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December 15, 2014

Ray Kistler
Kistler Small + White Architects
552 A Street Suite 1
Ashland, Oregon 97520

**RE: GEOTECHNICAL INVESTIGATION REPORT
REPLACE SEISMICALLY DEFICIENT DOM BUILDING 206
DVA SOUTHERN OREGON REHABILITATION CENTER & CLINICS
WHITE CITY, OREGON
MAI JOB NO. 14-1196**

Dear Mr. Kistler:

Introduction

We are pleased to present our geotechnical investigation report for the proposed Replace Seismically Deficient Dom Building 206 project at the DVA Southern Oregon Rehabilitation Center & Clinics in White City, Oregon. The purpose of this investigation was to evaluate the subsurface conditions at the site and develop earthwork and foundation engineering recommendations for the project design. The proposed development is shown on Drawing 1, Site Plan.

The existing Building 206 will be demolished and replaced with a new structure as approximately located on Drawing 1. The new structure will be two stories and will utilize a raised structural floor at the lower level. The lower level floor of the new building is expected to nearly match the grade of the existing lower level floor (elevation 1330.27). A basement with a slab-on-grade floor is planned at the middle of the structure and will connect to the existing basement level corridor as approximated on Drawing 1. The basement grade will be about 11 feet below the ground floor level. Present plans suggest that the basement floor will consist of gravel, rather than a slab.

As part of this project, a large portion of Building 250 will be demolished and removed to make room for new paved parking extending easterly from the existing adjacent parking lot. As shown on Drawing 1, the proposed paving will extend north from Building 249 to about the northern limits of Building 250 and will extend easterly to the utility tunnel.

This report has been prepared for the specific use of Kistler Small + White Architects and their consultants in accordance with generally accepted soil and foundation engineering principles and practices. No other warranty, either expressed or implied, is made. In the event that any substantial changes in the nature, design, or locations of the improvements are planned, the conclusions and recommendations of this report shall not be considered valid unless such changes are reviewed and the conclusions of this report modified or verified in writing.

It should be recognized that changes in the site conditions may occur with the passage of time due to environmental processes or man-made changes. Furthermore, building code or state of the practice changes may require modifications in the recommendations presented herein. Accordingly, the recommendations of this report should not be relied on beyond a period of three years without being reviewed by a geotechnical engineer.

Method of Investigation

Two exploratory borings were drilled on November 21, 2014, with a truck-mounted Mobile B59 hollow stem auger at the locations shown on Drawing 1. Two hand-excavated borings were also drilled adjacent to Building 250 on December 1, 2014, and their locations are also shown on Drawing 1. The borings were located in the field by taping from the features shown on the site plan provided us. A key describing the soil classification system and soil consistency terms used in this report is presented on Drawing 2 and the boring sampling procedures are presented on Drawing 3. Logs of the exploratory borings are presented on Drawings 5, 6, and 7.

Samples of the soil materials from the exploration were returned to our laboratory for classification and testing. The results of moisture content and percent finer than No. 200 sieve are shown on the logs. The laboratory test procedures followed during this investigation are summarized on Drawing 4.

A bibliography of references is included at the end of the text.

Site Conditions

A. Surface

The lower level floor (elevation 1330.27) of the existing building is elevated about 2.8 feet above existing site grades. There is a basement of sorts beneath the center of the existing building. The general site grades in the building vicinity are relatively flat and lie at about elevation 1327.5. The ground surface is landscaped with trees, shrubs, and lawn areas, and improved with concrete sidewalks and ramps leading to the building. A small storm water detention/retention basin lies between the north wings of Buildings 205 and 206. Additionally, a large storm water detention/retention basin lies just south of Building 205.

The ground adjacent to Building 250 is relatively level and varies between elevations 1326.5 and 1327.5. The ground is mostly grassy with sidewalk slabs leading to door entrances.

B. Subsurface

The borings generally encountered three layers of earth materials: artificial fill, natural sandy clay, and dense granular alluvium.

Artificial Fill. Artificial fill was encountered to a depth of 1.0' below existing grade at Boring 1 and to a depth of 1.5' below existing grade at Boring 3, but was not encountered in the other borings. The fill consisted of a variety of crushed rock, clayey sand, and silty sand.

Natural Sandy Clay. This soil was observed below the fill at Boring 1 and at the ground surface at Borings 2 and 4. This material consisted of medium stiff to stiff very sandy clay and was about 1' thick or less.

Dense Granular Alluvium. Dense granular alluvium consisting of sandy gravels with varying silt and clay and scattered layers of clayey sand was encountered beneath the natural sandy clay. These dense soils were encountered to the maximum depth explored in the borings (20 feet at Boring 2).

The surficial clayey soil is considered to be moderately expansive based on our previous work for adjacent structures. The dense granular alluvium is considered to be non-expansive to slightly expansive based on its low fines content.

The attached boring logs and related information depict subsurface conditions only at the specific locations shown on Drawing 1 and on the dates drilled. Subsurface conditions at other locations may differ from conditions occurring at these locations. Also, the passage of time may result in a change of soil conditions at these locations due to environmental changes.

C. Groundwater

Free groundwater was observed in Boring 1 at a depth of 12 feet and in Boring 2 at a depth of 9.5 feet. We speculate that the higher groundwater level at Boring 2 is at least partially due to the presence of the large nearby storm water detention/retention basin south of Building 205. Prior experience at the VA facility has shown that the groundwater has risen to as high as seven to eight feet below the existing ground surface. The relatively high groundwater has been controlled by the installation of basement level underdrain systems and sump pumps in the newer buildings at the facility.

Free groundwater was not observed in the shallow borings drilled near Building 250. Fluctuations in the groundwater level can occur because of variations in rainfall, temperature, runoff, irrigation, and other factors not evident at the time our observations were made and reported herein.

Geology and Seismicity

A. Geology

The VA facility lies within the Rogue Valley. The Rogue Valley is bounded by the Western Cascades physiographic province on the north and east and the Klamath Mountains province on the west and south.

The geologic map of the area (Wiley and Smith, 1993) indicates the site is underlain by Pleistocene Age Older Alluvium. This surface deposit consists of mixtures of gravel, sand, silt, and clay that are locally cemented and the deposit is characteristically at least 30 feet in elevation above major modern stream channels. The Older Alluvium is the oldest of four mapped stages of Quaternary alluvial fans and valley fill in the Rogue Valley.

The geologic map of the area (Wiley and Smith, 1993) also suggests that bedrock underlies the site at a depth of about 40 feet. However, recent drilling by others suggests that bedrock underlies the site at depths greater than 60 feet below existing grade.

B. Seismicity

Southern Oregon is in an area of moderate to potentially high seismic activity. As with the entire Oregon coastal belt, the site is in a region that is dominated seismically by the Cascadia Subduction Zone. The subduction zone is formed by the sinking of the offshore Juan de Fuca Plate beneath the onshore North American Plate. Earthquakes are generated within the subducting Juan de Fuca Plate (intraslab), at the frictional contact between the two plates (interface), and within the upper North American Plate (crustal). From an historical perspective, recorded seismicity in the region has been relatively low in comparison to Northern Oregon and Northern California.

The Cascadia Subduction Zone (CSZ) is capable of great earthquakes with Moment Magnitudes (M_w) of 8.5 plus and lies about 110 miles to the west. The potential ground shaking from the CSZ would likely be of greater severity and duration than earthquakes generated from intraslab and crustal faults.

Intraslab earthquakes of M_w 7.0 plus are capable on the seismogenic part of the subducting plate in the CSZ. These earthquakes typically occur at depths of 40 to 60 km.

Crustal earthquakes of up to M_w 6.5 can occur at relatively shallow depths of 25 km or less. Crustal faults typically produce most of the earthquakes in the region. Historically, crustal earthquakes have not exceeded M_w 6.0 and are usually less than M_w 4.5.

All of the above types of earthquakes are considered potential “design earthquakes” by the building code.

C. Faulting

The nearest mapped (Wiley and Smith 1993) fault lies about four miles to the southeast of the V.A. facility. This fault, and others in the Rogue Valley, offset pre-Quaternary geologic formations and are not considered active or potentially active.

The nearest known active fault (fault displaying movement within the last 10,000 years) system is the Sky Lakes Fault Zone that lies about 35 miles east of the site.

A few miles east of the Sky Lakes Fault Zone lies the active Klamath Graben faults. The Klamath Falls earthquakes of 1993 (M_w 5.9, M_w 6.0, and several small aftershocks) occurred on the Klamath Graben faults.

Geologic and Seismic Hazards Evaluation

A. Design Earthquake

The design earthquake for the project area is based on methodologies in the Code and was determined from on-line USGS seismic design maps (2012 IBC). The site has a Maximum Considered Earthquake (MCE) spectral response acceleration at 0.2 seconds for Site Class B of S_s=0.600g. The site also has an MCE spectral response acceleration at 1.0 second for Site Class B of S_1=0.324g.

Based on the subsurface boring information and the provisions in the Code, a Site Class C designation may be assumed for this site.

B. Fault Offset

Based on our review of existing geologic information, we conclude that there are no known active or potentially active faults in the vicinity of the project site. Therefore, the hazard resulting from surface rupture or fault offset is considered low.

C. Shaking

Based on on-line USGS seismic design maps and the previous Code (ASCE 7-10 Standard), the expected peak ground acceleration at this site for the Maximum Considered Earthquake is about 0.28g.

Ground amplification effects at the site are expected to be properly accounted for using the Code seismic design methodology.

Moderate to strong ground shaking could occur at the site as a result of an earthquake in the region. The proposed improvements should be designed and constructed in accordance with current standards of earthquake-resistant construction.

Ground shaking during an earthquake could cause objects within the building which are not rigidly attached to the structure to undergo some movements with respect to the structure. The building should, therefore, include design measures that minimize such potential movements and also minimize the adverse effects of such movements where they cannot be prevented.

D. Soil Liquefaction

Liquefaction is a phenomenon in which saturated cohesionless soils lose strength during strong shaking and experience horizontal and vertical movements. Soils that are most susceptible to liquefaction are clean, loose, saturated, uniformly graded, fine-grained, clay-free sands that lie within 50 feet of the ground surface.

The existing dense granular soils at the site are considered to be resistant to liquefaction. As such, we believe the potential for liquefaction at the site is low.

E. Other Hazards

Due to the relatively strong nature of the dense underlying gravels, the potential for other hazards such as shaking settlement, lateral spreading, and lurching are low. Due to the relatively flat ground in the site vicinity, the risk of landsliding affecting the site is low.

Conclusions and Recommendations

From a soil and foundation engineering standpoint, it is our opinion that the proposed development can be constructed as proposed provided the recommendations contained in this report are incorporated into the design and construction of the project.

Soil Conditions. The site soils can generally be characterized as a thin surficial layer of soft clayey soil underlain at a shallow depth by dense granular soil. The surficial sandy clay is too soft to provide reliable support for footings.

Foundations. Conventional spread footings bearing on the dense granular soils may be used to support the new building. The footings may either bear directly on the dense soil or on structural fill underlain directly by the dense granular soil.

Groundwater and Subdrainage. Normally long-term monitoring is required to determine the highest groundwater level at a site. This monitoring was not possible for the purpose of this investigation. For the sake of prudent design, we recommend that the high groundwater level at the site be assumed to be eight feet below the ground surface.

Basement gravel floors and concrete retaining walls should be protected with subdrainage and membrane protection systems. These measures should include, but not be necessarily limited to, floor underdrains, retaining wall backdrains, and retaining wall waterproofing materials.

Excavation. The dense granular soils are expected to be readily diggable with conventional equipment.

Detailed recommendations for the project design are presented in the remainder of the report. These recommendations are contingent on our review of the construction drawings and observation of the earthwork, foundation, and drainage installation phases of construction.

Recommendations

A. Earthwork

1. The site should be cleared of the buildings, slabs, footings, utilities to be abandoned, and any remaining obstructions and stripped of topsoil and surface organics within the work area. Holes resulting from the removal of underground obstructions should be backfilled with suitable material and compacted to the requirements for structural fill given below. The clearing of holes beneath the proposed finished grades and the backfilling operations should be performed under our observation.

2. After the site excavations are completed, the exposed subgrade soils in areas to be filled should be recompacted prior to placing additional fill. The recompaction may be waived by the soil engineer if the subgrade materials are hard and undisturbed. The recompaction should consist of scarifying the upper six inches, moisture conditioning the soils to approximately three percent above optimum, and compacting them to at least 95 percent relative compaction as determined by ASTM Test Method D698. Compaction should be performed using heavy equipment such as a large vibratory roller.

3. In order to achieve satisfactory compaction in the subgrade and fill soils, it may be necessary to adjust the soil moisture content at the time of construction. Soils which are too dry will require the addition of water while scarification and aeration will be required for soils which are too wet.

4. Structural fill may include high quality 3/4"-0 crushed rock or 4"-0 crushed rock beneath footings, slabs, and pavements. The choice of structural fill materials should be compatible with the method of placement and compaction.

5. Structural fill should be compacted to at least 95 percent relative compaction as determined by ASTM Test Method D698. Fill materials should be moisture-conditioned and spread in lifts not exceeding eight inches in uncompacted thickness. Where practical, compaction should be performed with a heavy self-propelled vibratory roller capable of producing at least 24,000 pounds dynamic force. Compaction behind retaining walls should be performed with portable light equipment to prevent overstressing of the wall.

   The compaction of the fill, thickness of lifts, and control of the moisture content should be monitored and tested by our field representative. Compaction should be evaluated by the use of nuclear gauge field density testing and, where appropriate, by proofrolling with loaded 10 cy gravel trucks.

6. Utility trenches should be backfilled with compacted fill placed in lifts not exceeding eight inches in uncompacted thickness, except thicker lifts may be used with the approval of the soils engineer provided satisfactory compaction is

achieved. The upper three feet of trench backfill should be compacted to at least 95 percent relative compaction (ASTM D698). Jetting of backfill to obtain compaction should not be permitted.

7. Grading and earthwork should be monitored and tested by our representative for conformance with the project plans/specifications and our recommendations. This work includes site preparation, selection of satisfactory fill materials, and placement and compaction of the subgrades and fills. Sufficient notification prior to commencement of earthwork is essential to make certain that the work will be properly observed.

B. Foundations

1. Spread footings may bear directly on the underlying dense granular alluvial soils. Footings may also bear on structural fill that is in turn underlain by dense granular alluvial soil. All existing weak soil, including the existing sandy clay and possibly any old fill, must be removed from beneath foundations. The dense granular soils were encountered in Borings 1 and 2 at depths of 1.0 to 1.5 feet below existing grades.

   The bottoms of all footing excavations should be cleaned of loosened material and checked by our field representative for soft material.

2. Where structural fills are utilized beneath footings, the structural fills should extend laterally at least 1.0 feet beyond the sides of the footings.

3. Footings should bear at least twelve inches below lowest adjacent finished grade. Footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the edge of the bottom of the trench.

4. Footings constructed in accordance with these recommendations can be designed for an allowable bearing pressure of 3000 psf for dead plus live loads. This pressure may be increased by one-third for total loads including wind or seismic. All footings should be provided with sufficient reinforcement to provide structural continuity.

5. Lateral loads can be resisted by friction between the foundation bottoms and the supporting subgrade. A friction coefficient of 0.35 may be used. In addition, a passive pressure equal to an equivalent fluid pressure of 300 pcf may be used starting at a depth of 0.5 feet below finished grade.

6. Foundation settlements are expected to be less than 3/4 inch and within tolerable limits for the proposed construction.

C. Slabs-On-Grade

1. No basement level slabs are anticipated. Sidewalk slabs at the ground surface should be underlain by at least 12" of structural fill such as compacted ¾"-0 crushed rock.

D. Basement Retaining Walls

1. Basement retaining walls should be supported on foundations designed in accordance with our previous recommendations. Unrestrained walls with level backslopes and backfilled with well-draining backfill may be designed to resist an equivalent fluid pressure of at least 40 pcf. Where restrained, walls should be designed for an equivalent fluid pressure of 60 pcf. The walls should also be designed for a peak horizontal ground acceleration of 0.13 g. These pressures do not account for any surcharge loadings or saturated backfills. Surcharge loadings and saturated backfills should be evaluated on a case-by-case basis.

2. The preceding pressures assume that sufficient drainage is provided behind the walls to prevent the build-up of hydrostatic pressures from surface or subsurface water infiltration. Adequate drainage may be provided by means of 3/4 inch drain rock material enclosed in a filter fabric and a four inch diameter rigid perforated pipe placed at the base of the wall. The drainrock should extend up the walls to within one foot of finished grade. The drain pipes should be tied into closed pipes that discharge into suitable facilities. A detail displaying the recommended backdrainage is shown on Drawing 8, Retaining Wall Detail.

3. The backfill placed behind retaining walls should be granular, free-draining, and non-expansive and compacted to at least 95 percent relative compaction using light compaction equipment. All interior walls should be waterproofed and the waterproofing should be protected with protection boards.

E. Flexible Pavements

1. Based on our previous experience with similar soil conditions, we recommend the following sections:

Recommended Pavement Sections

Traffic Condition	Asphalt Concrete (inches)	3/4"-0 Crushed Rock (inches)	4"-0 Crushed Rock (inches)
Auto Parking Only	2.0	4.0	12.0
Drive Aisles and Heavy Vehicle Lanes	3.0	6.0	12.0

2. The 3/4"-0 crushed rock should meet Section 02630, latest ODOT/APWA Standard Specifications. The crushed rock should be placed in a manner to prevent segregation and should be uniformly moisture-conditioned and compacted to at least 95 percent relative compaction (ASTM D698, Method A) to provide a smooth, unyielding surface.

3. The 4"-0 crushed rock should be high quality, processed 4"-0 crushed rock that is approved for use on City streets. The crushed rock must be dense after compaction and non-deflecting under proofrolling with a fully loaded ten-yard gravel truck.

   The crushed rock should be underlain by a 5 oz/yd minimum non-woven permeable geotextile fabric.

4. The subgrade beneath the 4"-0 crushed rock should be unyielding under the wheels of a fully loaded 10 cu. yd. dump truck and compacted to at least 95 percent relative compaction (ASTM D698). If the subgrade is not hard and unyielding at the time of construction, the weak subgrade will need to be removed and replaced with structural fill such as compacted 4"-0 crushed rock. All fill placed beneath the pavement section should be compacted to at least 95 percent relative compaction (ASTM D698). Grading for pavements should be performed during the dry and warm months of the year.

F. <u>Drainage</u>

1. Positive surface gradients of at least two percent on paved surfaces and five percent in landscaping areas should be maintained away from the building so that water does not collect in the vicinity of the foundations. Water from roof downspouts should be collected into closed pipes that discharge the water into approved drainage facilities.

2. A foundation drain should be placed adjacent to the perimeter building footings (where retaining wall backdrains are not present) to help control moisture in the crawlspace.

3. The gravel floor in the basement should be constructed as a drainage system to collect and convey groundwater to an acceptable drainage facility for pumped discharge into the street drains. The gravel floor should consist of at least 12" of mechanically tamped free-draining ¾" crushed rock (no fines, no round rock) over the subgrade soil. The subgrade soil surface should be uniformly graded to promote drainage to the drainage facility.

G. <u>Construction Observation</u>

1. We should be retained to provide monitoring services during the grading, foundation, and drainage installation phases of the project. This will provide the opportunity for correlation of the soil conditions found in our investigation with

those actually encountered in the field, and thus permit any necessary modifications in our recommendations resulting from changes in anticipated conditions.

We have provided our findings and recommendations in accordance with generally accepted geotechnical engineering principles and practices. No other warranty, either expressed or implied, is made.

REGISTERED PROFESSIONAL ENGINEER
16885
OREGON
SEPT. 21, 1993
ERIC A. SWANSON
EXPIRES: 6-30-2016

Very truly yours,

MARQUESS & ASSOCIATES, INC.

Rick Swanson, P.E., G.E.
Civil Engineer 16885

RS/ler

Copies:	Addressee (4), and by email
	Teresa Kellim, by email
Attachments:	Bibliography
	Site Plan, Drawing 1
	Key to Boring and Pit Logs, Drawing 2
	Boring Sampling Procedures, Drawing 3
	Laboratory Testing Procedures, Drawing 4
	Logs of Borings 1 and 2, Drawings 5 and 6
	Logs of Borings 3 and 4, Drawing 7
	Retaining Wall Detail, Drawing 8

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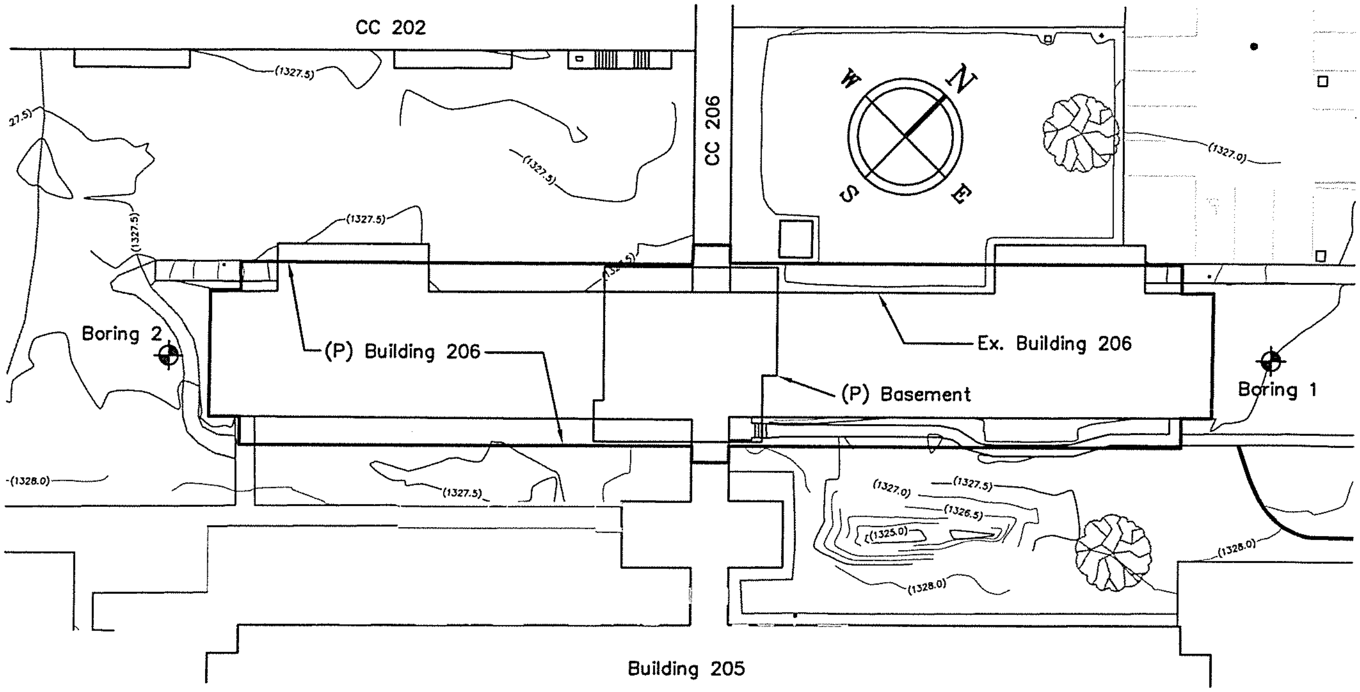

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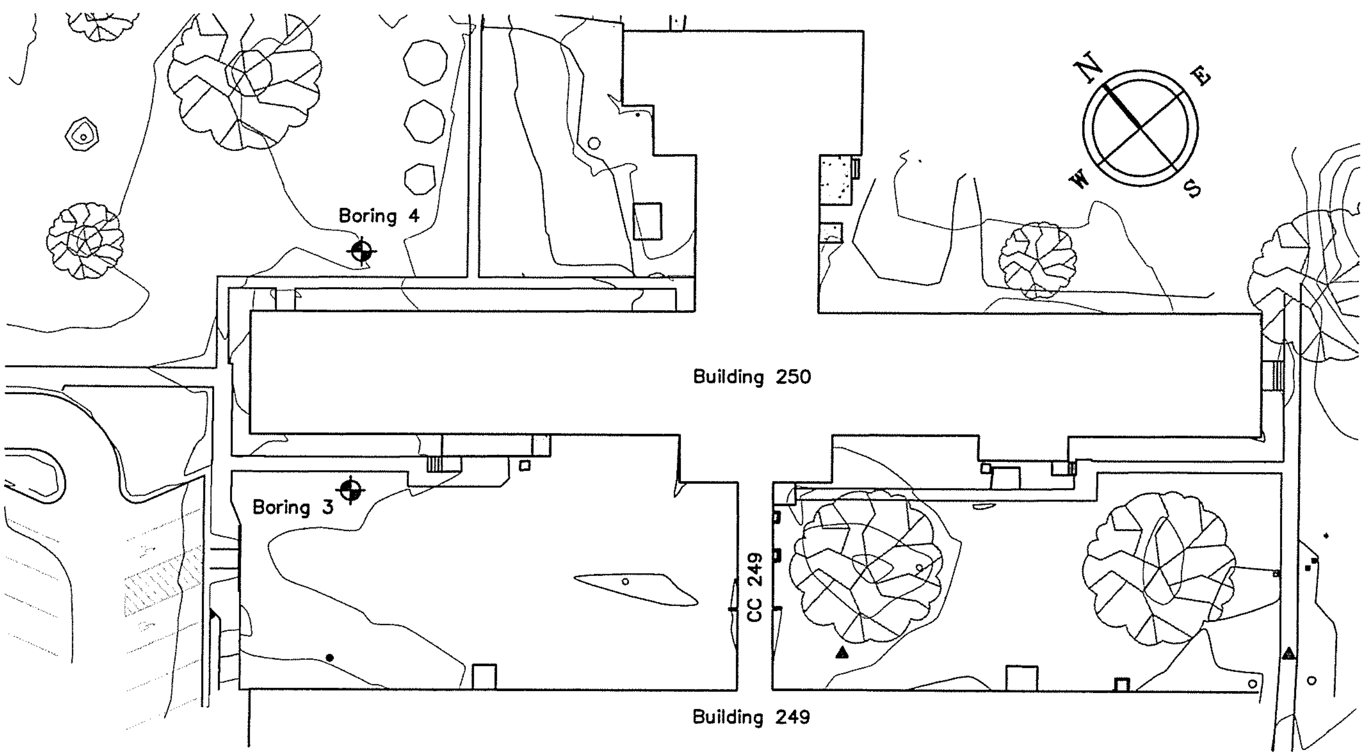

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Site Plan

Replace Dom Building 206

DVA Southern Oregon Rehabilitation Center & Clinics

White City — Oregon

MAI JOB NO.	14-1196	DRAWN	RS
ISSUE DATE	Dec 2014	CHECKED	RS

DRAWING

1

OF 8 DWGS

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN No. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN No. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel–sand mixtures, little or no fines.
			GP	Poorly graded gravels, or gravel–sand mixtures, little or no fines.
		GRAVEL WITH FINES	GM	Silty gravels, gravel–sand–silt mixtures, non–plastic fines.
			GC	Clayey gravels, gravel–sand–clay mixtures, plastic fines.
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN No. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand–silt mixtures, non–plastic fines
			SC	Clayey sands, sand–clay mixtures, plastic fines.
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN No. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D–2487)

U.S. STANDARD SERIES SIEVE / CLEAR SQUARE SIEVE OPENINGS

	200	40	10	4	3/4"	3"	12"
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

GRAIN SIZES

SANDS & GRAVELS	BLOWS/FOOT†
VERY LOOSE	0 – 4
LOOSE	4 – 10
MEDIUM DENSE	10 – 30
DENSE	30 – 50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS & CLAYS	STRENGTH‡	BLOWS/FOOT†
VERY SOFT	0 – 1/4	0 – 2
SOFT	1/4 – 1/2	2 – 4
FIRM	1/2 – 1	4 – 8
STIFF	1 – 2	8 – 16
VERY STIFF	2 – 4	16 – 32
HARD	OVER 4	OVER 32

CONSISTENCY

† Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1–3/8 inch I.D.) split spoon (ASTM D–1586).

‡ Unconfined compressive strength in tons/sq. ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D–1586), pocket penetrometer, torvane, or visual observation.

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KEY TO BORING AND PIT LOGS

Replace Dom Building 206

DVA Southern Oregon Rehabilitation Center & Clinics

White City Oregon

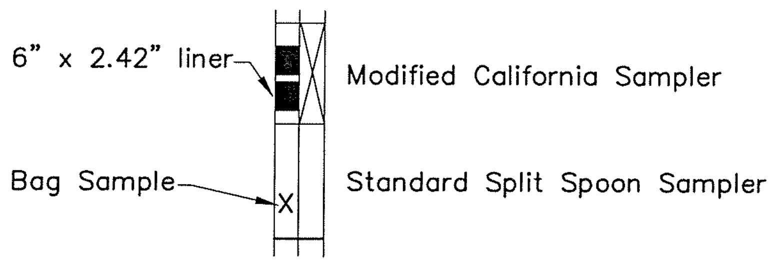
MAI JOB NO.	14-1196	DRAWN	RS
ISSUE DATE	Dec 2014	CHECKED	RS

BORING SAMPLING PROCEDURES

The soils encountered in the borings were continuously logged in the field by our representative and described in general accordance with the Unified Soil Classification System (ASTM D-2487).

Representative soil samples were obtained from the borings at selected depths appropriate to the soil investigation. All samples were returned to our laboratory for classification and testing.

The standard penetration resistance blow counts were obtained in general accordance with the ASTM D1586 procedure by dropping a 140 pound hammer through a 30-inch free fall. The 2-inch O.D. split spoon sampler was driven 18 inches or to practical refusal and the number of blows were recorded for each 6-inch penetration interval. The blows per foot recorded on the boring logs represent the accumulated number of blows required to drive the penetration sampler the final 12 inches. In addition, 3.0 inch O.D. x 2.42 inch I.D. drive samples were obtained using a Modified California Sampler and a 140 pound hammer. Blow counts for the Modified California Sampler are shown converted to standard penetration resistance by multiplying by 0.6. The sample type is shown on the logs in accordance with the designation below.



Where obtained, the shear strength of the soil samples using either Torvane (TV) or Pocket Penetrometer (P) devices is shown on the boring logs in the far right hand column.

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BORING SAMPLING PROCEDURES
Replace Dom Building 206
DVA Southern Oregon Rehabilitation Center & Clinics
White City Oregon

MAI JOB NO.	14-1196	DRAWN	RS
ISSUE DATE	Dec 2014	CHECKED	RS

LABORATORY TESTING PROCEDURES

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site.

The natural water content was determined on eleven samples of the materials recovered from the borings in general accordance with the ASTM Test Method D2216. These water contents are recorded on the logs at the appropriate sample depths.

The percent soil fraction passing the #200 sieve was determined on three samples of the subsurface soils in general accordance with the ASTM Test Method D1140 to aid in the classification of the soils. The results of these tests are shown on the logs at the appropriate sample depths.

EQUIPMENT: Mobile B-59 hollow stem auger	ELEVATION: 1327.5 approx.	LOGGED BY: RS
DEPTH TO GROUNDWATER: 12'	DEPTH TO BEDROCK: Not Observed	DATE DRILLED: 11-21-14

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
FILL. Gray crushed rock and gray clayey sand									
SANDY CLAY, very moist, with gravel	Brn-Gry	Stiff	CL	1					
SANDY GRAVEL, moist	Brown	Very Dense	GP-GM	2	X	50/6"	7		
				3					
				4					
				5	X	79	12		
CLAYEY SAND, mist	Brown to Dark Brown	Dense	SC	6					
				7					
SANDY GRAVEL, moist	Brown	Very Dense	GM	8					
				9					
@10.5': Finer than #200 = 12 %				10	X	50/5"	22		
@11.5' – bottom: occ'l 1' thick interbeds of dark brown, medium dense to dense clayey sand				11					
				12			▼	(during drilling)	
				13					
				14					
				15	X	63	34		
				16					
				17					
				18	X	50/6"	14		
				19					
Bottom of Boring = 19'				20					

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LOG OF BORING 1

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EQUIPMENT: Mobile B-59 hollow stem auger	ELEVATION: 1327.5 approx.	LOGGED BY: RS
DEPTH TO GROUNDWATER: 9.5'	DEPTH TO BEDROCK: Not Observed	DATE DRILLED: 11-21-14

DESCRIPTION AND REMARKS	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
SANDY CLAY, very moist	Dk Brn	Medium Stiff	CL-SC	0-1	X		29		
SANDY GRAVEL, moist, with occ'l 1' seams of gray or brown clayey sand	Brown	Dense	GM-GC	1					
		Very Dense		2	X	20/3"	13		
@0.5': Finer than #200 = 51 %				3					
@6': Finer than #200 = 20 %				6	X	52	18		
				9.5			▼ (during drilling)		
		Dense		11	X	48	20		
		Very Dense		14.5	X	50/5"	13		
Bottom of Boring = 20'				19.5	X	80	29		

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LOG OF BORING 2

Replace Dom Building 206

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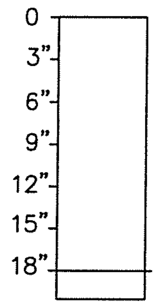
DRAWING

6

OF 8 DWGS

Boring 3

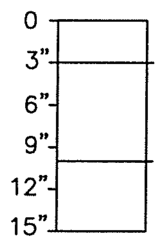


FILL. Upper 12" is CLAYEY SAND (SC), dark brown, loose, moist, followed by FILL consisting of mixed red–brown CLAYEY SAND (SC), gravels to 2" in diameter, and small inclusions green–gray SILTY SAND (SM) (decomposed granite)

CLAYEY SAND (SC), brown, dense to very dense, moist, gravelly, hard to dig

Bottom of hand dug boring = 20"
Practical excavation refusal at 20" due to packed gravels

Boring 4



SANDY CLAY (CL–SC), dark brown and gray, loose/soft, wet

SANDY CLAY (CL–CH), gray, medium stiff, very moist

CLAYEY SAND (SC), brown, dense to very dense, moist, gravelly, hard to dig

Bottom of hand dug boring = 15"

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Logs of Borings 3 and 4
Replace Dom Building 206
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MAI JOB NO.	14-1196	DRAWN	RS
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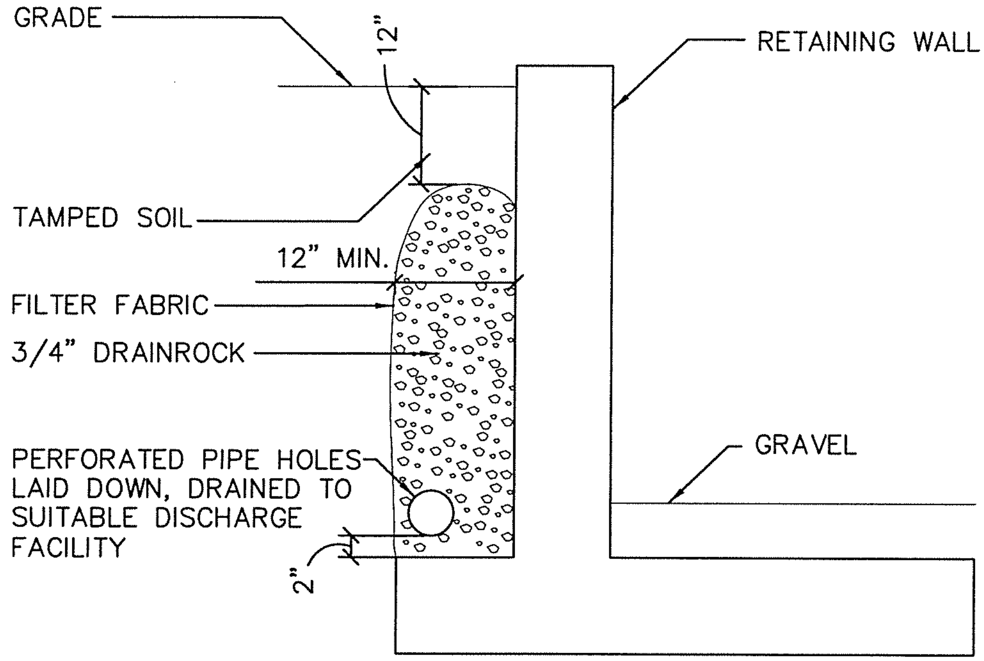
DRAWING
7
OF 8 DWGS



Retaining Wall

NOTES:

1. Perforated pipe should be at least 4" diameter and rigid (SDR of 35 or less). Subdrain should be located several inches below adjacent finished grade.

2. Weepholes may be used behind exterior walls where seepage will not detract from the wall's usefullness or create future moisture or drainage problems.

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RETAINING WALL DETAIL
Replace Dom Building 206
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