut and fill slopes. The setback should be increased to 10 feet for buildings. The slopes should be planted with appropriate vegetation to provide protection against erosion as soon as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

5.7 MATERIALS

5.7.1 Structural Fill

5.7.1.1 General

Fill should be placed on subgrade that has been prepared in conformance with the "Site Preparation" section of this report. A variety of material may be used as structural fill at the site. However, all material used as structural fill should be free of organic matter or other unsuitable material and should meet the specifications provided in OSSC 00330 (Earthwork), OSSC 00400 (Drainage and Sewers), and OSSC 02600 (Aggregates), depending on the application. A brief characterization of some of the acceptable materials and our recommendations for their use as structural fill is provided below.

5.7.1.2 On-Site Soil

The native on-site soil is suitable for use as general structural fill, provided it is properly moisture conditioned; free of debris, organic material, and particles over 3 inches in diameter; and meets the specifications provided in OSSC 00330.12 (Borrow Material). We anticipate some moisture conditioning may be required to dry the soil to a moisture content near optimum. This will require an extended period of dry weather, typically experienced between early July and mid-October. It will be difficult, if not impossible, to adequately compact on-site soil during the rainy season or during prolonged periods of rainfall.

When used as structural fill, the on-site soil should be placed in lifts with a maximum uncompacted thickness of 6 to 8 inches and compacted to not less than 92 percent of the maximum dry density for fine-grained soil and 95 percent of the maximum dry density for granular soil, as determined by ASTM D 1557.

5.7.1.3 Imported Granular Material

Imported granular material used as structural fill should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in OSSC 00330.14 (Selected Granular Backfill) or OSSC 00330.15 (Selected Stone Backfill). The imported granular material should also be angular, fairly well graded between coarse and fine material, have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, and have at least two fractured faces.

Imported granular material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557. During the wet season or when wet subgrade conditions exists, the initial lift should be approximately 18 inches in uncompacted thickness and should be compacted by rolling with a smooth-drum roller without using vibratory action.

5.7.1.4 Stabilization Material

Stabilization material used in staging or haul road areas, or as trench stabilization material, should consist of 4- or 6-inch-minus pit- or quarry-run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in OSSC 00330.15 (Selected Stone Backfill). The material should have a maximum particle size of 6 inches, have less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve, and have at least two mechanically fractured faces. The material should be free of organic matter and other deleterious material. Stabilization material should be placed in lifts between 12 and 24 inches thick and compacted to a firm condition.

5.7.1.5 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should consist of well-graded granular material with a maximum particle size of 1½ inches and less than 10 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in OSSC 00405.13 (Pipe Zone Material). The pipe zone backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D 1557, or as required by the pipe manufacturer or local building department.

Within roadway alignments, the remainder of the trench backfill up to the subgrade elevation should consist of well-graded granular material with a maximum particle size of 2½ inches and less than 10 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in OSSC 00405.14 (Trench Backfill; Class B, C, or D). This material should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D 1557, or as required by the pipe manufacturer or local building department. The upper 3 feet of the trench backfill should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D 1557.

Outside of structural improvement areas (e.g., roadway alignments or building pads) trench backfill placed above the pipe zone may consist of general fill material that is free of organics and material over 6 inches in diameter and meets the specifications provided in OSSC 00405.14 (Trench Backfill; Class A, B, C, or D). This general trench backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D 1557, or as required by the pipe manufacturer or local building department.

5.7.1.6 Drain Rock

Drain rock should consist of angular, granular material with a maximum particle size of 2 inches and should meet the specifications provided in OSSC 00430.11 (Granular Drain Backfill Material). The material should be free of roots, organic matter, and other unsuitable material; have less than 2 percent by dry weight passing the U.S. Standard No. 200 sieve (washed analysis); and have at least two mechanically fractured faces. Drain rock should be compacted to a well-keyed, firm condition.

5.7.1.7 Aggregate Base Rock

Imported granular material used as base rock for building floor slabs and pavements should consist of ¾- or 1½-inch-minus material (depending on the application) and meet the requirements in OSSC 00641 (Aggregate Subbase, Base, and Shoulders). In addition, the aggregate should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve. The aggregate base should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557.

5.7.1.8 Retaining Wall Select Backfill

Backfill material placed behind retaining walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of select granular material that meets the specifications provided in OSSC 00510.12 (Granular Wall Backfill) or OSSC 00510.13 (Granular Structure Backfill).

The backfill should be placed and compacted as recommended for structural fill, with the exception of backfill placed immediately adjacent to walls. Backfill adjacent to walls should be compacted to a lesser standard to reduce the potential for generation of excessive pressure on the walls. Backfill located within a horizontal distance of 3 feet from the retaining walls should be compacted to approximately 90 percent of the maximum dry density, as determined by ASTM D 1557. Backfill placed within 3 feet of the wall should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (such as a jumping jack or vibratory plate compactor). If flatwork (slabs, sidewalk, or pavement) will be placed adjacent to the wall, we recommend that the upper 2 feet of fill be compacted to 95 percent of the maximum dry density, as determined by ASTM D 1557.

5.7.1.10 Recycled On-Site Material

On-site AC, conventional concrete, and aggregate base or gravel may be used as fill if they are processed to meet the requirements for their intended use and the use of these materials do not result in an environmental concern. Processing includes crushing and screening, grinding in place, or other methods to meet the requirements for structural fill as described above. The processed material should be fairly well graded and contain no metal, organic, or other deleterious material. The processed material may be mixed with on-site soil or imported fill to assist in achieving the gradation requirements. We recommend that processed recycled fill have the maximum particle sizes listed in Table 4.

Table 4. Processed Fill Maximum Particle Size

Depth of Placement[1]	Maximum Particle Size
0 to 2 feet	½ inch
2 to 6 feet	2 inches
6 to 10 feet	4 inches
deeper than 10 feet	8 inches

1. below subgrade of structural element

Recycled on-site fill material should not be used within a depth of 2 feet from foundations, floor slabs, pavements, or other subsurface elements. We also caution that excavation through recycled material that is placed as structural fill may be difficult if a significant fraction of oversized particles is present. In addition, these excavations may also be prone to raveling and caving.

5.7.1.11 AC

The AC should be Level 2, ½-inch, dense ACP according to OSSC 00744 (Asphalt Concrete Pavement). Minimum lift thickness for ½-inch ACP is 2.0 inches. Asphalt binder should be performance graded and conform to PG 64-22. The AC should be compacted using minimum and maximum lifts of 2.0 and 3.0 inches, respectively.

5.7.1.12 Geotextile Fabric

Subgrade Geotextile Fabric

A subgrade geotextile fabric should be placed as a barrier between the subgrade and granular material in staging areas, haul road areas, or in areas of repeated construction traffic. The geotextile should meet the specifications provided in OSSC 02320 (Geosynthetics) for separation geotextiles (Table 02320-4) and be installed in accordance with OSSC 00350 (Geosynthetic Installation). The geotextile should have a Level "B" certification.

Drainage Geotextile Fabric

Drain rock, and other granular material used for subsurface drains, should be wrapped in a geotextile fabric that meets the specifications provided in OSSC 02320 (Geosynthetics) for drainage geotextiles (Table 02320-1) and be installed in accordance with OSSC 00350 (Geosynthetic Installation).

5.8 EROSION CONTROL

The site soil is susceptible to erosion; therefore, erosion control measures should be carefully planned and in place before construction begins. Surface water runoff should be collected and directed away from slopes to prevent water from running down the slope face. Erosion control measures (such as straw bales, sediment fences, and temporary detention and settling basins) should be used in accordance with local and state ordinances.

6.0 OBSERVATION OF CONSTRUCTION

Satisfactory foundation and earthwork performance depends to a large degree on quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during the subsurface exploration. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect if subsurface conditions change significantly from those anticipated.

We recommend that GeoDesign be retained to observe earthwork activities, including stripping, proof rolling of the subgrade and repair of soft areas, footing subgrade preparation, performing laboratory compaction and field moisture-density tests, observing final proof rolling of the pavement subgrade and base rock, and asphalt placement and compaction.

7.0 LIMITATIONS

We have prepared this report for use by Andy Schreck and members of the design and construction teams for the proposed project. The data and report can be used for bidding or estimating purposes, but our report, conclusions, and interpretations should not be construed as warranty of the subsurface conditions and are not applicable to other nearby building sites.

Exploration observations indicate soil conditions only at specific locations and only to the depths penetrated. They do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, re-evaluation will be necessary.

The site development plans and design details were preliminary at the time this report was prepared. When the design has been finalized and if there are changes in the site grades or location, configuration, design loads, or type of construction for the buildings, and walls, the conclusions and recommendations presented may not be applicable. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification.

The scope does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with generally accepted practices in this area at the time the report was prepared. No warranty, express or implied, should be understood.

◆ ◆ ◆

We appreciate the opportunity to be of service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

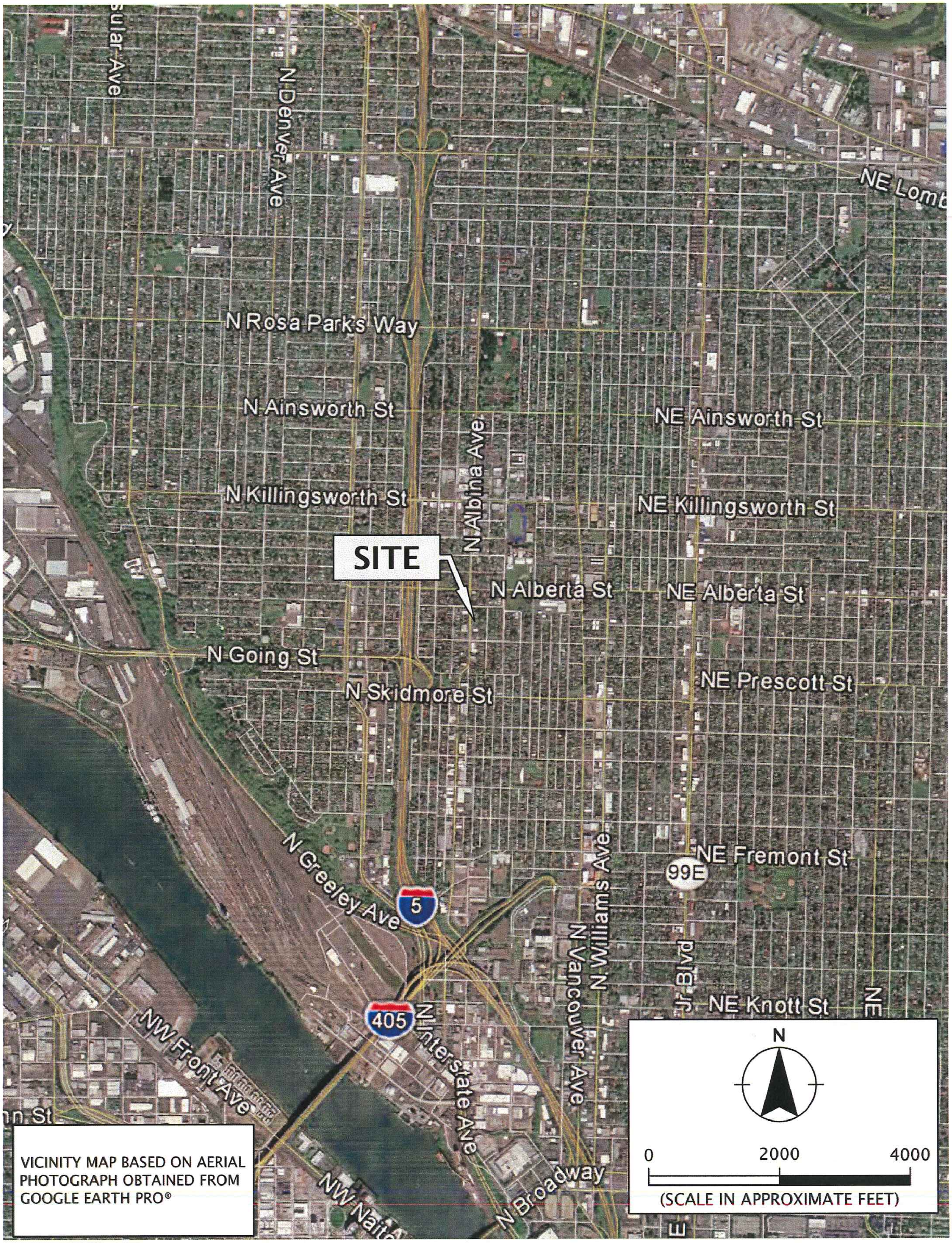
GeoDesign, Inc.

Viola C. Lai, P.E., G.E.
Project Engineer

Brett A Shipton, P.E., G.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
56207
OREGON
July 11, 2000
BRETT A. SHIPTON
EXPIRES: 6/30/18

FIGURES



VICINITY MAP BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®

GEODESIGN INC
15575 SW Sequoia Parkway - Suite 100
Portland OR 97224
Off 503.968.8787 Fax 503.968.3068

SCHRECKA-1-01	**VICINITY MAP**		
JULY 2016	PROPOSED ALBINA APARTMENTS PORTLAND, OR	**FIGURE 1**	

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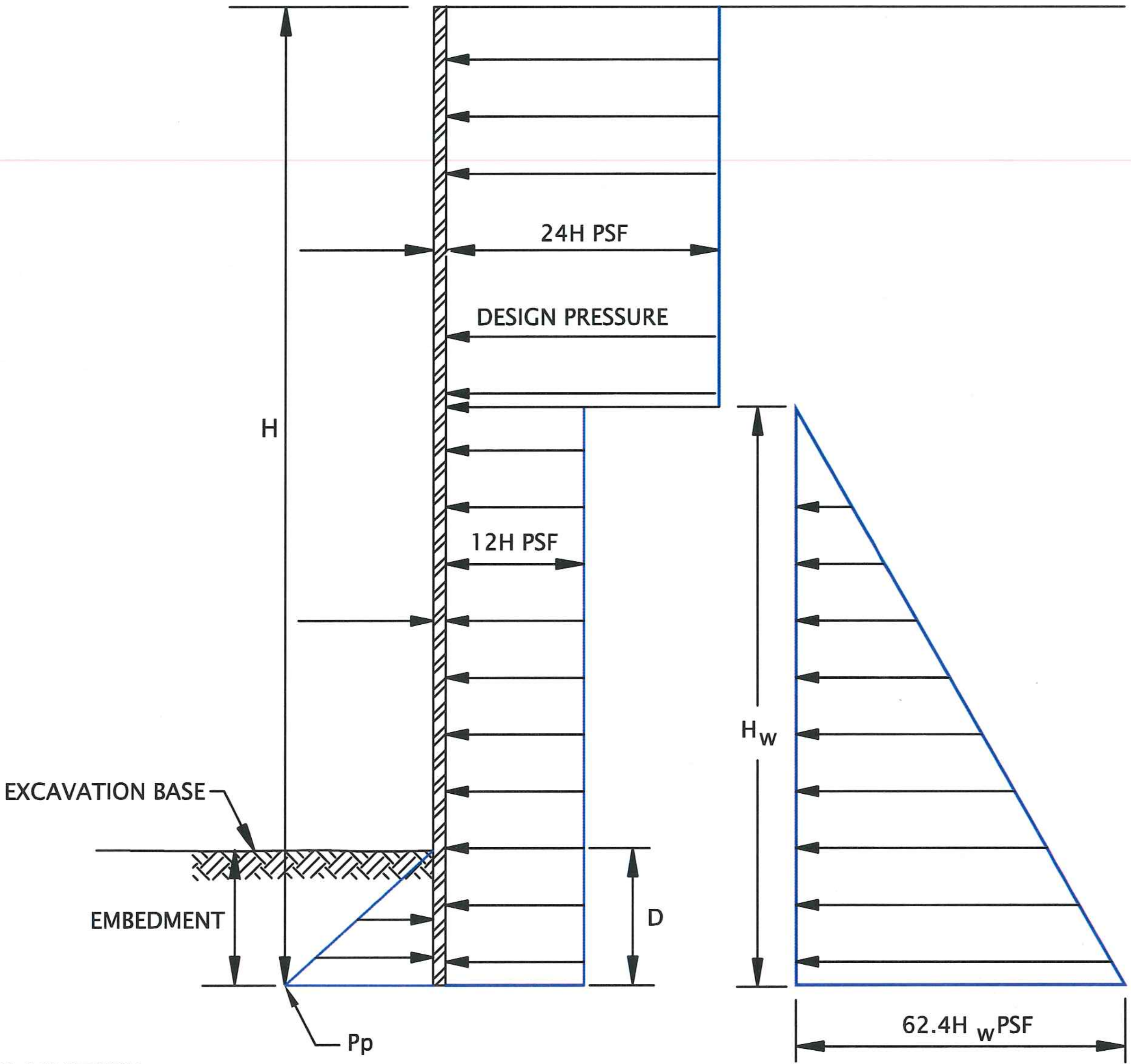
File Name: J:\S-Z\SchreckA\SchreckA-1\SchreckA-1-01\Figures\CAD\SchreckA-1-01-VM01.dwg | Layout: FIGURE 1

SITE PLAN BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®, JULY 11, 2016

GEODESIGN INC 15575 SW Sequoia Parkway - Suite 100 Portland OR 97224 Off 503.968.8787 Fax 503.968.3068	SCHRECKA-1-01	SITE PLAN	
	JULY 2016	PROPOSED ALBINA APARTMENTS PORTLAND, OR	FIGURE 2

RECOMMENDED DESIGN PARAMETERS FOR BRACED BASEMENT WALLS WITH HYDROSTATIC PRESSURES



EXPLANATION:

Pp = 300D PCF, 180D PCF (BELOW GROUNDWATER)
H_W = DEPTH OF WATER ABOVE BOTTOM OF WALL IN FEET
H = DEPTH OF WALL IN FEET
D = EFFECTIVE PILE EMBEDMENT DEPTH

NOTES:

1. FIGURE SHOULD BE USED IN CONJUNCTION WITH REPORT TEXT.
2. SURCHARGE LOADS ASSOCIATED WITH CONSTRUCTION ACTIVITIES AND TRAFFIC OR ADJACENT STRUCTURES SHOULD BE ADDED TO THE EARTH PRESSURE SHOWN ABOVE, WHERE APPLICABLE.
3. ASSUMES WALL IS INTERALLY BRACED AT MORE THAN ONE LEVEL.
4. HYDROSTATIC PRESSURE CAUSED BY THE STATIC GROUNDWATER TABLE ON THE ACTIVE PRESSURE SIDE SHOULD BE INCLUDED IN FINAL DESIGN. CONSTRUCTION DEWATERING EXTERNAL TO THE EXCAVATION WILL BE REQUIRED TO REMOVE PERCHED GROUNDWATER BEFORE THE EXCAVATION IS ADVANCED.
5. THE LATERAL EARTH PRESSURES ARE UNFACTORED.
6. VALUES DO NOT INCLUDE SEISMIC LOADS.

GEODESIGN INC
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SCHRECKA-1-01

JULY 2016

LATERAL EARTH PRESSURES FOR PERMANENT BASEMENT WALLS

PROPOSED ALBINA APARTMENTS
PORTLAND, OR

FIGURE 3

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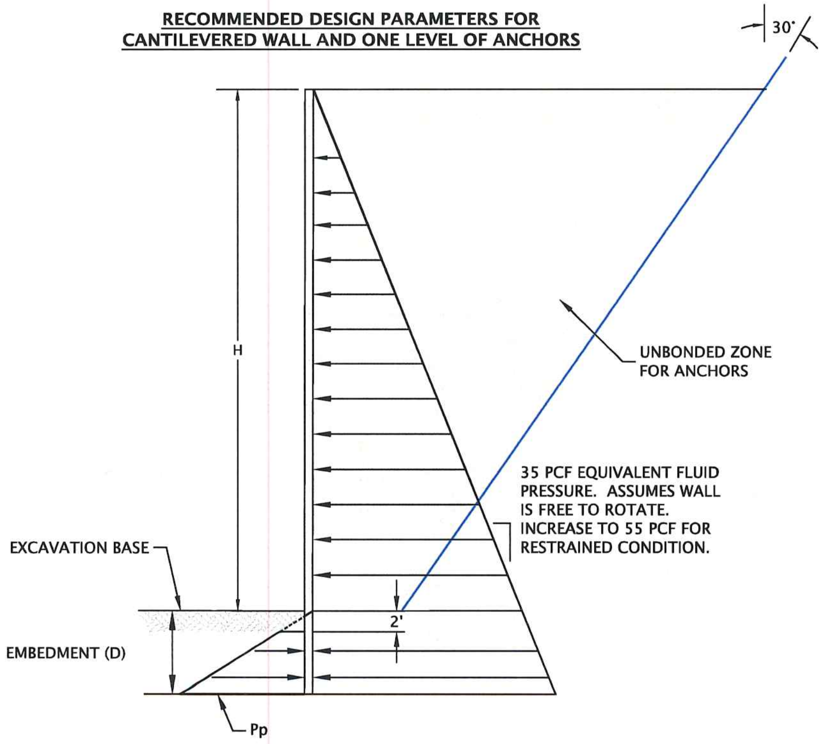
File Name: J:\S-Z\SchreckA\SchreckA-1\SchreckA-1-01\Figures\CAD\SchreckA-1-01-DET01.dwg | Layout: FIGURE 3

RECOMMENDED DESIGN PARAMETERS FOR CANTILEVERED WALL AND ONE LEVEL OF ANCHORS



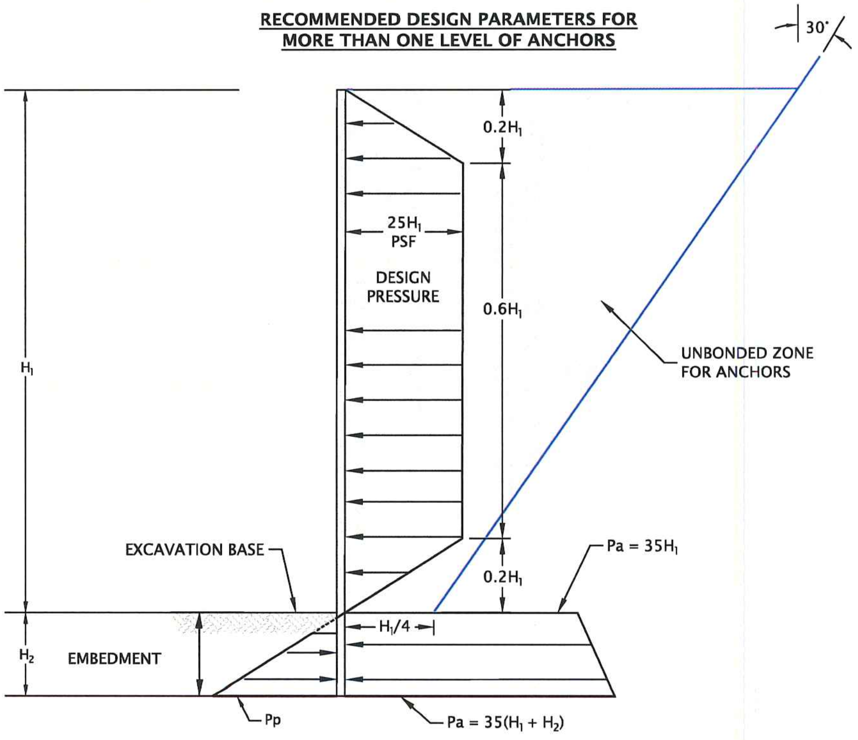
EXPLANATION:

Pp = 350D PCF (ABOVE GROUNDWATER)
H = DEPTH OF SOLDIER PILE IN FEET
D = SOLDIER PILE EMBEDMENT DEPTH
PASSIVE PRESSURE ACTS OVER 3X THE PILE WIDTH
ACTIVE PRESSURE ACTS OVER 1X THE PILE WIDTH BELOW EXCAVATION BASE

NOTES:

1. FIGURE DOES NOT INCLUDE LATERAL EARTH PRESSURES INDUCED BY SLOPED BACKFILL OR SURROUNDING LOADS.
2. LATERAL EARTH PRESSURES BASED ON WATER TABLE BELOW DEPTH OF SOLDIER PILE EMBEDMENT.
3. THE LATERAL EARTH PRESSURES ARE UNFACTORED.

RECOMMENDED DESIGN PARAMETERS FOR MORE THAN ONE LEVEL OF ANCHORS



EXPLANATION:

Pp = 300H_2 PCF
H_1 = DEPTH OF SOLDIER PILE EXPOSED HEIGHT IN FEET
H_2 = SOLDIER PILE EMBEDMENT DEPTH IN FEET
PASSIVE PRESSURE ACTS OVER 3X THE PILE WIDTH
ACTIVE PRESSURE ACTS OVER 1X THE PILE WIDTH BELOW THE EXCAVATION BASE

NOTES:

1. DOES NOT INCLUDE SURCHARGE OR SEISMIC LOADS.
2. TIEBACKS SHOULD BE LOCKED OFF AT 100 PERCENT OF DESIGN LOAD.
3. THE LATERAL EARTH PRESSURES ARE UNFACTORED.
4. PASSIVE PRESSURE RESISTANCE SHOULD BE NEGLECTED 2 FEET BELOW THE BOTTOM OF THE EXCAVATION.
5. LATERAL EARTH PRESSURES BASED ON WATER TABLE BELOW DEPTH OF SOLDIER PILE EMBEDMENT.

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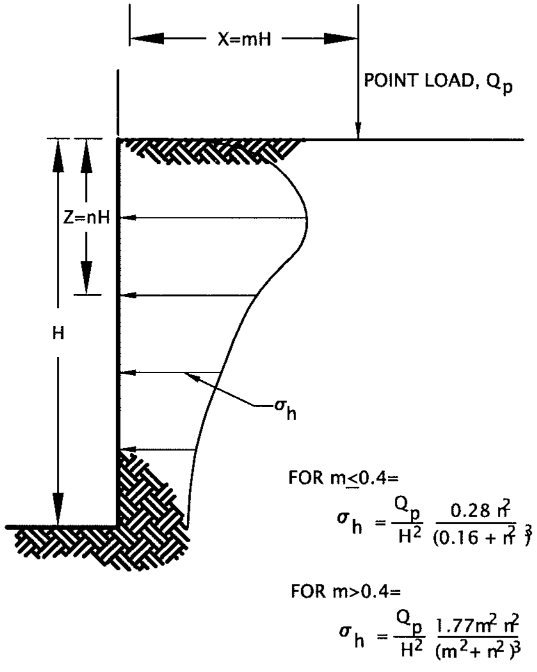
JULY 2016

CANTILEVERED AND BRACED WALLS DESIGN CRITERIA

PROPOSED ALBINA APARTMENTS
PORTLAND, OR

FIGURE 4

Printed By: aday | Print Date: 7/21/2016 3:24:46 PM
File Name: J:\S-Z\SchreckA\SchreckA-1\SchreckA-1-01\Figures\CAD\SchreckA-1-01-DET03.DWG | Layout: FIGURE 5

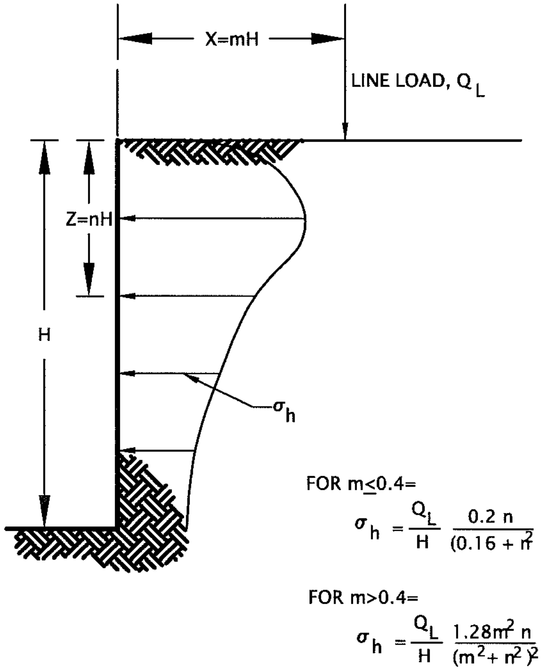


FOR $m \leq 0.4$=

$$\sigma_h = \frac{Q_p}{H^2} \frac{0.28\, n^2}{(0.16 + n^2)^3}$$

FOR $m > 0.4$=

$$\sigma_h = \frac{Q_p}{H^2} \frac{1.77 m^2 n^2}{(m^2 + n^2)^3}$$



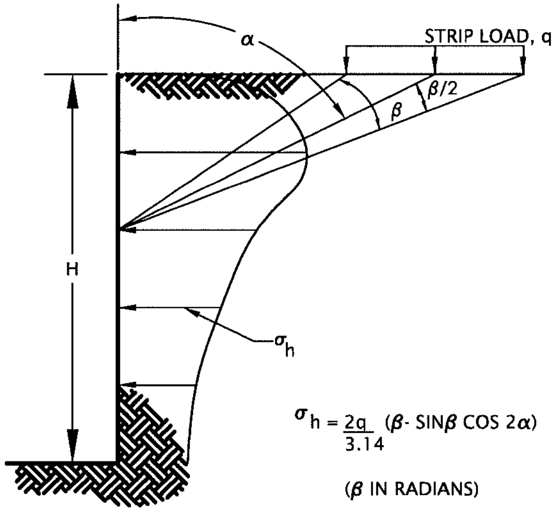
FOR $m \leq 0.4$=

$$\sigma_h = \frac{Q_L}{H} \frac{0.2\, n}{(0.16 + n^2)^2}$$

FOR $m > 0.4$=

$$\sigma_h = \frac{Q_L}{H} \frac{1.28 m^2 n}{(m^2 + n^2)^2}$$

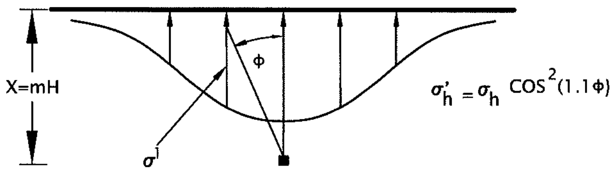
LINE LOAD PARALLEL TO WALL



$$\sigma_h = \frac{2q}{3.14} (\beta - \text{SIN}\beta \text{ COS } 2\alpha)$$

(β IN RADIANS)

STRIP LOAD PARALLEL TO WALL



$$\sigma'_h = \sigma_h \text{ COS}^2(1.1\Phi)$$

DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

NOTES:

1. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.
2. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.
3. VALUES ON THIS FIGURE ARE UNFACTORED.

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SURCHARGE-INDUCED LATERAL EARTH PRESSURES

PROPOSED ALBINA APARTMENTS
PORTLAND, OR

FIGURE 5

APPENDIX

APPENDIX

FIELD EXPLORATIONS

GENERAL

Our field explorations consisted of three borings (B-1 through B-3) drilled to depths ranging from approximately 26.5 to 41.5 feet BGS. Two borings (INF-1 and INF-2) were drilled near boring B-2 for performing infiltration testing. The borings were drilled on July 5, 2016 by Western States Soil Conservation, Inc., of Hubbard, Oregon. Borings B-1 through B-3 were drilled using mud rotary drilling methods. The infiltration borings (INF-1 and INF-2) were drilled using hollow-stem auger drilling methods. The exploration logs are presented in this appendix.

The approximate locations of our explorations are shown on Figure 2. The exploration locations were chosen based on a preliminary site plan provided to our office by Andy Schreck. The locations of the explorations were determined in the field by pacing from existing site features. This information should be considered accurate only to the degree implied by the methods used.

SOIL SAMPLING

The explorations were observed by a member of our geology staff. We obtained representative samples of the various soil encountered in the explorations for geotechnical laboratory testing. Soil samples were obtained from the borings using SPT sampling methods. SPTs were performed in general conformance with ASTM D 1586. The sampler was driven with a 140-pound hammer free-falling 30 inches. The number of blows required to drive the sampler 1 foot, or as otherwise indicated, into the soil is shown adjacent to the sample symbols on the exploration logs. Disturbed samples were obtained from the split barrel for subsequent classification and index testing. Sampling intervals are shown on the exploration logs

The average efficiency of the automatic SPT hammer used by Western States Soil Conservation, Inc. was 85 percent. The calibration testing results are presented at the end of this appendix.

SOIL CLASSIFICATION

The soil samples were classified in accordance with the "Exploration Key" (Table A-1) and "Soil Classification System" (Table A-2), which are presented in this appendix. The exploration logs indicate the depths at which the soil or its characteristics change, although the change could be gradual. A horizontal line between soil types indicates an observed (visual or drill action) change. If the change occurred between sample locations and was not observed or obvious, the depth was interpreted and the change is indicated using a dashed line. Classifications are shown on the exploration logs.

LABORATORY TESTING

CLASSIFICATION

The soil samples were classified in the laboratory to confirm field classifications. The laboratory classifications are shown on the exploration log if those classifications differed from the field classifications.

MOISTURE CONTENT

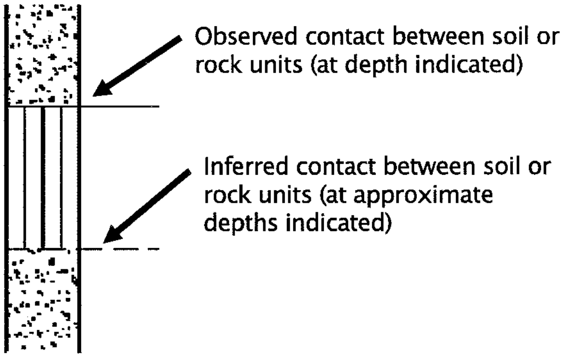
We tested the natural moisture content of selected samples in general accordance with ASTM D 2216. The natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The test results are presented in this appendix.

GRAIN-SIZE TESTING

Grain-size testing was performed on selected samples to determine the distribution of soil particle sizes. The testing consisted of particle-size analysis completed in accordance with percent fines determination (percent passing the U.S. Standard No. 200 sieve) completed in general accordance with ASTM D 1140 (P200). The test results are presented in this appendix.

SYMBOL	SAMPLING DESCRIPTION
◨	Location of sample obtained in general accordance with ASTM D 1586 Standard Penetration Test with recovery
▮	Location of sample obtained using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D 1587 with recovery
▤	Location of sample obtained using Dames & Moore sampler and 300-pound hammer or pushed with recovery
▤	Location of sample obtained using Dames & Moore and 140-pound hammer or pushed with recovery
▩	Location of sample obtained using 3-inch-O.D. California split-spoon sampler and 140-pound hammer
☒	Location of grab sample
▯	Rock coring interval
▽	Water level during drilling
▼	Water level taken on date shown

Graphic Log of Soil and Rock Types



Observed contact between soil or rock units (at depth indicated)

Inferred contact between soil or rock units (at approximate depths indicated)

GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits	PP	Pocket Penetrometer
CBR	California Bearing Ratio	P200	Percent Passing U.S. Standard No. 200 Sieve
CON	Consolidation		
DD	Dry Density	RES	Resilient Modulus
DS	Direct Shear	SIEV	Sieve Gradation
HYD	Hydrometer Gradation	TOR	Torvane
MC	Moisture Content	UC	Unconfined Compressive Strength
MD	Moisture-Density Relationship	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal
P	Pushed Sample		

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
P	Pushed Sample	NS	No Visible Sheen
PID	Photoionization Detector Headspace Analysis	SS	Slight Sheen
		MS	Moderate Sheen
ppm	Parts per Million	HS	Heavy Sheen

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EXPLORATION KEY

TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOILS

Relative Density	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)
Very Loose	0 – 4	0 - 11	0 - 4
Loose	4 – 10	11 - 26	4 - 10
Medium Dense	10 – 30	26 - 74	10 - 30
Dense	30 – 50	74 - 120	30 - 47
Very Dense	More than 50	More than 120	More than 47

CONSISTENCY - FINE-GRAINED SOILS

Consistency	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)	Unconfined Compressive Strength (tsf)
Very Soft	Less than 2	Less than 3	Less than 2	Less than 0.25
Soft	2 - 4	3 – 6	2 - 5	0.25 - 0.50
Medium Stiff	4 - 8	6 – 12	5 - 9	0.50 - 1.0
Stiff	8 - 15	12 – 25	9 - 19	1.0 - 2.0
Very Stiff	15 - 30	25 – 65	19 – 31	2.0 - 4.0
Hard	More than 30	More than 65	More than 31	More than 4.0

PRIMARY SOIL DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOILS (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVELS (< 5% fines)	GW or GP	GRAVEL
		GRAVEL WITH FINES (≥ 5% and ≤ 12% fines)	GW-GM or GP-GM	GRAVEL with silt
			GW-GC or GP-GC	GRAVEL with clay
		GRAVELS WITH FINES (> 12% fines)	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND (50% or more of coarse fraction passing No. 4 sieve)	CLEAN SANDS (<5% fines)	SW or SP	SAND
		SANDS WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SANDS WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS (50% or more passing No. 200 sieve)	SILT AND CLAY	Liquid limit less than 50	ML	SILT
			CL	CLAY
			CL-ML	silty CLAY
			OL	ORGANIC SILT or ORGANIC CLAY
		Liquid limit 50 or greater	MH	SILT
			CH	CLAY
			OH	ORGANIC SILT or ORGANIC CLAY
HIGHLY ORGANIC SOILS			PT	PEAT

MOISTURE CLASSIFICATION

Term	Field Test
dry	very low moisture, dry to touch
moist	damp, without visible moisture
wet	visible free water, usually saturated

ADDITIONAL CONSTITUENTS

Secondary granular components or other materials such as organics, man-made debris, etc.

Percent	Silt and Clay In:		Percent	Sand and Gravel In:	
	Fine-Grained Soils	Coarse-Grained Soils		Fine-Grained Soils	Coarse-Grained Soils
< 5	trace	trace	< 5	trace	trace
5 - 12	minor	with	5 - 15	minor	minor
> 12	some	silty/clayey	15 - 30	with	with
			> 30	sandy/gravelly	Indicate %

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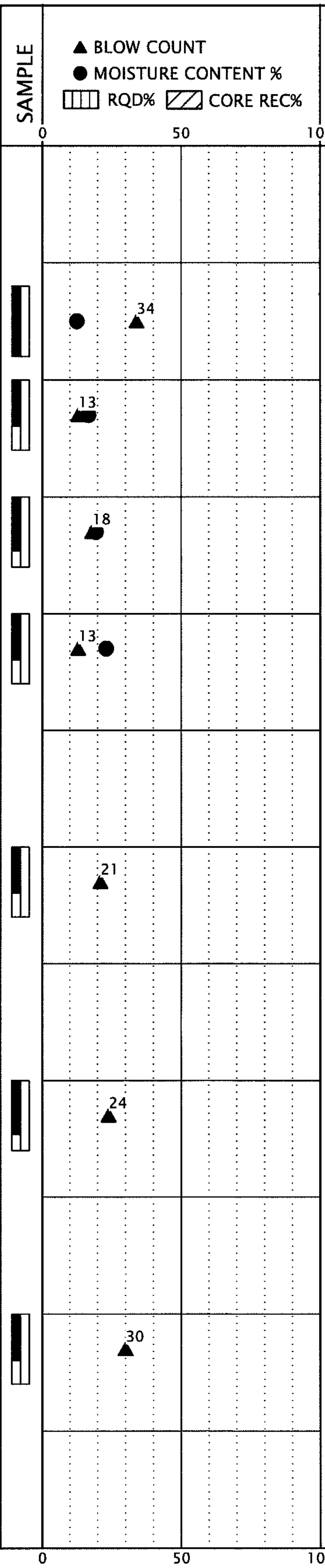
SOIL CLASSIFICATION SYSTEM

TABLE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	CONCRETE (5.0 inches).	199.0 198.6 0.4		
	AGGREGATE BASE (4.0 inches).	198.2 0.8		
	Medium dense, brown, silty SAND (SM); moist, fine - **FILL**.			
2.5				
	Dense, dark gray-brown SAND with silt (SP-SM); moist, medium.	196.0 3.0		Abandoned utility encountered at 3.0 feet. Chipped due to mud loss.
5.0	medium dense; layers of coarse sand (0.5 to 3 inches thick) at 5.0 feet			
7.5	trace organics (tree roots); without interbeds at 7.5 feet		P200	P200 = 12%
10.0	increase in silt at 10.0 feet			
12.5				
15.0	trace clay; ~10% decomposed sand clasts at 15.0 feet			
17.5				
	Medium dense, dark gray-brown SAND (SP), trace silt; moist, medium to coarse.	180.5 18.5		
20.0				
	Medium dense, gray-brown SAND with silt (SP-SM); moist, fine to medium.	178.5 20.5		
22.5				
	Medium dense, dark gray-brown SAND (SP), trace silt; moist, medium to coarse and subrounded.	176.0 23.0		
25.0				
	Medium dense to dense, light gray-brown SAND with silt (SP-SM); moist, fine.	173.0 26.0 172.5 26.5		
27.5	Exploration completed at a depth of 26.5 feet.			
	Hammer efficiency factor is 85 percent.			
30.0				



DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: JCH
COMPLETED: 07/05/16

BORING METHOD: mud rotary (see document text)
BORING BIT DIAMETER: 3 7/8 inches

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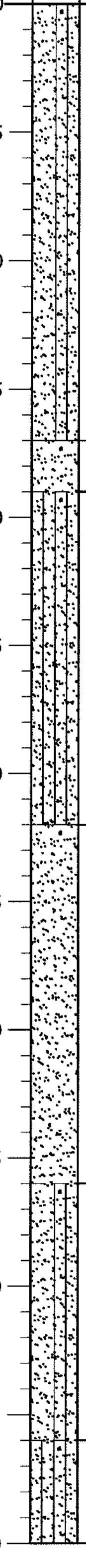
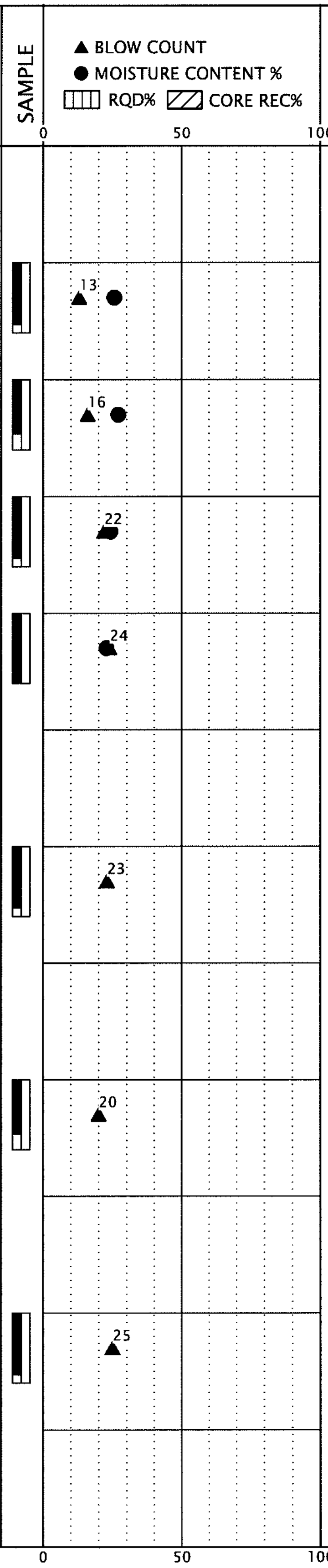
BORING B-1

PROPOSED ALBINA APARTMENTS
PORTLAND, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	Medium dense, gray-brown SAND with silt (SP-SM); moist, medium, interbeds of medium dense, gray-brown, silty SAND (SM) (2 to 3 inches thick) (3-inch-thick root zone).	200.0		
5.0	without interbeds at 5.0 feet			
	Medium dense, dark gray-brown SAND (SP), trace silt; moist, medium to coarse and subangular.	191.5 8.5		
10.0	Medium dense, gray-brown, silty SAND (SM); moist, fine to medium.	190.5 9.5	P200	P200 = 14%
	Medium dense, dark gray-brown SAND (SP), trace silt; moist, medium to coarse and subangular.	184.0 16.0		
20.0	gray-brown; interbeds of medium dense, gray-brown SAND with silt (SP-SM); moist, fine to medium (1 to 2 inches thick) at 20.0 feet			
	Medium dense, gray-brown SAND with silt (SP-SM); moist, fine to medium.	177.0 23.0		
	Dense, dark gray-brown, silty SAND (SM); moist, medium to coarse and subangular to subrounded.	172.0 28.0		
30.0				





DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JCH COMPLETED: 07/05/16

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 4 7/8 inches

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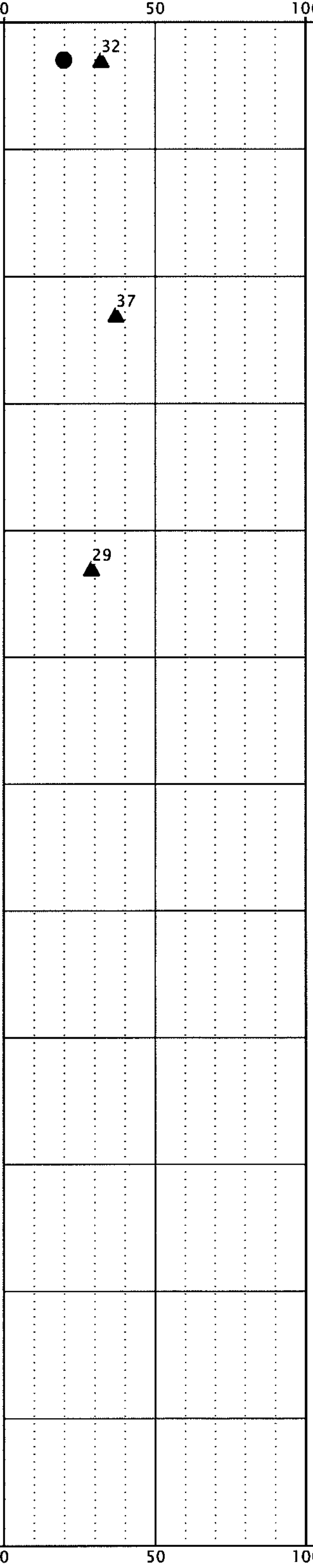
BORING B-2

PROPOSED ALBINA APARTMENTS
PORTLAND, OR

FIGURE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC%	INSTALLATION AND COMMENTS
30.0	(continued from previous page)		P200		32	P200 = 13%
32.5						
35.0	interbeds of gray-brown SAND with silt (SP-SM); moist, fine to medium (~2 inches thick) at 35.0 feet				37	
37.5						
40.0	medium dense; without interbeds at 40.0 feet				29	
	Exploration completed at a depth of 41.5 feet. Hammer efficiency factor is 85 percent.	158.5 41.5				
42.5						
45.0						
47.5						
50.0						
52.5						
55.0						
57.5						
60.0						



DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: JCH

COMPLETED: 07/05/16

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

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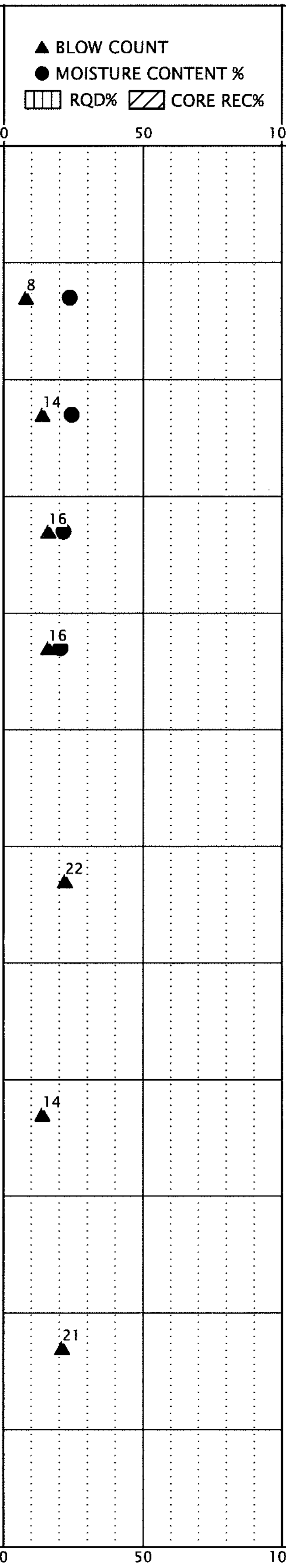
BORING B-2
(continued)

PROPOSED ALBINA APARTMENTS
PORTLAND, OR

FIGURE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	Medium stiff to stiff, red-brown SILT with sand (ML), trace organics (charcoal and rootlets); moist, sand is fine (3-inch-thick root zone) - **FILL**.	210.0			
2.5	sandy at 3.5 feet			8	
5.0	stiff, minor sand at 5.0 feet			14	
	Medium dense, gray-brown, silty SAND (SM); moist, fine.	204.0 6.0			
7.5	Medium dense, gray-brown SAND with silt (SP-SM); moist, fine to medium, interbeds of medium dense, gray-brown, silty SAND (SM); moist, fine (1 to 2 inches thick).	203.0 7.0		16	
10.0	Medium dense, gray-brown, silty SAND (SM); moist.	200.0 10.0	P200	16	P200 = 16%
12.5					
15.0				22	
17.5					
20.0	Medium dense, dark gray-brown SAND (SP), trace silt; moist, medium.	189.5 20.5		14	
22.5					
25.0	interbeds of medium dense, gray-brown, silty SAND (SM); moist, fine (1 inch thick) at 25.0 feet			21	
27.5	Exploration completed at a depth of 26.5 feet. Hammer efficiency factor is 85 percent.	183.5 26.5			
30.0					



DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: JCH
COMPLETED: 07/05/16

BORING METHOD: mud rotary (see document text)
BORING BIT DIAMETER: 4 7/8 inches

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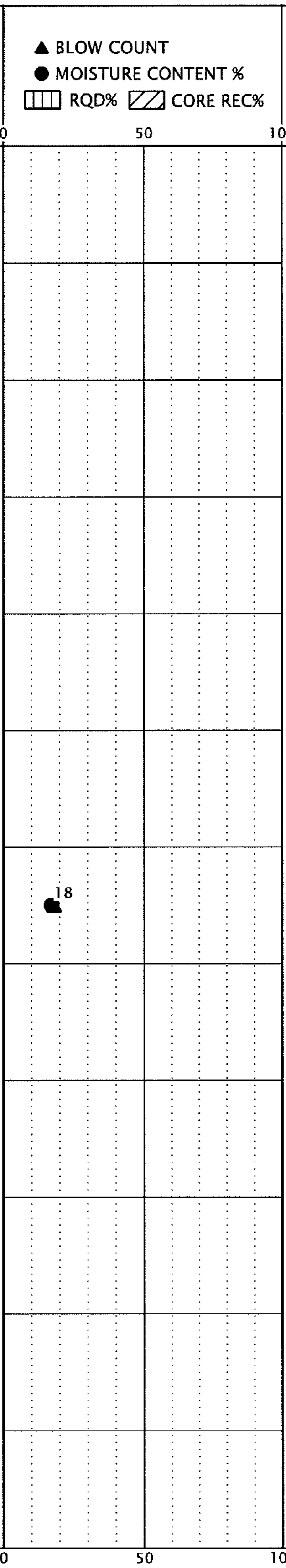
BORING B-3

PROPOSED ALBINA APARTMENTS
PORTLAND, OR

FIGURE A-3

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	INSTALLATION AND COMMENTS
0.0		Boring for infiltration test only. See boring B-2 for lithology.	200.0			
15.5		Medium dense, dark gray-brown SAND with silt (SP-SM); wet, medium.	184.5 15.5	P200		Infiltration test: 4 inches per hour at 15.5 feet. P200 = 11%
17.0		Exploration completed at a depth of 17.0 feet. Hammer efficiency factor is 85 percent.	183.0 17.0			



DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: JCH

COMPLETED: 07/05/16

BORING METHOD: hollow-stem auger (see document text)

BORING BIT DIAMETER: 10 inches

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BORING INF-1

PROPOSED ALBINA APARTMENTS
PORTLAND, OR

FIGURE A-4

BORING LOG SCHRECKA-1-01-B1_3-INF1_2.GPJ GEODESIGN.GDT PRINT DATE: 7/21/16:RC

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC% 0 50 100	INSTALLATION AND COMMENTS
0.0		Boring for infiltration test only. See boring B-2 for lithology.	201.0				
2.5							
5.0							
7.5							
		Medium dense, gray-brown SAND with silt (SP-SM), wet, fine to medium.	192.5 8.5				Infiltration test: 4 inches per hour at 8.5 feet. P200 = 7%
		Medium dense, dark gray-brown SAND with silt (SP-SM); wet, medium to coarse.	192.0 9.0	P200		● ▲ 20	
10.0		Exploration completed at a depth of 10.0 feet. Hammer efficiency factor is 85 percent.	191.0 10.0				
12.5							
15.0							
17.5							
20.0							
22.5							
25.0							
27.5							
30.0						0 50 100	

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: JCH

COMPLETED: 07/05/16

BORING METHOD: hollow-stem auger (see document text)

BORING BIT DIAMETER: 10 inches

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BORING INF-2

PROPOSED ALBINA APARTMENTS
PORTLAND, OR

FIGURE A-5

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-1	3.0	201.0	13							
B-1	5.0	199.0	17							
B-1	7.5	196.5	19				12			
B-1	10.0	194.0	23							
B-2	2.5	202.5	26							
B-2	5.0	200.0	27							
B-2	7.5	197.5	24							
B-2	10.0	195.0	23				14			
B-2	30.0	175.0	20				13			
B-3	2.5	212.5	24							
B-3	5.0	210.0	24							
B-3	7.5	207.5	21							
B-3	10.0	205.0	20				16			
INF-1	15.5	189.5	17				11			
INF-2	8.5	197.5	15				7			

LAB SUMMARY SCHRECKA-1-01-B1_3-INF1_2.GPJ GEODESIGN.GDT PRINT DATE: 7/12/16:KT

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SUMMARY OF LABORATORY DATA

PROPOSED ALBINA APARTMENTS
PORTLAND, OR

FIGURE A-6

WSSC-7-01 - TEST BORING B-7 25FT
OP: WMN

TRACK RIG NO. 2
Date: 30-May-2015

AR: 1.41 in²
LE: 29.25 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft³
EM: 30,000 ksi
JC: 0.00 ▯

ETR: Energy Transfer Ratio
EMX: Max Transferred Energy
CSB: Compression Stress at Bottom
BPM: Blows per Minute
FFS: Force Full Scale

DMX: Maximum Displacement
SFR: Skin friction w/ damping correction
MEX: Maximum Strain
VMX: Maximum Velocity

BL#	depth ft	BLC bl/ft	ETR (%)	EMX k-ft	CSB ksi	BPM bpm	FFS kips	DMX in	SFR kips	MEX µE	VMX f/s
10	25.16	6	81.3	0.3	0.0	48.5	60	1.09	0	957	13.6
12	25.48	6	87.0	0.3	0.0	49.1	60	0.82	0	973	14.4
14	25.81	6	83.8	0.3	0.0	49.0	60	0.67	0	915	14.0
16	26.13	6	86.2	0.3	0.0	48.7	60	0.57	0	986	14.4
18	26.45	6	83.8	0.3	0.0	48.7	60	0.53	0	974	14.1
20	26.77	6	85.7	0.3	0.0	48.7	60	0.56	0	950	14.3
22	27.10	6	85.0	0.3	0.0	48.8	60	0.57	0	942	14.2
24	27.42	6	86.7	0.3	0.0	48.3	60	0.61	0	985	14.5
26	27.74	6	84.7	0.3	0.0	48.6	60	0.57	0	968	14.4
28	28.06	6	86.4	0.3	0.0	48.2	60	0.82	0	941	14.5
30	28.39	6	84.0	0.3	0.0	48.3	60	0.56	0	939	14.0
40	30.00	6	85.1	0.3	0.0	48.5	60	1.03	0	900	14.8
42	30.27	7	81.9	0.3	0.0	48.8	60	0.59	0	893	13.8
44	30.54	7	82.0	0.3	0.0	48.8	60	0.97	0	912	13.8
46	30.81	7	85.4	0.3	0.0	48.5	60	0.59	0	936	14.3
48	31.08	7	81.7	0.3	0.0	48.6	60	0.49	0	873	13.7
50	31.35	7	85.7	0.3	0.0	48.3	60	0.55	0	920	14.0
52	31.62	7	84.4	0.3	0.0	48.3	60	0.56	0	928	14.0
54	31.89	7	84.0	0.3	0.0	48.4	60	0.48	0	865	13.9
56	32.16	7	88.3	0.3	0.0	48.4	60	0.89	0	914	14.5
58	32.43	7	82.2	0.3	0.0	48.5	60	0.38	0	937	13.9
60	32.70	7	84.1	0.3	0.0	48.7	60	1.12	0	858	13.5
62	32.97	7	86.7	0.3	0.0	48.4	60	0.93	0	883	14.1
64	33.24	7	83.0	0.3	0.0	48.6	60	0.95	0	929	13.9
66	33.51	7	81.1	0.3	0.0	48.3	60	0.36	0	911	13.7
82	35.67	7	84.7	0.3	0.0	48.7	60	0.66	0	809	16.1
84	35.83	13	82.9	0.3	0.0	48.7	60	0.53	0	780	15.2
86	35.98	13	84.6	0.3	0.0	48.8	60	0.67	0	796	15.6
88	36.14	13	84.7	0.3	0.0	48.5	60	0.75	0	790	15.8
90	36.30	13	83.8	0.3	0.0	48.2	60	0.55	0	794	16.1
92	36.46	13	85.8	0.3	0.0	48.5	60	0.43	0	867	17.1
94	36.61	13	87.3	0.3	0.0	48.6	60	1.18	0	858	17.0
96	36.77	13	82.1	0.3	0.0	48.5	60	0.38	0	803	15.5
98	36.93	13	83.0	0.3	0.0	48.5	60	0.67	0	782	15.6
100	37.09	13	83.9	0.3	0.0	48.7	60	0.37	0	861	16.8
102	37.24	13	85.3	0.3	0.0	48.5	60	0.37	0	882	17.1
104	37.40	13	83.7	0.3	0.0	48.4	60	0.37	0	879	16.7
106	37.56	13	84.7	0.3	0.0	48.3	60	0.37	0	848	17.4
108	37.72	13	84.8	0.3	0.0	48.3	60	0.38	0	855	16.8
110	37.87	13	84.2	0.3	0.0	48.4	60	0.37	0	869	16.6
112	38.03	13	86.9	0.3	0.0	48.3	60	0.45	0	883	17.5
114	38.19	13	86.1	0.3	0.0	48.5	60	0.44	0	869	17.3
116	38.35	13	84.3	0.3	0.0	48.4	60	0.83	0	858	16.2
118	38.50	13	84.8	0.3	0.0	48.3	60	0.38	0	860	16.4
120	38.66	13	85.2	0.3	0.0	48.3	60	0.68	0	839	16.1

WSSC-7-01 - TEST BORING B-7 25FT
OP: WMN

TRACK RIG NO. 2
Date: 30-May-2015

BL#	depth	BLC	ETR	EMX	CSB	BPM	FFS	DMX	SFR	MEX	VMX
	ft	bl/ft	(%)	k-ft	ksi	bpm	kips	in	kips	µE	f/s
122	38.82	13	84.2	0.3	0.0	48.4	60	0.37	0	872	16.7
124	38.98	13	84.7	0.3	0.0	48.4	60	0.75	0	836	16.6
126	39.13	13	83.4	0.3	0.0	48.3	60	0.48	0	834	16.0
137	40.00	13	84.9	0.3	0.0	50.8	60	0.56	0	925	14.9
139	40.16	13	85.4	0.3	0.0	50.3	60	0.54	0	917	14.7
141	40.31	13	84.1	0.3	0.0	50.4	60	0.50	0	914	14.3
143	40.47	13	87.5	0.3	0.0	50.3	60	0.83	0	933	14.6
145	40.63	13	86.8	0.3	0.0	50.6	60	0.85	0	929	14.1
147	40.79	13	86.4	0.3	0.0	50.6	60	0.66	0	948	14.6
149	40.94	13	84.2	0.3	0.0	50.7	60	0.44	0	929	14.4
151	41.10	13	85.2	0.3	0.0	50.5	60	0.45	0	933	14.0
153	41.26	13	85.8	0.3	0.0	50.5	60	0.56	0	924	14.4
155	41.42	13	86.6	0.3	0.0	50.4	60	0.63	0	936	14.5
157	41.57	13	85.7	0.3	0.0	50.8	60	0.55	0	926	14.7
159	41.73	13	86.8	0.3	0.0	50.6	60	0.51	0	930	14.4
161	41.89	13	85.6	0.3	0.0	50.5	60	0.55	0	899	13.7
163	42.05	13	87.3	0.3	0.0	50.5	60	0.92	0	918	13.8
165	42.20	13	85.5	0.3	0.0	50.1	60	0.86	0	923	13.5
167	42.36	13	85.1	0.3	0.0	50.8	60	0.71	0	922	13.8
		Average	85.0	0.3	0.0	49.0	60	0.64	0	892	15.0
		Std. Dev.	1.8	0.0	0.0	0.9	0	0.25	0	53	1.2

Total number of blows analyzed: 128

BL#	Sensors
9-167	F3: [SPT B1] 217.8 (1.00); F4: [SPT B2] 218.9 (1.00); A3: [K0232] 290.0 (1.00); A4: [K0231] 325.0 (1.00)

BL#	Comments
31	N: 7,9,14
40	LE = 34.20 ft; WC = 16,765.8 f/s
67	N: 7,10,18
82	LE = 39.42 ft; WC = 16,764.7 f/s
127	N: 13, 20, 26
137	LE = 44.10 ft; WC = 16,774.5 f/s
167	N: 8,15,16

Time Summary

Drive	27 seconds	5:31 PM - 5:32 PM (5/30/2015) BN 9 - 31
Stop	14 minutes 52 seconds	5:32 PM - 5:47 PM
Drive	33 seconds	5:47 PM - 5:47 PM BN 40 - 67
Stop	19 minutes 59 seconds	5:47 PM - 6:07 PM
Drive	55 seconds	6:07 PM - 6:08 PM BN 82 - 127
Stop	16 minutes 13 seconds	6:08 PM - 6:24 PM
Drive	35 seconds	6:24 PM - 6:25 PM BN 137 - 167

Total time [00:53:37] = (Driving [00:02:31] + Stop [00:51:06])

ACRONYMS AND ABBREVIATIONS

ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AC	asphalt concrete
ACP	asphalt concrete pavement
ASTM	American Society for Testing and Materials
BGS	below ground surface
g	gravitational acceleration (32.2 feet/second^2)
H:V	horizontal to vertical
IBC	International Building Code
ksf	kips per square foot
MCE	maximum considered earthquake
OSHA	Occupational Safety and Health Administration
OSSC	Oregon Standard Specifications for Construction (2015)
pcf	pounds per cubic foot
PG	performance grade
PGA	peak ground acceleration
psf	pounds per square foot
psi	pounds per square inch
SOSSC	State of Oregon Structural Specialty Code
SPT	standard penetration test

Stormwater Management Facilities

Private Stormwater Report

Albina Apartments

HDG Job #: EMA018

Prepared For:
ABN Development Company,
LLC
P.O. Box 13607
Portland, OR 97213

Prepared By:
Humber
Design
Group, Inc.

117 SE Taylor St. Suite 001
Portland, OR 97214
(P) 503 946 6690

'I hereby certify that this Stormwater Management Report for the Albina Apartments project has been prepared by me or under my supervision and meets minimum standards of The City of Portland and normal standards of engineering practice.

I hereby acknowledge and agree that the jurisdiction does not and will not assume liability for the sufficiency, suitability, or performance of drainage facilities designed by me.'

REGISTERED PROFESSIONAL ENGINEER
76,071
OREGON
JULY 12, 2005
MARTHA S. WILLIAMS
EXPIRES 6-30-2018

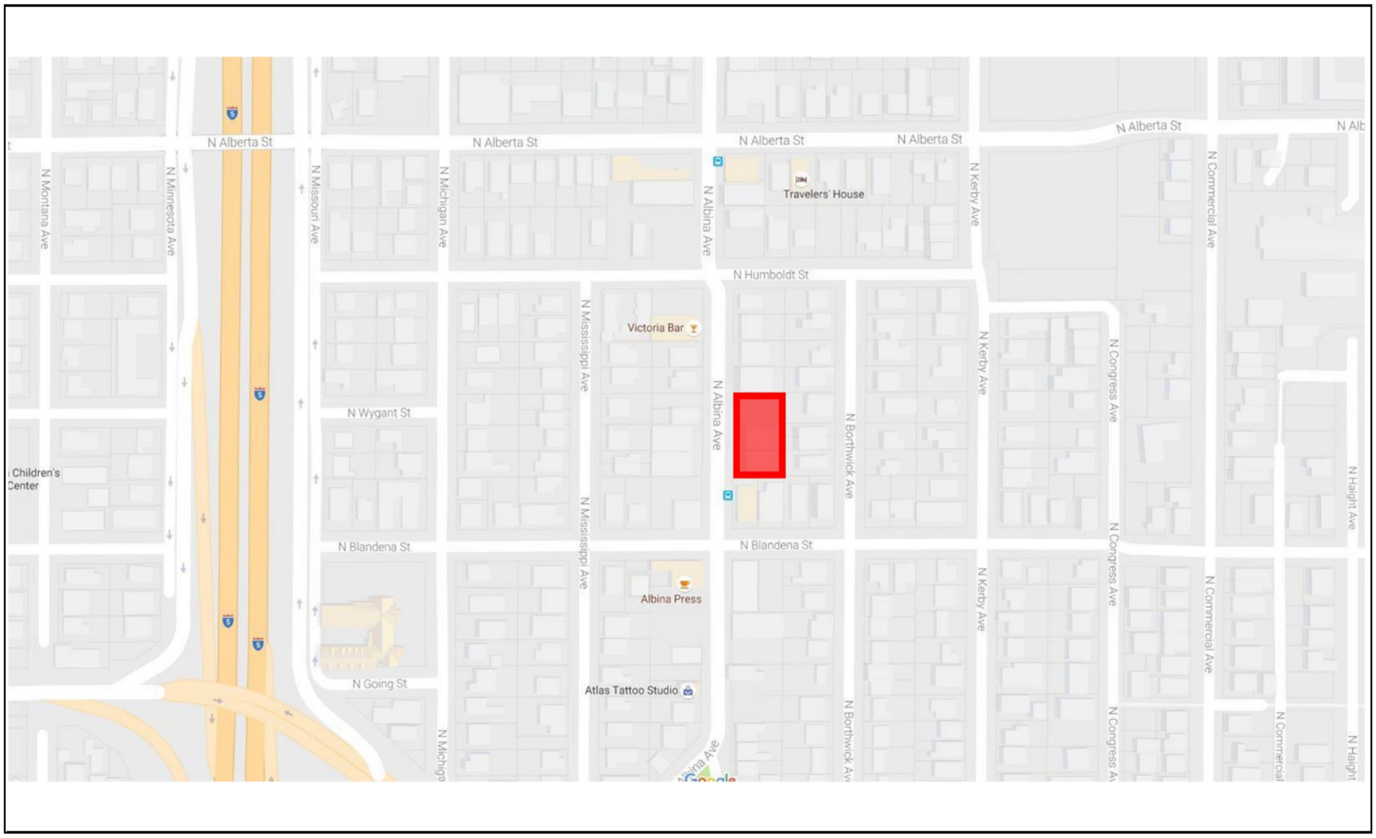
Date: January 23, 2017

Table of Contents

Project Overview and Description

Location of Project	4732 N Albina Ave.
Site Area/Acreage	15,732 sf
Nearest Cross Street	N Blandena St and N Albina Ave
Property Zoning	EX- Central Employment (Design Zone Overlay)
Existing Conditions	The existing site is currently occupied by two single-story structures, paved parking area, and grass covered lot. The existing building will be demolished and the entire site cleared.
Proposed Development	The proposed development includes the construction of a 4-story apartment complex with at-grade open air parking. The new building and parking area will cover the entire site.
Watershed Description	Willamette River
Subwatershed	Beech-Essex-Wheeler
R#	R135716, R135627, R135626
Tax Map	1N 1E 22BD
Tax Lot	10400, 10500, 10600
Permits Required	Building Permit Public Street Permit DEQ UIC Permit

Vicinity Map



🟥 Site Location

Methodology

Existing Drainage	Runoff from the existing site is conveyed to the 10" combined sewer in N Albina Avenue.
Infiltration Results	Encased falling head infiltration testing was completed by GeoDesign, Inc. at 15.5 and 8.5 feet below ground surface. Infiltration rates for the site were measured at 4 inches/hour at both depths.
PRIVATE Proposed Stormwater Management Techniques	Stormwater runoff from the private site will be managed with a private drywell system. The site includes 15,732 SF of new imperious area which will be collected and piped to the the private drywell system located in parking area.
PUBLIC Proposed Stormwater Management Techniques	The existing curb alignment along N Albina Avenue public frontage will remain unchanged, therefore stormwater management is not required for the public right-of-way.
Discharge Point **Receiving Body**	Drywell or Soakage Trench (UIC)
Stormwater Hierarchy Justification	On-site inflitration with a surface infiltration facility (Category 1) is not feasible due to the proposed building and parking area covering the entire site. This project will fall under Category 2, on-site infiltration with a private drywell or soakage trench.

Analysis

Computational Method Used HydroCAD models of a SBUH Type 1A Storm were used to calculate the stormwater management facility sizes for the catchment areas. See attached calculations. Below is a summary of the results.

Hydrologic Soil Group B

Hydrologic Soil Sand with Silt

Table 1 – Curve Numbers

Predeveloped Pervious CN	79
Predeveloped Impervious CN	98
Post-Developed Pervious CN	79
Post-Developed Impervious CN	98

Table 2 – Design Storms

WQ Storm	0.83 inches
2-year	2.40 inches
10-year	3.40 inches
25-year	3.90 inches
100-year	4.40 inches

Table 3 – Time of Concentration

Predeveloped TOC	5 min
Post-Developed TOC	5 min

Table 3– Catchment Areas and Facility Table

Catchment/ Facility ID	Source (roof, road, etc.)	Treatment Area (sf)	Ownership (private/ public)	Facility Type/ Function	Facility Size
A	Roof/Parking	15,732	Private	Drywell	4 x 48" dia. X 15' deep

Engineering Conclusions

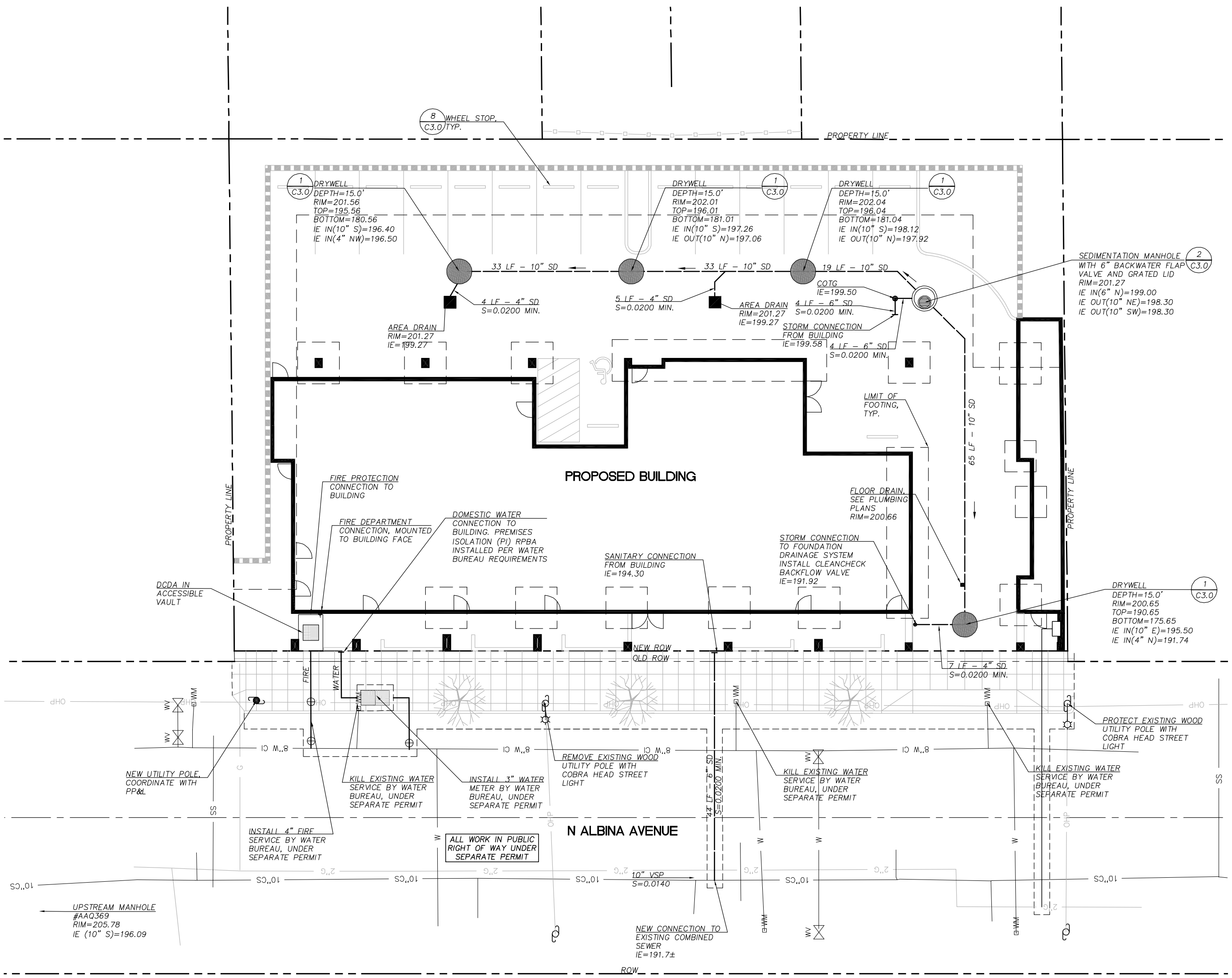
The preceding methodologies and calculations presented indicate compliance with the current jurisdictional stormwater management codes and requirements. A summarized breakdown is presented below:

Water Quality	The proposed development will meet the provisions for water quality per the 2016 Portland Stormwater Management Manual.
Water Quantity	The proposed development will meet the provisions for water quantity per the 2016 Portland Stormwater Management Manual.
Downstream / Upstream Impacts	There are no upstream or downstream impacts created by this proposed development.
100 year storm	The private drywell system will infiltrate the 10-year storm event and the 100-year storm event will be safely conveyed away from structures by overland flow to the public right-of-way.

Appendix A

Stormwater Facility Details / Exhibits

- Site Utility Plan
- Catchment Map
- Private Drywell Detail
- Sedimentation Manhole Detail



UTILITY PLAN

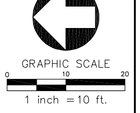
SCALE: 1"=10'

SHEET NOTES

1. ALL DOMESTIC WATER AND FIRE PROTECTION WORK IN THE PUBLIC RIGHT OF WAY BY PORTLAND WATER BUREAU AT OWNER'S EXPENSE. CONTRACTOR TO COORDINATE WORK WITH PORTLAND WATER BUREAU.
2. INSTALL 4" PERFORATED FOUNDATION DRAINAGE PIPE AROUND PERIMETER OF BUILDING INSTALLED PER DETAIL 3/C3.0. CONNECT PERFORATED PIPE TO SOLID PIPE WITH CLEANCHECK BACKFLOW PREVENTOR.
3. RETAINING WALL WITH GUARDRAIL BY STRUCTURAL.

SHEET LEGEND

●	DRYWELL	1 / C3.0
◎	SEDIMENTATION MANHOLE WITH GRATED LID	2 / C3.0
■	AREA DRAIN	7 / C3.0
▪	FLOOR DRAIN	
•	COTG	2 / C3.1



NOTE: INFORMATION IN THESE DOCUMENTS IS NOT APPROVED FOR CONSTRUCTION UNTIL A BUILDING PERMIT HAS BEEN ISSUED.

PERMIT

EMERICK ARCHITECTS
321 SOUTHWEST FOURTH AVE SUITE 200 PORTLAND OR 97204
P 503.235.9400 W EMERICK-ARCHITECTS.COM

Humber Design Group, Inc.
Portland, OR • 503.946.6690 • hdgpdx.com

REGISTERED PROFESSIONAL ENGINEER
76,071
OREGON
JULY 12, 2005
MARTHA S. WILLIAMS
EXPIRES 6-30-2018

REV. #: DATE: DESCRIPTION

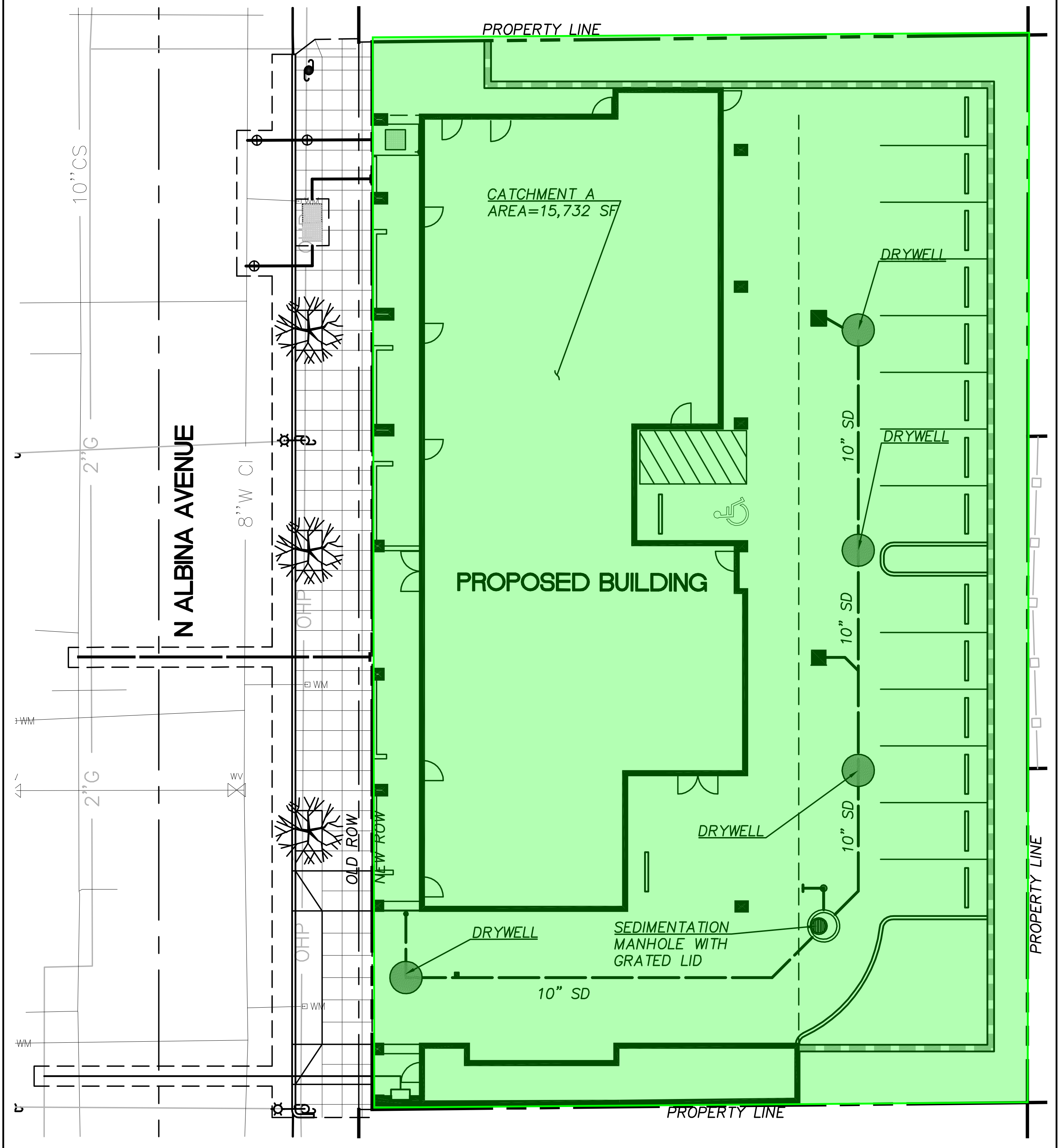
ALBINA APARTMENTS

4734 N. ALBINA AVE.
PORTLAND, OR 97217

UTILITY PLAN

PROJECT NO: 1531
DRWN BY: PLM CHK'D BY: MSW
DATE: 01.30.17

C2.0



ALBINA APARTMENTS

CATCHMENT MAP

GRAPHIC SCALE

0 20 40

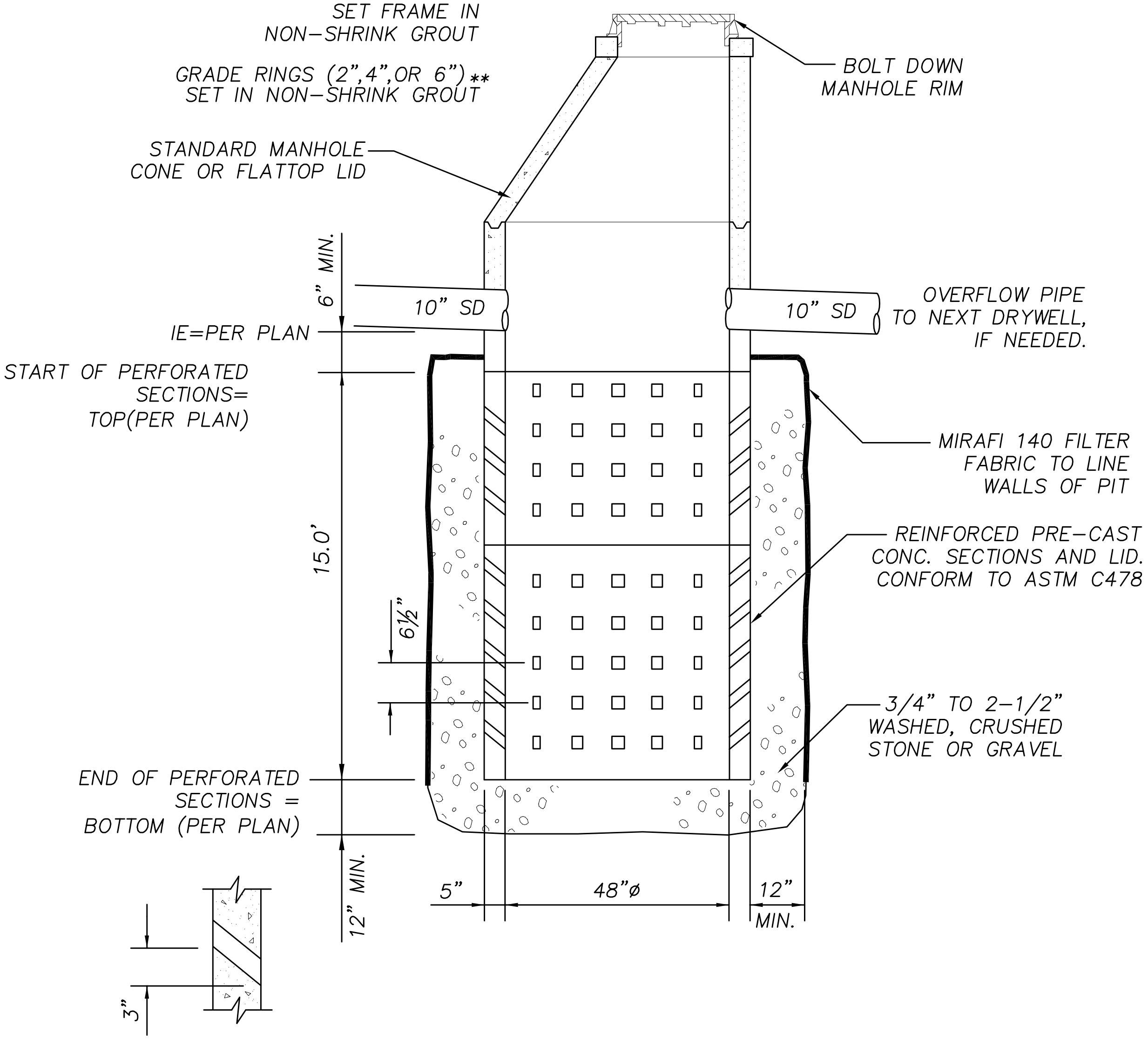
1 inch = 20 ft.

PROJECT NO.:	EMA018
DRAWN BY:	PLM
DESIGN BY:	PLM
REVIEWED BY:	MSW
DATE:	01/23/2017

Humber Design Group, Inc.

Portland, OR ▪ 503.946.6690 ▪ hdgpdx.com

P:\EMA018 – Albina Apartments\7 – CAD Drawings\B – Exhibits\Catchment Map.dwg Mar 31, 2017 – 8:46am

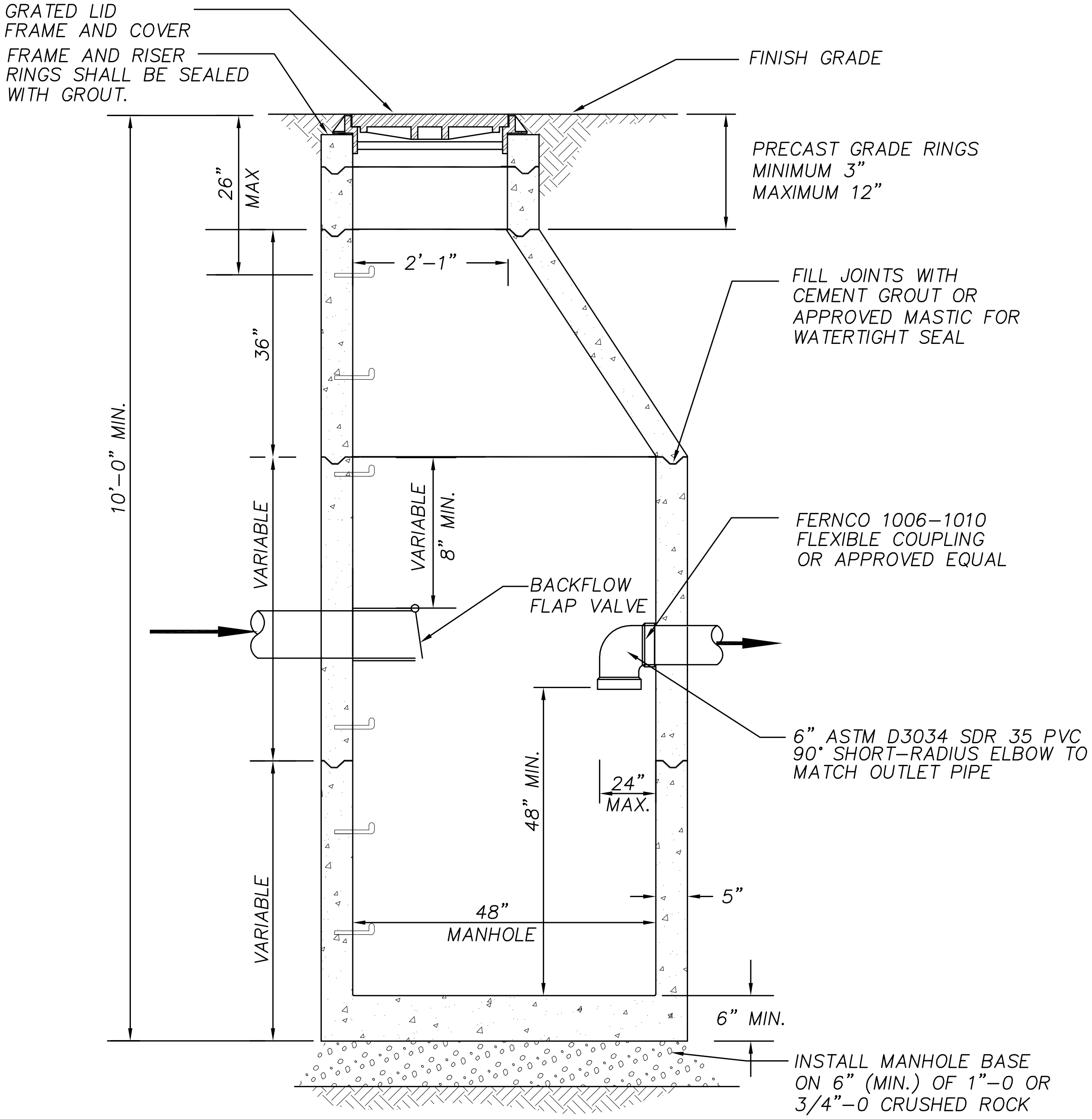


1 **DRYWELL**

NTS

DRYWELL TESTING NOTES

1. DRYWELL SYSTEM SHALL HAVE THE CAPACITY TO DISPOSE OF STORMWATER AT THE COMBINED RATE OF XXXGPM.
2. SHOULD DRYWELL(S) FAIL TO TEST AT THE MINIMUM DESIGN RATE ADDITIONAL DRYWELLS MAY BE CONSTRUCTED, AS APPROVED, IN ORDER TO ACHIEVE THE REQUIRED MINIMUM DISPOSAL RATE. ALTERNATIVE DRYWELL LOCATIONS AND/OR ADDITIONAL DRYWELLS IN PIPED SERIES MUST BE PRE–APPROVED AND IDENTIFIED ON THE CONSTRUCTION DRAWINGS.
3. INDICATED DRYWELL(S) (AS NOTED ABOVE IN NOTE#1) SHALL BE TESTED BY THE CONTRACTOR, AS DIRECTED AND APPROVED BY THE ENGINEER.
4. DRYWELL(S) SHALL BE TESTED AFTER CONSTRUCTION OF THE DRYWELL STRUCTURE (INCLUDING DRAIN ROCK AND PERIMETER BACKFILL) BUT PRIOR TO THE CONSTRUCTION OF THE TOP SLAB AND FINISH BACKFILL.
5. NOTIFY CIVIL AND GEOTECHNICAL ENGINEERS 5 DAYS PRIOR TO TESTING.
6. CONTRACTOR SHALL CONTACT CITY OF PORTLAND WATER BUREAU OR APPLICABLE WATER DISTRICT TO ARRANGE FOR DRYWELL TEST WATER SUPPLY. CONTRACTOR SHALL BEAR RESPONSIBILITY FOR SECURING ALL NECESSARY PERMITS, AUTHORIZATION AND ANY FEES.
7. CONTRACTOR SHALL ARRANGE FOR THE PROVISION OF ALL DRYWELL TESTING EQUIPMENT, INCLUDING BUT NOT LIMITED TO FLOW METER, PIPING, AND TRAFFIC CONTROL.
8. CLEAN WATER SHALL BE PROVIDED TO TEST DRYWELLS, AS APPROVED, INTRODUCTION OF SEDIMENT MAY RESULT IN FAILURE OF THE DRYWELL CAPACITY TEST.
9. PROVIDE CIVIL ENGINEER WITH RECORDED TEST DATA.



NOTE:

1. ALL PRECAST SECTIONS SHALL CONFORM TO THE REQUIREMENTS OF ASTM C478. SEE CITY OF PORTLAND STANDARDS AND SPECIFICATIONS FOR MH STEP DETAIL.
2. ALL CONNECTING PIPE SHALL HAVE A FLEXIBLE JOINT WITHIN 18" OF MANHOLE WALL.

2 SEDIMENTATION MANHOLE

NTS

Appendix B

Support Calculations

HydroCAD Calculations

Summary for Pond 6P: Drywells

Inflow Area = 15,732 sf,100.00% Impervious, Inflow Depth = 3.17" for 10yr event
Inflow = 0.29 cfs @ 7.90 hrs, Volume= 4,152 cf
Outflow = 0.09 cfs @ 9.03 hrs, Volume= 4,152 cf, Atten= 69%, Lag= 68.0 min
Discarded = 0.09 cfs @ 9.03 hrs, Volume= 4,152 cf

Routing by Stor-Ind method, Time Span= 0.00-32.00 hrs, dt= 0.05 hrs
Peak Elev= 112.60' @ 9.03 hrs Surf.Area= 154 sf Storage= 1,026 cf

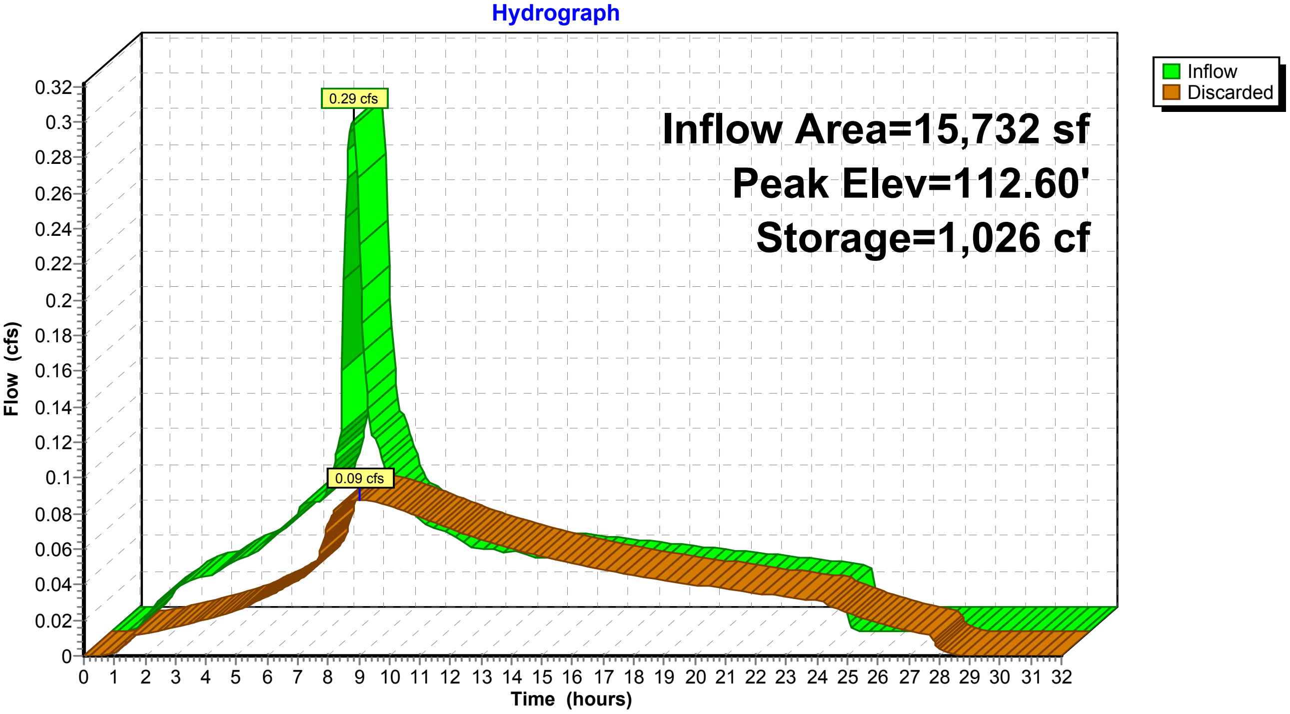
Plug-Flow detention time= 166.8 min calculated for 4,145 cf (100% of inflow)
Center-of-Mass det. time= 166.9 min (831.7 - 664.8)

Volume	Invert	Avail.Storage	Storage Description
#1	100.00'	754 cf	**4.00'D x 15.00'H Vertical Cone/Cylinder**x 4 Inside #2
#2	100.00'	467 cf	**7.00'D x 15.00'H Vertical Cone/Cylinder**x 4 2,309 cf Overall - 754 cf Embedded = 1,555 cf x 30.0% Voids
		1,221 cf	Total Available Storage

Device	Routing	Invert	Outlet Devices
#1	Discarded	100.00'	**3.000 in/hr Exfiltration over Wetted area**

Discarded OutFlow Max=0.09 cfs @ 9.03 hrs HW=112.60' (Free Discharge)
↑—**1=Exfiltration** (Exfiltration Controls 0.09 cfs)

Pond 6P: Drywells



Appendix C

Operations and Maintenance Plan

CITY OF PORTLAND
Stormwater Management Manual

FORM 2

OPERATIONS & MAINTENANCE FORM

PRIVATE STORMWATER MANAGEMENT FACILITIES

☐ This O&M Form supercedes document number ______

(for official county use only)

PROJECT NAME Albina Apartments

PERMIT INFORMATION

Permit # ______

Permit Submittal Date ______

SITE INFORMATION *(include all parcels)*

R# (6 Digits) 135626, 135627, 135716

Site Address 4734 N Albina Avenue

City / State / Zip Portland, OR 97217

Preparation Date: 01/23/2017

OWNER INFORMATION (ALL LEGAL OWNERS)

Name (1) ABN Development Company, LLC

Name (2) Andrew Schreck

Address *(Mailing)* P.O. Box 13607

City / State / Zip Portland, OR 97213

O&M PREPARER INFORMATION

Name Paige Miller, EI

Address *(Mailing)* 117 SE Taylor St #001

City / State / Zip Portland, OR 97214

Phone *(area code required)* 503-946-6690

Email paige.miller@hdgpdx.com

Site Legal Description:

CLIFFORD ADD, BLOCK 11, LOT 3 & 4 & K

Responsible Party for Maintenance *(check one)*

☐ Homeowners Association
☒ Property Owner
☐ Property Management Company
☐ Tenant
☐ Other (describe) ______
(not Contractor or Consultant)

Contact Information for Responsible Party

Contact Name Andrew Schreck

Contact Organization ABN Development Company, LLC

Phone *(area code required)* 503-568-3552

Email: schreck.andy@gmail.com

Maintenance Practices and Schedule

These operation and maintenance practices are required in accordance with Portland City Code, Chapter 17.38.

The requirements are based on the current version of the *City of Portland Stormwater Management Manual* on the date of permit submittal.

For the **Simplified Approach,** please attach the current O&M Specifications for each facility type from the *Stormwater Management Manual*, Chapter 3.3.1.

For the **Presumptive and Performance Approaches,** please attach the approved, site specific O&M Plan per the *Stormwater Management Manual,* Chapter 3.3.2.

OPERATIONS & MAINTENANCE FORM

PRIVATE STORMWATER MANAGEMENT FACILITIES

SITE PLAN

Provide a site plan sketch in the area provided below, or attach a scaled site plan to this submittal that includes all of the information required as shown in Appendix D6 on page D.6-1, in Operations & Maintenance Form Instructions, Site Plan.

STEP 1 – COMPLETE THE FOLLOWING TABLE

Stormwater Facility Type (Chapter 2)	Stormwater Facility Size (sf)	Drainage is from Roof or Lot?	Impervious Area Treated (sf)	Discharge Point
Drywell	(4) - 48" dia. x 15'	Roof/Parking	15,732	Infiltration
Totals				

Maintaining the stormwater management facility or facilities listed above shown on the following (or attached) site plan is a required condition of building permit approval for the identified property. Property owners are required to operate and maintain facilities in accordance with the O&M plan on file with the City of Portland. This requirement is binding on all current and future owners of the property. Failure to comply with the O&M plan can trigger an enforcement action, including penalties. The O&M plan may be modified by written consent of current owners and written approval of the Bureau of Environmental Services.

STEP 2 – REQUIRED SITE PLAN

(insert or draw here, or attach separate sheet)

■ I Have Attached a Site Plan

OPERATIONS & MAINTENANCE FORM

PRIVATE STORMWATER MANAGEMENT FACILITIES

SIGNATURE AND ACKNOWLEDGEMENT

By signing below, the owner accepts and agrees to the terms and conditions contained in this O&M Form and in any document executed by filer and recorded with it. The owner further acknowledges that this documentation has been prepared on their behalf and that they are responsible for the quality and completeness of the O&M Plan. Any failure to comply with the terms of these plans may result in enforcement actions by BES requiring the property owner to restore the stormwater facilities to a functional state as approved under original requirements.

The owner also accepts that the City requires property owners to submit and record, with the County, complete and accurate O&Ms enforceable under City Code 17.38 and that substantial changes to the O&M require City approval prior to County recording.
A revised O&M must state that it supersedes a previous O&M (with cited county document number; See Page 1).

THIS PAGE MUST BE SIGNED IN THE PRESENCE OF A NOTARY.

______________________________	______________________________
Property Owner or Authorized Representative (1) Signature	*Property Owner or Authorized Representative (2) Signature*

NOTARY SIGNATURE AND STAMP

☐ INDIVIDUAL Acknowledgement	**OR**	☐ CORPORATE Acknowledgement
This acknowledgement is intended for property owned by individuals or trusts.		*This acknowledgement is intended for corporation, government agencies, school districts, or other formal entities*
STATE of OREGON county of: ____________		STATE of OREGON county of: ____________
This instrument was acknowledged before me on: *(date)* ____________		This instrument was acknowledged before me on: *(date)* ____________
By: *(owner 1)* ____________		By: *(representative)* ____________
By: *(owner 2)* ____________		As: *(Title)* ____________
Notary Signature ____________		Of: *(Corporation)* ____________
My Commission Expires ____________		Notary Signature ____________
Notary Seal:		My Commission Expires ____________
		Notary Seal:

Stormwater Management Facilities

Private Operations & Maintenance Plan

Albina Apartments

Prepared By:

Humber Design Group, Inc.

117 SE Taylor St. Suite 001
Portland, OR 97214
(P) 503 946 6690

Date: January 23, 2017

Site O&M Responsible Party

This facility is to be maintained by property owner ABN Development Company, LLC. ABN Development Company, LLC contact is Andrew Schreck, Manager, 503-568-3552.

Onsite Stormwater System Description

All stormwater runoff generated on-site is managed with a Drywell and Sedimentation Manhole system. The Sedimentation Manhole is a large manhole that allows pollutants to settle out as stormwater collects in the large sump then flows out of an elbowed pipe to the Drywell. The Drywell is a large perforated manhole where stormwater infiltrates through washed, crushed stone or gravel wrapped in filter fabric.

Table 1 - Facility Description Table

Facility Name	Type	Facility Size (sf)	Source	Impervious Area Managed (sf)	Discharge Point
Drywell	Drywell	(4) 48" dia. X 15' deep	Roof/ Parking	15,732	Infiltration

Inspection & Maintenance Schedule

All stormwater facilities must be inspected at least:

First two years:	Quarterly
Thereafter:	Twice a year
After major rainfall events:	Within 48 hours of major rainfall events (more than 1 inch of rain over a 24-hour period)

Inspection & Stormwater Maintenance Prodecures

The following items shall be inspected and maintained as stated.

Overflow Drains, Area Drains, and Piped Storm System	Sediment shall be removed biannually. Debris shall be removed from inlets and outlets quarterly. Quarterly inspection for clogging shall be performed. Grates shall be tamper-proof. Repair/seal cracks. Replace when repair is insufficient.
Drywell and Soakage Trenches	Clean gutters, rain drains, and silt traps twice a year Repair/seal cracks. Replace when repair is insufficient. Prevent large root systems from damagin subsurfaced structural components. Remove sediment and debris from all accessible components to prevent ponding. Ponding/lack of infiltration may require decommissioning and replacement. Consult with the City prior to subgrade work.
Vectors	Stormwater facilities shall not harbor mosquito larvae or rats that pose a threat to public health or that undermine the facility structure. Monitor standing water for small wiggling sticks perpandicular to the water's surface. Note holes/burrows in and around facilities. Call Multnomah County Vector Control at 503-988-3464 for immediate assistance to eradicate fectors. Record the time/date, weatehr, and site conditions when vector activity is observed.
Best Management Practices (BMPs)	BMPs prevent pollutants from mixing with stormwater. Typical nonstructural control measures include raking and removing leaves, street sweeping, vacuum sweeping, and limited and controlled application of pesticides, herbicides, and fertilizers.
Spill Prevention	Spill prevention measures shall be exercised when handling substances that can contaminate sormwater. Virtually all sites, including residential and commercial, present dangers from spills. It is important to exercise caution when handling substances that can contaminate stormwater. Activities that pose the chance of hazardous material spills shall not take place near collection facilities. ● The proper authority and the property owner shall be contacted immediately if a spill is observed. ● A spill kit shall be kept near spill-prone operations and refreshed annually. ● Employees shall be trained on spill control measures. ● Shut-off valves shall be tested quarterly. ● Releases of pollutants shall be corrected within 12 hours.
Access	Access shall be maintained for all facilities so O&M can be performed as regularly scheduled.

Inspection & Maintenance Logs

The facility owner shall keep a log to record all inspection and maintenance activities (see Sample Log). Record date, description, and contractor (if applicable) for all inspections and any maintenance or repairs performed. Keep work orders and invoices on file and make available upon request of the city inspector.

Inspection Log	Record the date and the personnel who conducted the site inspection. Record the infiltration rate if greater than 48 hours, a description of any and all spills and vector issues, sediment & oil depth, the percentage of vegetation coverage (deseriable and undesirable), and the condition of the system components every quarter for the first 2 years of operation and twice a year after a major storm even thereafter.
Pollution Prevention	All sites shall implement BMPs to prevent hazardous wastes, litter, or excessive oil and sediment from contaminating stormwater. Contact Spill Prevention & Citizen Response at 503-823-7180 for immediate assistance with responding to spills. Record time/date, weather, and site conditions if site activities are found to contaminate stormwater.
Vectors (mosquitoes and rodents)	Stormwater facilities shall not harbor mosquito larvae or rats that pose a threat topublic health or that undermine the facility structure. Monitor standing water for small wiggling sticks perpendicular to the water's surface. Note holes/burrows in and around facilities. Call Multnomah County Vector Control at 503-988-3464 for immediate assistance with eradicating vectors. Record time/date, weather, and site conditions when vector activity is observed.
Depth of Sediment & Oil	Take and record measurement at catch basins, conveyance systems, inlets, outlets and within the facility itself. Compare to capacity thresholds defined in the *Stormwater Management Manual* Section 3.2.4, Summary of Thresholds for Maintenance, or the site-specific O&M
Percent Vegetation Coverage	Record percent cover of desireable, dead, and invasive vegetation.
Condition of Structural Components	Record type and size of missing or broken components (i.e. width of cracks and/or extent of settling.)
Maintenance	Record date, description, and contractor (if applicable) for all structural repairs, landscape maintenance, and facility cleanout activities.

Sample Log Form

Date: __________ Time: __________ Initial: ________

Work performed by: ____________________

Work performed: ____________________

Details: ____________________

Date: __________ Time: __________ Initial: ________

Work performed by: ____________________

Work performed: ____________________

Details: ____________________

Date: __________ Time: __________ Initial: ________

Work performed by: ____________________

Work performed: ____________________

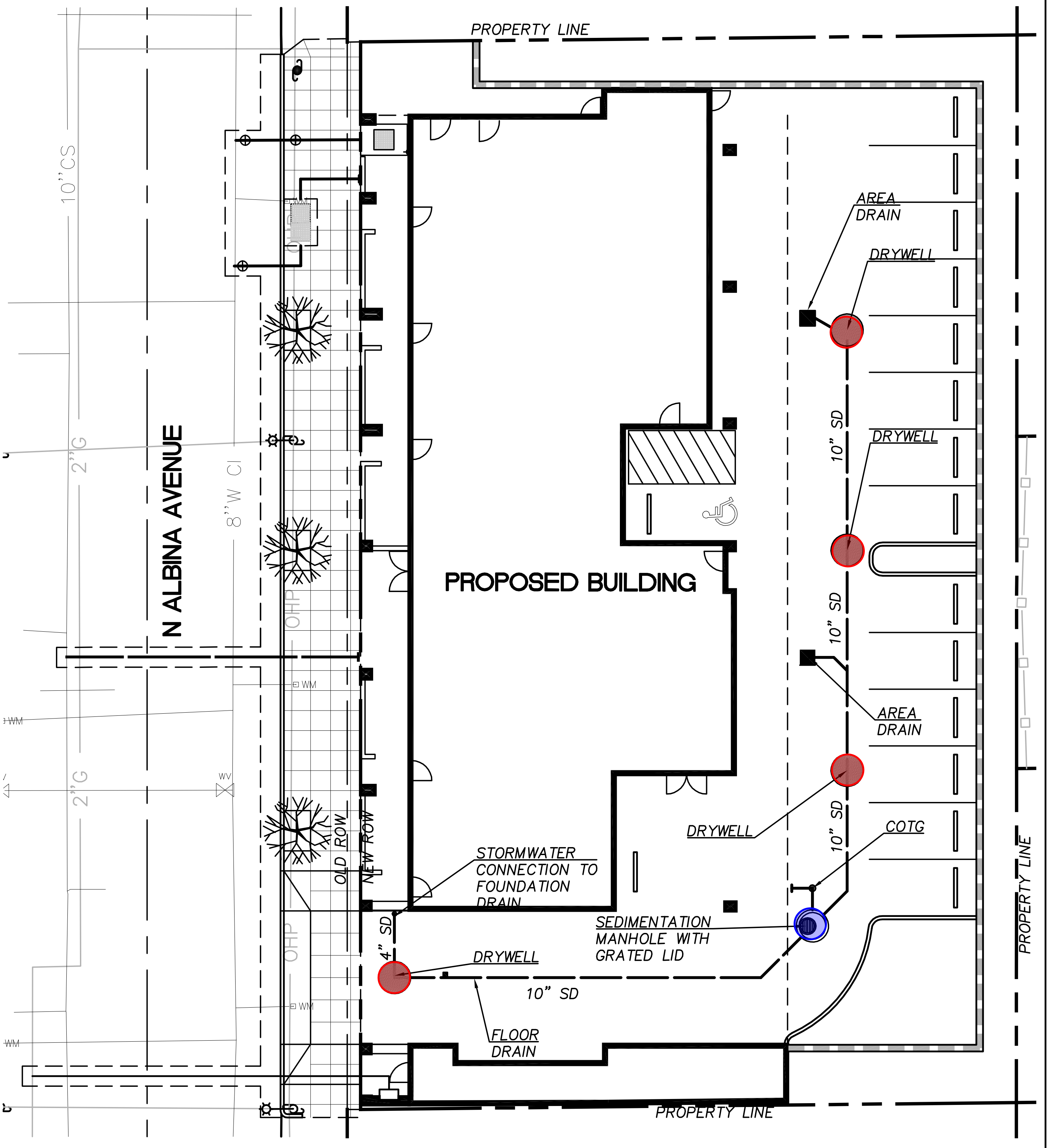
Details: ____________________

Date: __________ Time: __________ Initial: ________

Work performed by: ____________________

Work performed: ____________________

Details: ____________________



ALBINA APARTMENTS

O+M PLAN

PROJECT NO.:	EMA018
DRAWN BY:	PLM
DESIGN BY:	PLM
REVIEWED BY:	MSW
DATE:	XXX

Humber Design Group, Inc.

Portland, OR ▪ 503.946.6690 ▪ hdgpdx.com

P:\EMA018 – Albina Apartments\7 – CAD Drawings\B – Exhibits\O&M Plan.dwg Mar 31, 2017 – 8:46am

Appendix D

Additional Forms & Associated Reports

Report of Geotechnical Engineering Services
(GeoDesign Inc. dated July 22, 2016)

GeoDesign Inc

REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Proposed Albina Apartments
4732 N Albina Avenue
Portland, Oregon

For
Andy Schreck
July 22, 2016

GeoDesign Project: SchreckA-1-01

GEODESIGN INC

July 22, 2016

Andy Schreck
17154 Lowenberg Terrace
Lake Oswego, OR 97034

Report of Geotechnical Engineering Services
Proposed Albina Apartments
4732 N Albina Avenue
Portland, Oregon
GeoDesign Project: SchreckA-1-01

GeoDesign, Inc. is pleased to submit our report of geotechnical engineering services for the proposed Albina Apartments development located at 4732 N Albina Avenue in Portland, Oregon. Our services for this project were conducted in accordance with our proposal dated June 20, 2016.

We appreciate the opportunity to be of service to you. Please call if you have questions regarding this report.

Sincerely,

GeoDesign, Inc.

Brett A. Shipton, P.E., G.E.
Principal Engineer

VCL:BAS:kt
Attachments
One copy submitted (via email only)
Document ID: SchreckA-1-01-072216-geor.docx

EXECUTIVE SUMMARY

The following is a summary of our findings and recommendations for design and construction of the proposed development. We recommend that the main report be referenced for a more thorough description of the subsurface conditions and geotechnical recommendations for the project.

- The proposed structure can be supported on shallow foundations bearing on firm native soil. Approximately 3 to 6 feet of fill was encountered in our explorations. Any fill material encountered at footing subgrade be removed and replaced with structural fill.

- We recommend that in all building slab areas where the cut is less than 3 feet, and fill is present at the subgrade elevation, the surface foot of material should either be removed and replaced with structural fill or the subgrade scarified and compacted as structural fill to a depth of 1 foot.

- If portions of the building will be embedded below ground surface, excavation sidewalls will require shoring if they are adjacent to existing settlement-sensitive structures or to stay within property boundaries.

- The fine-grained soil at the site can be sensitive to small changes in moisture content and difficult to adequately compact during wet weather or when the moisture content of the soil is more than a couple of percent above the optimum required for compaction. If the moisture content of the soil is currently above optimum, drying will be required if used as structural fill.

- The site will require demolition of existing buildings, concrete slabs, and other site features. In particular, wet, sensitive subgrade should be anticipated beneath the pavement areas.

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1.0 INTRODUCTION

GeoDesign, Inc. is pleased to submit this geotechnical engineering report for the proposed Albina Apartments development located at 4732 N Albina Avenue in Portland, Oregon. Figure 1 shows the site relative to existing physical features. Figure 2 shows the current site layout and our approximate exploration locations. Acronyms and abbreviations used herein are defined at the end of this document.

The property is currently occupied by two single-story warehouse structures; a paved parking area; and a fenced, grass-covered area on the north portion of the site. We understand that the existing structures will be demolished.

Plans are preliminary at the time of this report. Based on our review of a conceptual site plan, the development will consist of a new four-story apartment building on the west portion of the property along N Albina Avenue, with surface parking at the rear (east) of the property. A basement is not planned at this time. However, based on existing site grades, portions of the building (north and south) would possibly be embedded below the ground surface. Structural loads were not available at the time of this report. We have assumed that the column loads will be between 300 and 400 kips and wall loads will be less than 4 kips per foot. Floor slab loads are assumed to be less than 150 psf. Finish floor grades are not known; however, based on the existing topography, cuts of up to approximately 12 feet could be required to develop the site.

2.0 SCOPE OF SERVICES

The purpose of our geotechnical engineering services was to provide geotechnical engineering recommendations for use in design and construction of the proposed development. Our scope of work included the following:

- Reviewed readily available published geologic data and our in-house files for existing information on subsurface conditions in the site vicinity.
- Completed a subsurface exploration program consisting of three borings (B-1 through B-3) to depths ranging from approximately 26.5 to 41.5 feet BGS. Two borings (INF-1 and INF-2) were drilled to depths of 10.0 to 17.0 feet BGS for infiltration testing. Infiltration testing was conducted at depths of 15.5 and 8.5 feet BGS in borings INF-1 and INF-2, respectively.
- Maintained continuous logs of the explorations and collected samples at representative intervals.
- Performed the following laboratory tests:
  - Fifteen moisture content determinations in accordance with ASTM D 2216
  - Six fines content determinations in accordance with ASTM D 1140
- Provided recommendations for site preparation and grading, including demolition, temporary and permanent slopes, fill placement criteria, suitability of on-site soil for fill, subgrade preparation, and recommendations for wet weather construction.
- Provided foundation support recommendations for the proposed structure. Our recommendations include allowable bearing capacity and lateral resistance parameters.
- Provided recommendations for use in design of conventional retaining walls, including backfill and drainage requirements and lateral earth pressures.

- Evaluated groundwater conditions at the site, and provided general recommendations for dewatering during construction and subsurface drainage, if required.
- Provided recommendations for construction of asphalt pavements for on-site access roads and parking areas, including subbase, base course, and AC paving thickness.
- Provided seismic design recommendations in accordance with the procedures outlined in the 2012 IBC and 2014 SOSSC.
- Prepared this geotechnical engineering report that presents our findings, conclusions, and recommendations.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The site is located on the east side of N Albina Avenue in Portland, Oregon, and is surrounded by a mixt of residential and commercial properties. The site is currently occupied by two single-story buildings with a paved parking lot to the west of the buildings. The north approximately one-third of the site consists of a fenced grass area. Since the adjacent properties to the north, east, and south are located at higher elevations, the existing buildings are partially embedded on the east and south sides. There is a concrete retaining wall at the south boundary of the site that extends from the south building towards N Albina Avenue. The parking area on the west side of the buildings is relatively flat with ground surface elevations ranging from approximately 199 to 200 feet. The vacant grass area is sloped with an approximate ground surface elevation of 200 feet on the west side near N Albina Avenue, increasing up to an elevation of approximately 212 feet at the east side of the grass area.

3.2 SUBSURFACE CONDITIONS

3.2.1 General

We explored subsurface conditions at the site by drilling three borings (B-1 through B-3) to depths ranging from 26.5 to 41.5 feet BGS. Two borings (INF-1 and INF-2) were drilled adjacent to boring B-2 to conduct infiltration testing. The approximate exploration locations are shown on Figure 2. The exploration logs and laboratory test results are presented in the Appendix.

Our explorations generally encountered variable undocumented fill underlain by sand. At the location of boring B-1, which was drilled in the parking area, the surface consists of approximately 5 inches of concrete underlain by 4 inches of aggregate base. The following sections summarize the subsurface units encountered.

3.2.2 Undocumented Fill

Fill was encountered to depths of approximately 3 and 6 feet BGS in borings B-1 and B-3, respectively. Boring B-3 was drilled in the grass area in the north portion of the site. Fill was not encountered in boring B-2. The fill consists of medium dense, silty sand and medium stiff to stiff silt with varying amounts of sand. In boring B-3, the fill includes trace organics consisting of charcoal and rootlets. Laboratory testing on a selected sample of the fill indicates the moisture content was approximately 24 percent at the time of our explorations.

3.2.3 Sand

The fill is underlain by silty sand and sand with silt. The sand is generally medium dense to dense. Laboratory testing on selected samples of the sand indicates the moisture contents varied from approximately 13 to 27 percent at the time of our explorations.

3.2.4 Groundwater

We did not observe groundwater in our explorations. Based on our review of water well logs on file with the Oregon Water Resources Department and projects completed in the site vicinity, groundwater is generally anticipated at a depth greater than 50 feet BGS. The depth to groundwater may fluctuate in response to seasonal changes, prolonged rainfall, changes in surface topography, and other factors not observed in this study.

3.3 INFILTRATION TESTING

Infiltration testing was completed to assist in the evaluation of stormwater infiltration facilities for the project. The infiltration testing was conducted in general accordance with the recommendations for the "Encased Falling Head" method included in the 2014 City of Portland Stormwater Management Manual. We performed the falling-head infiltration tests in the borings within a 10-inch-diameter casing. The infiltration rate was measured under low-head conditions of approximately 12 inches of water or less after saturated conditions had been achieved.

A representative sample was collected below the infiltration test depths for grain-size analysis. Table 1 summarizes the infiltration test results and fines content determinations. The exploration logs, a description of the infiltration test procedures, and the laboratory test results are presented in the Appendix.

Table 1. Infiltration Rates

Location	Depth (feet BGS)	Material	Infiltration Rate (inches/hour)	Fines Content[1] (percent)
INF-1	15.5	Sand with Silt	4	11
INF-2	8.5	Sand with Silt	4	7

1. Fines content: material passing a U.S. Standard No. 200 sieve

The infiltration rates provided in Table 1 are measured rates and are unfactored. Correction factors should be applied to the measured infiltration rates by the civil engineer during design to account for the degree of long-term maintenance and influent/pre-treatment control, as well as the potential for long-term clogging due to siltation and bio-buildup, depending on the proposed length, location, and type of infiltration facility. In addition, correction factors to be applied to the test results are provided in Exhibit F.2-1 of the 2014 City of Portland Stormwater Management Manual.

4.0 DESIGN RECOMMENDATIONS

4.1 GENERAL

The following sections provide our design recommendations for the project. All site preparation and structural fill should be prepared as recommended in the "Construction" section of this report.

4.2 SHALLOW FOUNDATIONS

4.2.1 General

The proposed structure can be supported on conventional spread footings bearing on undisturbed, firm native soil. Footings should not be directly supported on soft, loose soil or undocumented fill. We anticipate that most of the undocumented fill will be removed with site grading and excavation for the footings, and any remaining fill in the footing subgrade after cutting should be removed and replaced with structural fill.

4.2.2 Bearing Capacity

We recommend that spread footings bearing on the sand be sized based on an allowable bearing pressure of 3,000 psf. This is a net bearing pressure; the weight of the footing and overlying backfill can be ignored in calculating footing sizes. The recommended allowable bearing pressures apply to the total of dead and long-term live loads and may be increased by one-third for short-term loads, such as those resulting from wind or seismic forces.

We recommend that isolated column and continuous wall footings have minimum widths of 24 and 18 inches, respectively. The bottom of exterior footings should be founded at least 18 inches below the lowest adjacent grade. Interior footings should be founded at least 12 inches below the base of the floor slab.

4.2.3 Lateral Resistance

Lateral loads on footings can be resisted by passive earth pressure on the sides of the structure and by friction on the base of the footings. Our analysis indicates that the available passive earth pressure for footings confined by native soil and structural fill is 350 pcf, modeled as an equivalent fluid pressure. Adjacent floor slabs, pavements, or the upper 12-inch depth of adjacent unpaved areas should not be considered when calculating passive resistance.

A coefficient of friction equal to 0.30 may be used when calculating resistance to sliding for footings in direct contact with the sand. Footings in contact with crushed rock should be designed using a coefficient of friction of 0.40.

4.2.4 Settlement

We anticipate that total post-construction settlement will be less than 1 inch for spread foundations designed in accordance with the recommendations provided above. Differential settlement between similarly loaded footings is expected to be less than ½ inch.

4.2.5 Subgrade Observation

All footing and floor subgrades should be evaluated by a representative of GeoDesign to evaluate the bearing conditions. Observations should also confirm that all loose or soft material, organics, unsuitable fill, and softened subgrades (if present) have been removed. Localized deepening of footing excavations may be required to penetrate any deleterious material.

4.3 FLOOR SLABS

To help reduce moisture transmission and to provide uniform support, we recommend a minimum 6-inch-thick layer of floor slab base rock be placed and compacted over prepared subgrade. The floor slab base rock should meet the requirements in the "Materials" section of this report and compacted to at least 95 percent of ASTM D 1557.

Vapor barriers are often required by flooring manufacturers to protect flooring and adhesives. Many flooring manufacturers will warrant their products only if a vapor barrier is installed according to their recommendations. Selection and design of the appropriate vapor barrier (if needed) should be based on discussions among members of the design team. We can provide additional information to assist you with your decision.

Slabs should be reinforced according to their proposed use and per the structural engineer's recommendations. Load-bearing concrete slabs may be designed assuming a modulus of subgrade reaction, k, of 120 psi per inch.

4.4 RETAINING STRUCTURES

As indicated above, the north and south ends of the proposed building will likely be embedded below the ground surface. In addition, retaining walls will be required at the north, east, and south sides of the site, assuming the finish grade of the surface parking will be similar to that of N Albina Avenue. If shoring is required to protect the adjacent buildings or other settlement-sensitive structures during excavation for the site, our recommendations are described in the "Shoring" section of this report. Walls should be designed to resist the earth pressures developed by the shoring system, unless the shoring is designed as a permanent installation. Permanent retaining structures not in contact with temporary shoring should be designed as recommended below.

Our recommendations for permanent retaining walls are based on the following assumptions: (1) the walls are not in contact with temporary shoring, (2) the walls consist of conventional, cantilevered retaining walls or embedded building walls, (3) the walls are less than 15 feet in height, (4) the retained soil is level, and (5) drainage is provided behind the walls to prevent hydrostatic pressures for developing. Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project vary from these assumptions.

Walls not restrained from rotation should be designed using an equivalent fluid pressure of 35 pcf. An equivalent fluid pressure of 55 pcf should be used for design of walls restrained from rotation. These values do not consider hydrostatic pressures. Permanent basement walls with more than one level of bracing should be designed to resist lateral earth pressures presented on Figure 3.

Seismic earth pressures on embedded walls should be designed using a dynamic force of $7H^2$ pounds per linear foot of wall, where H is the wall height. This seismic force should be applied as a distributed load throughout the excavated depth of the retaining wall, with the centroid located at a distance of 0.6H from the base of the wall.

4.5 PAVEMENT

New pavement should be installed on competent subgrade or new engineered fill prepared in conformance with the "Site Preparation" and "Materials" sections of this report. Given the building proposed, our pavement recommendations are based on the assumption that the standard-duty traffic section will be subject to passenger cars and occasional maintenance and delivery truck trucks. We do not have specific information on the frequency and types of vehicles that will use the area; however, we have assumed that standard traffic conditions will consist of a maximum of 2 trucks per day and a maximum of 200 cars per day. We recommend the heavy-duty pavement section be constructed in areas that will be subject to higher traffic volumes (such as entrances and areas subject to repeated delivery vehicles). The heavy-duty section assumes traffic will consist of up to ten trucks per day.

We calculated pavement sections using the above-referenced traffic conditions using a design life of 10 and 20 years and AASHTO design methods. The design of the recommended pavement section is based on an assumed resilient modulus of 4,000 psi and the assumption that construction will be completed during an extended period of dry weather. Wet weather construction may require an increased thickness of aggregate base to support the rock trucks and compaction equipment. Table 2 summarizes the recommended pavement sections.

Table 2. Pavement Section Thickness

Design Life (years)	Standard-Duty Section		Heavy-Duty Section	
	AC Thickness (inches)	Aggregate Base Thickness (inches)	AC Thickness (inches)	Aggregate Base Thickness (inches)
10	2.5	7.0	3.0	10.0
20	2.5	8.0	3.5	10.0

The AC and aggregate base should meet the specifications for ACP and aggregate base rock provided in the "Materials" section of this report.

Construction traffic should be limited to non-building, unpaved portions of the site or haul roads. Construction traffic should not be allowed on new pavements. If construction traffic is to be allowed on newly constructed road sections, an allowance for this additional traffic will need to be made in the design pavement section.

4.6 SEISMIC DESIGN CONSIDERATIONS

4.6.1 IBC Parameters

Based on our explorations, the following design parameters can be applied if the building is designed using the applicable provisions of the 2012 IBC and 2014 SOSSC. The parameters in Table 3 should be used to compute seismic base shear forces.

Table 3. IBC Seismic Design Parameters

Seismic Design Parameter	Short Period (T_s = 0.2 second)	1 Second Period (T_1 = 1.0 second)
MCE Spectral Acceleration, S	S_s = 0.97 g	S_1 = 0.42 g
Site Class	D	
Site Coefficient, F	F_a = 1.11	F_v = 1.58
Adjusted Spectral Acceleration, S_M	S_{MS} = 1.08 g	S_{M1} = 0.66 g
Design Spectral Response Acceleration Parameters, S_D	S_{DS} = 0.72 g	S_{D1} = 0.44 g
Design Spectral PGA	0.29 g	

4.6.2 Liquefaction

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. The excessive buildup of pore water pressure results in the sudden loss of shear strength in a soil. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water. In general, loose, saturated sand soil with low silt and clay content is the most susceptible to liquefaction. Low plasticity, silty sand may be moderately susceptible to liquefaction under relatively higher levels of ground shaking. Liquefaction is not considered a site hazard.

5.0 CONSTRUCTION

5.1 SITE PREPARATION

5.1.1 Demolition

Demolition includes the complete removal of the existing structures, concrete footings, pavement, utilities, and various other former site improvements that may be encountered during construction. We recommend that all abandoned underground vaults, underground storage tanks, septic tanks, manholes, utility lines, foundation elements, and other subsurface structures that are beneath new structural components be entirely removed.

Voids resulting from the removal of improvements should be backfilled with compacted structural fill, as discussed in the "Structural Fill" section of this report. Utility lines abandoned under new structural components should be completely removed and backfilled with structural fill. Firm subgrade should be exposed at the bottom of the excavations before backfilling, and the sides of the temporary excavations should be sloped at a minimum of 1.5H:1V.

Demolished material should be transported off site for disposal. Soft soil encountered during site preparation should be replaced with structural fill.

5.1.2 Clearing

There are some grass areas and trees at the site that will need to be removed. In addition, stumps and root balls should be grubbed out to the depth of the roots, which could exceed 3 feet BGS. Depending on the methods used to remove the root balls, considerable disturbance and loosening of the subgrade could occur during site grubbing. We recommend that soil disturbed during grubbing operations be removed to expose firm, undisturbed subgrade. The resulting excavations should be backfilled with structural fill.

Where present, the existing topsoil zone should be stripped and removed from all fill areas. The average depth of stripping for vegetated areas will be approximately 1 to 2 inches, although greater stripping depths may be required to remove localized zones of loose or organic soil. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal or used in landscaped areas. Stripping should extend at least 5 feet beyond the limits of proposed structural areas.

5.1.3 Fill Improvement

Fill generally consisting of medium dense, silty sand and medium stiff to stiff silt with sand and trace organics was encountered in our explorations to depths of approximately 3 to 6 feet BGS. The thicker fill was encountered in boring B-3, which was drilled in the grass mound at the north end of the site.

Within all proposed structural fill, pavement, at-grade floor slabs, and improvement areas; for a 5-foot margin beyond such areas; and where less than 3 feet of cut is required, if fill is observed at the subgrade elevation, we recommend that the surface foot of the stripped subgrade be removed and replaced with structural fill or the subgrade scarified and compacted as structural fill to a depth of 1 foot.

The exposed subgrade should be closely evaluated by a geotechnical engineer during the process. Considerable soil processing, including moisture conditioning and the removal of roots or other deleterious material from the soil, may be required to use the excavated material as structural fill. Because of the moisture-sensitive nature of the on-site soil, scarification and compaction of the subgrade should be completed during the summer dry period. Compaction should be performed as described in the "Materials" section of this report.

5.1.4 Subgrade Evaluation

Upon completion of demolition, clearing, and subgrade stabilization, and prior to the placement of fill, structures, or pavement improvements, the exposed subgrade should be evaluated by proof rolling. Based on the results of our explorations, our experience with the local soil conditions, and experience with subgrade under prior structures (especially building slabs), we anticipate that relatively easily disturbed soil will be encountered under the existing buildings. The silty sand can be easily damaged during demolition and construction activities. Methods to protect the subgrade from disturbance are provided in the "Construction Considerations" section of this report.

A member of our geotechnical staff should observe the exposed subgrade after demolition, site cutting, and fill removal have been completed to determine if there are additional areas of unsuitable or unstable soil. Our representative should observe a proof roll with a fully loaded dump truck or similar heavy, rubber-tired construction equipment to identify soft, loose, or unsuitable areas. Areas that appear to be too wet and soft to support proof rolling equipment should be evaluated by probing and prepared in accordance with the recommendations for wet weather construction presented in the "Construction Considerations" section of this report.

5.2 CONSTRUCTION CONSIDERATIONS

The fine-grained soil present on this site is easily disturbed. If not carefully executed, site preparation, utility trench work, and excavations can create extensive soft areas and significant repair costs can result. Earthwork planning, regardless of the time of year, should include considerations for minimizing subgrade disturbance.

If construction occurs during or extends into the wet season, or if the moisture content of the surficial soil is more than a couple percentage points above optimum, site stripping and cutting may need to be accomplished using track-mounted equipment. Likewise, the use of granular haul roads and staging areas will be necessary for support of construction traffic during the rainy season or when the moisture content of the surficial soil is more than a few percentage points above optimum. The base rock thickness for pavement areas is intended to support post-construction design traffic loads. This design base rock thickness may not support construction traffic or pavement construction when the subgrade soil is wet. Accordingly, if construction is planned for periods when the subgrade soil is wet, staging and haul roads with increased thicknesses of base rock will be required. The amount of staging and haul road areas, as well as the required thickness of granular material, will vary with the contractor's sequencing of a project and type/frequency of construction equipment. Based on our experience, between 12 and 18 inches of imported granular material is generally required in staging areas and between 18 and 24 inches in haul roads areas. Stabilization material may be used as a substitute provided the top 4 inches of material consists of imported granular material. The actual thickness will depend on the contractor's means and methods and, accordingly, should be the contractor's responsibility. In addition, a geotextile fabric should be placed as a barrier between the subgrade and imported granular material in areas of repeated construction traffic. The imported granular material, stabilization material, and geotextile fabric should meet the specifications in the "Materials" section of this report.

5.3 EXCAVATION

5.3.1 General

Conventional heavy earthmoving equipment in proper working condition should be capable of making necessary excavations of the on-site soil for site cuts and utilities. Soil with more sand content may be prone to raveling, and shoring will be required to maintain vertical excavation walls and protect adjacent facilities.

5.3.2 Temporary Slopes

Where construction slopes are possible, temporary slopes of 1.5H:1V for excavation of the basement may be used to vertical depths of 15 feet or less, provided groundwater seepage is not encountered. At this inclination, the slopes will likely ravel and require some ongoing repair. If seepage is encountered, the slopes should be flattened to protect the surface from raveling. All cut slopes should be protected from erosion by covering them with plastic sheeting during the rainy season. If sloughing or instability is observed, the slope might need to be flattened or the cut supported by shoring.

Excavations should not undermine adjacent utilities, foundations, walkways, streets, or other hardscapes unless special shoring or underpinned support is provided. We recommend a minimum horizontal distance of 5 feet from the edge of the existing improvements to the top of the temporary slope. Unsupported excavations should not be conducted within a downward and outward projection of a 1H:1V line from 2 feet outside the edge of an adjacent structural feature.

5.3.3 Utility Trench Excavation

Trench cuts should stand vertical to a depth of approximately 4 feet in competent soil provided groundwater seepage does not occur in the trench walls. As discussed in the "Temporary Slopes" section of this report, open excavation techniques may be used to excavate trenches with depths up to 10 feet, provided the walls of the excavation are cut at a slope of 1H:1V, groundwater seepage is not present, and surcharge loads are not present within 10 feet of the top of the slope. The walls of the trench should be flattened or braced for stability and a dewatering system installed if seepage is encountered or excessive sloughing and caving occurs. Use of a trench box or other approved temporary shoring is recommended for cuts below the water table. If shoring is used, we recommend that the type and design of the shoring system be the responsibility of the contractor who is in the best position to choose a system that fits the overall plan of operation.

5.3.4 Excavation Dewatering

Excavation dewatering might be required to maintain dry working conditions in excavations depending on the time of year and the severity of rainfall during construction. Based on the results of previous studies at the site, groundwater is anticipated to be relatively deep, at a depth greater than 50 feet BGS. However, perched or static groundwater could be present at shallower depths after prolonged wet periods. Excavation dewatering will be necessary if groundwater is encountered. Dewatering systems are best designed by the contractor; however, assuming that excavations will not exceed more than approximately 6 to 8 feet BGS, it is our opinion that it should be possible to remove groundwater encountered by pumping from a sump in trenches. More intense use of pumps may be required at certain times of the year and where more intense seepage occurs. Removed water should be routed to a suitable discharge point.

If groundwater is present at the base of utility excavations, we recommend placing up to 12 inches of stabilization material at the base of the excavation. Specifications for stabilization material are provided in the "Materials" section of this report.

5.3.5 Safety

All excavations should be made in accordance with applicable OSHA and state regulations. While we have described certain approaches to utility trench excavations in the foregoing discussion, the contractor should be responsible for selecting the excavation and dewatering methods, monitoring the trench excavations for safety, and providing shoring as required to protect personnel and adjacent improvements.

5.4 SHORING

5.4.1 General

If excavations for site development are within the influence zone of the footings of the adjacent structures, shoring will be required to protect the adjacent structures. The influence zone of the existing footings generally extends downwards at a 1H:1V slope from the bottom corner of the footings. We recommend the locations and depths of the existing footings be checked in the field to verify these assumptions. We have provided recommendations below for shoring design.

5.4.2 Lateral Earth Pressures

Shoring should be designed using the values on Figure 4. The recommended design parameters for cantilevered shoring and anchored shoring are shown on Figure 4. Shoring with one level of anchors or bracing should be designed using a triangular pressure distribution as shown for a cantilevered wall on Figure 4 (left). Shoring with more than one level of anchors or bracing should be designed using the earth pressure diagram provided on Figure 4 (right). The above equivalent fluid pressures do not include effects from surcharge loads. The values on Figure 5 can be used to compute surcharge-induced lateral earth pressures.

5.4.3 Soldier Piles

Structural design of the soldier piles should consider the lateral earth pressures discussed above. In addition to lateral earth pressures, the soldier piles will be subject to compressive forces as a result of the downward component of the tieback anchor loads. We recommend a minimum soldier pile embedment of 10 feet below the base of the excavation. We recommend an allowable end bearing capacity of 4 ksf for piles embedded in the sand. An allowable skin friction of 0.5 ksf between the grout and surrounding soil is recommended. In addition, we recommend the grout at the tip of the pile have sufficient strength to withstand the imposed loads. These values should be verified by the structural engineer designing the shoring. Grout should be placed using tremie pipe methods.

We anticipate that lagging will consist of pressure-treated lumber. To maintain the integrity of the excavation, prompt and careful installation of lagging, particularly in areas of seepage and loose soil, is recommended. All voids behind the lagging should be backfilled promptly. To minimize the risk of hydrostatic pressures from developing behind the wall, lean concrete or other low-permeability material should not be used as backfill.

5.4.4 Tieback Anchors

We have provided recommendations for anchored or braced shoring if necessary. The bonded zone for the tieback anchors should be maintained outside of the "unbonded zone" shown on Figure 4. We anticipate the tieback anchors will be capable of achieving allowable bond strengths of between 3 and 5 kips per foot in the sand, depending on the method of construction. A variety of methods are available for construction of tieback anchors. Therefore, we recommend the contractor be responsible for selecting the appropriate bonded length and installation methods to achieve the required anchor capacity. Tieback anchors should be locked off at 100 percent of the design load.

Prior to installing production anchors, we recommend performance tests be conducted on a minimum of two anchors. The purpose of these tests is to verify the installation procedure selected by the contractor before a large number of anchors are installed. Performance tests should be performed to 150 percent of the design load and in accordance with the guidelines provided in *Recommendations for Prestressed Rock and Soil Anchors* (Post Tensioning Institute, 2014).

We recommend proof tests be conducted on all production anchors in accordance with the guidelines presented in *Recommendations for Prestressed Rock and Soil Anchors*. The anchors should be proof tested to at least 133 percent of the design load.

5.4.5 Shoring Dewatering

We do not anticipate dewatering will be required for shoring systems; however, the contractor should be prepared to employ dewatering techniques if necessary. The selection, design, and construction of the temporary dewatering system should be the responsibility of the contractor who is in the best position to modify or adapt the system to changing groundwater conditions and construction sequencing and requirements. The construction dewatering system should be adaptable to varying flow and conditions and capable of lowering the level of the groundwater to a minimum of 2 feet below the base of the excavation.

5.5 DRAINAGE

Where possible, the finished ground surface around the building should be sloped away from the structure at a minimum 2 percent gradient for a distance of at least 5 feet. Downspouts or roof scuppers should discharge into a storm drain system that carries the collected water to an appropriate stormwater system. Trapped planter areas should not be created adjacent to the building without providing means for positive drainage (e.g., swales or catch basins).

5.6 PERMANENT SLOPES

Permanent cut and fill slopes should not exceed 2H:1V. Access roads and pavements should be located at least 5 feet from the top of cut and fill slopes. The setback should be increased to 10 feet for buildings. The slopes should be planted with appropriate vegetation to provide protection against erosion as soon as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

5.7 MATERIALS

5.7.1 Structural Fill

5.7.1.1 General

Fill should be placed on subgrade that has been prepared in conformance with the "Site Preparation" section of this report. A variety of material may be used as structural fill at the site. However, all material used as structural fill should be free of organic matter or other unsuitable material and should meet the specifications provided in OSSC 00330 (Earthwork), OSSC 00400 (Drainage and Sewers), and OSSC 02600 (Aggregates), depending on the application. A brief characterization of some of the acceptable materials and our recommendations for their use as structural fill is provided below.

5.7.1.2 On-Site Soil

The native on-site soil is suitable for use as general structural fill, provided it is properly moisture conditioned; free of debris, organic material, and particles over 3 inches in diameter; and meets the specifications provided in OSSC 00330.12 (Borrow Material). We anticipate some moisture conditioning may be required to dry the soil to a moisture content near optimum. This will require an extended period of dry weather, typically experienced between early July and mid-October. It will be difficult, if not impossible, to adequately compact on-site soil during the rainy season or during prolonged periods of rainfall.

When used as structural fill, the on-site soil should be placed in lifts with a maximum uncompacted thickness of 6 to 8 inches and compacted to not less than 92 percent of the maximum dry density for fine-grained soil and 95 percent of the maximum dry density for granular soil, as determined by ASTM D 1557.

5.7.1.3 Imported Granular Material

Imported granular material used as structural fill should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in OSSC 00330.14 (Selected Granular Backfill) or OSSC 00330.15 (Selected Stone Backfill). The imported granular material should also be angular, fairly well graded between coarse and fine material, have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, and have at least two fractured faces.

Imported granular material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557. During the wet season or when wet subgrade conditions exists, the initial lift should be approximately 18 inches in uncompacted thickness and should be compacted by rolling with a smooth-drum roller without using vibratory action.

5.7.1.4 Stabilization Material

Stabilization material used in staging or haul road areas, or as trench stabilization material, should consist of 4- or 6-inch-minus pit- or quarry-run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in OSSC 00330.15 (Selected Stone Backfill). The material should have a maximum particle size of 6 inches, have less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve, and have at least two mechanically fractured faces. The material should be free of organic matter and other deleterious material. Stabilization material should be placed in lifts between 12 and 24 inches thick and compacted to a firm condition.

5.7.1.5 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should consist of well-graded granular material with a maximum particle size of 1½ inches and less than 10 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in OSSC 00405.13 (Pipe Zone Material). The pipe zone backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D 1557, or as required by the pipe manufacturer or local building department.

Within roadway alignments, the remainder of the trench backfill up to the subgrade elevation should consist of well-graded granular material with a maximum particle size of 2½ inches and less than 10 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in OSSC 00405.14 (Trench Backfill; Class B, C, or D). This material should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D 1557, or as required by the pipe manufacturer or local building department. The upper 3 feet of the trench backfill should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D 1557.

Outside of structural improvement areas (e.g., roadway alignments or building pads) trench backfill placed above the pipe zone may consist of general fill material that is free of organics and material over 6 inches in diameter and meets the specifications provided in OSSC 00405.14 (Trench Backfill; Class A, B, C, or D). This general trench backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D 1557, or as required by the pipe manufacturer or local building department.

5.7.1.6 Drain Rock

Drain rock should consist of angular, granular material with a maximum particle size of 2 inches and should meet the specifications provided in OSSC 00430.11 (Granular Drain Backfill Material). The material should be free of roots, organic matter, and other unsuitable material; have less than 2 percent by dry weight passing the U.S. Standard No. 200 sieve (washed analysis); and have at least two mechanically fractured faces. Drain rock should be compacted to a well-keyed, firm condition.

5.7.1.7 Aggregate Base Rock

Imported granular material used as base rock for building floor slabs and pavements should consist of ¾- or 1½-inch-minus material (depending on the application) and meet the requirements in OSSC 00641 (Aggregate Subbase, Base, and Shoulders). In addition, the aggregate should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve. The aggregate base should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557.

5.7.1.8 Retaining Wall Select Backfill

Backfill material placed behind retaining walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of select granular material that meets the specifications provided in OSSC 00510.12 (Granular Wall Backfill) or OSSC 00510.13 (Granular Structure Backfill).

The backfill should be placed and compacted as recommended for structural fill, with the exception of backfill placed immediately adjacent to walls. Backfill adjacent to walls should be compacted to a lesser standard to reduce the potential for generation of excessive pressure on the walls. Backfill located within a horizontal distance of 3 feet from the retaining walls should be compacted to approximately 90 percent of the maximum dry density, as determined by ASTM D 1557. Backfill placed within 3 feet of the wall should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (such as a jumping jack or vibratory plate compactor). If flatwork (slabs, sidewalk, or pavement) will be placed adjacent to the wall, we recommend that the upper 2 feet of fill be compacted to 95 percent of the maximum dry density, as determined by ASTM D 1557.

5.7.1.10 Recycled On-Site Material

On-site AC, conventional concrete, and aggregate base or gravel may be used as fill if they are processed to meet the requirements for their intended use and the use of these materials do not result in an environmental concern. Processing includes crushing and screening, grinding in place, or other methods to meet the requirements for structural fill as described above. The processed material should be fairly well graded and contain no metal, organic, or other deleterious material. The processed material may be mixed with on-site soil or imported fill to assist in achieving the gradation requirements. We recommend that processed recycled fill have the maximum particle sizes listed in Table 4.

Table 4. Processed Fill Maximum Particle Size

Depth of Placement[1]	Maximum Particle Size
0 to 2 feet	½ inch
2 to 6 feet	2 inches
6 to 10 feet	4 inches
deeper than 10 feet	8 inches

1. below subgrade of structural element

Recycled on-site fill material should not be used within a depth of 2 feet from foundations, floor slabs, pavements, or other subsurface elements. We also caution that excavation through recycled material that is placed as structural fill may be difficult if a significant fraction of oversized particles is present. In addition, these excavations may also be prone to raveling and caving.

5.7.1.11 AC

The AC should be Level 2, ½-inch, dense ACP according to OSSC 00744 (Asphalt Concrete Pavement). Minimum lift thickness for ½-inch ACP is 2.0 inches. Asphalt binder should be performance graded and conform to PG 64-22. The AC should be compacted using minimum and maximum lifts of 2.0 and 3.0 inches, respectively.

5.7.1.12 Geotextile Fabric

Subgrade Geotextile Fabric

A subgrade geotextile fabric should be placed as a barrier between the subgrade and granular material in staging areas, haul road areas, or in areas of repeated construction traffic. The geotextile should meet the specifications provided in OSSC 02320 (Geosynthetics) for separation geotextiles (Table 02320-4) and be installed in accordance with OSSC 00350 (Geosynthetic Installation). The geotextile should have a Level "B" certification.

Drainage Geotextile Fabric
Drain rock, and other granular material used for subsurface drains, should be wrapped in a geotextile fabric that meets the specifications provided in OSSC 02320 (Geosynthetics) for drainage geotextiles (Table 02320-1) and be installed in accordance with OSSC 00350 (Geosynthetic Installation).

5.8 EROSION CONTROL

The site soil is susceptible to erosion; therefore, erosion control measures should be carefully planned and in place before construction begins. Surface water runoff should be collected and directed away from slopes to prevent water from running down the slope face. Erosion control measures (such as straw bales, sediment fences, and temporary detention and settling basins) should be used in accordance with local and state ordinances.

6.0 OBSERVATION OF CONSTRUCTION

Satisfactory foundation and earthwork performance depends to a large degree on quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during the subsurface exploration. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect if subsurface conditions change significantly from those anticipated.

We recommend that GeoDesign be retained to observe earthwork activities, including stripping, proof rolling of the subgrade and repair of soft areas, footing subgrade preparation, performing laboratory compaction and field moisture-density tests, observing final proof rolling of the pavement subgrade and base rock, and asphalt placement and compaction.

7.0 LIMITATIONS

We have prepared this report for use by Andy Schreck and members of the design and construction teams for the proposed project. The data and report can be used for bidding or estimating purposes, but our report, conclusions, and interpretations should not be construed as warranty of the subsurface conditions and are not applicable to other nearby building sites.

Exploration observations indicate soil conditions only at specific locations and only to the depths penetrated. They do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, re-evaluation will be necessary.

The site development plans and design details were preliminary at the time this report was prepared. When the design has been finalized and if there are changes in the site grades or location, configuration, design loads, or type of construction for the buildings, and walls, the conclusions and recommendations presented may not be applicable. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification.

The scope does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with generally accepted practices in this area at the time the report was prepared. No warranty, express or implied, should be understood.

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We appreciate the opportunity to be of service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

GeoDesign, Inc.

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