Department of Community Services
Land Use Planning Division
www.multco.us/landuse

Multnomah County

1600 SE 190th Avenue, Portland Oregon 97233-5910 • PH. (503) 988-3043 • Fax (503) 988-3389

AGENCY REVIEW

Attached is a site review permit application (as submitted). Please evaluate and comment on these materials so that we can incorporate your feedback into our completeness review. This is not a substitute for public notice of a complete application. Once we determine the application is complete an additional notice will be mailed (with any revised information), offering you the opportunity to comment or informing you of a date for public hearing, as appropriate.

National Scenic Area Site Review

To:
☒ Gorge Commission/Cultural Advisory Committee
☒ U.S. Forest Service NSA Office
☒ Confederated Tribes of Warm Springs
☒ Confederated Tribes of the Umatilla Indian Reservation
☒ Nez Perce Tribe
☒ Yakama Indian Nation
☒ State Historic Preservation Office

From: Lisa Estrin, Planner

Vicinity Map N↑

Case File: T2-2017-8783

Location: 2501 NE Rasmussen Rd, Corbett
Tax Lot 200, Section 27DA, Township 1 North, Range 4 East, W.M.
Alternative Account #RR177900560

Proposal: Applicant is applying for a NSA Post-Emergency / Disaster Response Site Review Application to authorize the construction of a new retaining wall on the subject property due to the failure of an existing wall.

Your written comments are needed no later than 4:00 p.m., Thursday, September 14, 2017.

Zoning: Gorge General Residential – 10 (GGR-10) / Hillside Development (HD) ☒ GMA ☐ SMA

National Scenic Area resources that may be impacted by this project include:

☒ Key Viewing Areas	☐ Cultural Resource	☐ Wetland/Stream/Lake Buffer
☐ Sensitive Wildlife Habitat	☐ Rare Plants	☐ Deer/Elk Wintering Range
☐ Historic Uses/Structures	☐ Natural Area	☐ Adjacent to Recreational Uses

Enclosures

$1545 NSA Em Prm
$159 ntc fee

08/24/2017 3:51PM 000001 #7493
0007 SARAH
PERMITS-TYPE 2 $1545.00
NOTICE/TPR $159.00
CHECK $1704.00

Multnomah County

Land Use Planning Division
1600 SE 190th Ave, Ste 116
Portland OR 97233
Ph: 503-988-3043 Fax: 503-988-3389
multco.us/landuse

NSA Application Form

PROPERTY IDENTIFICATION

Property Address: 2501 NE RASMUSSEN RD CORBETT
State Identification#: 1N 4E 27DA-00200
Site Size: APPROX 17,000 S.F.
A&T Alternate Account Number R#: 177900560

PROPERTY OWNER(S) ☐ OR CONTRACT PURCHASER(S) ☐

Name: Linda K Moles
Mailing Address: 2501 NE Rasmussen Rd.
City: Corbett State: OR Zip Code: 97019 Phone#: 503 3804928

I authorize the applicant below to make this application.

Linda K Moles

Property Owner Signature #1 — Property Owner Signature #2

NOTE: By signing this form, the property owner or property owner's agent is granting permission for Planning Staff to conduct site inspections on the property.

If no owner signature above, a letter of authorization from the owner is required. ☐

APPLICANT'S NAME AND SIGNATURE

Applicant's Name: PETER KAAP FROM PLI SYSTEMS
Mailing Address: 3045 SE 61ST CT.
City: HILLSBORO State: OR Zip Code: 97123 Phone #: 503-649-8111
Fax: 503-649-4621 e-mail: peter@plisystems.com

Applicant's Signature

GENERAL DESCRIPTION OF APPLICATION (REQUIRED)
Please provide a brief description of your project.
CONSTRUCT RETAINING WALL TO REMEDIATE LANDSLIDE

For Staff Use

CASE NUMBER: T2-2017-8783
504976
LAND USE PERMIT(S): NSA Emergency Repair

DATE SUBMITTED: 8-24-2017

Compliance Related ☐

Potential Transportation Impact ☐

PF-2017-7628
PF/PA No.

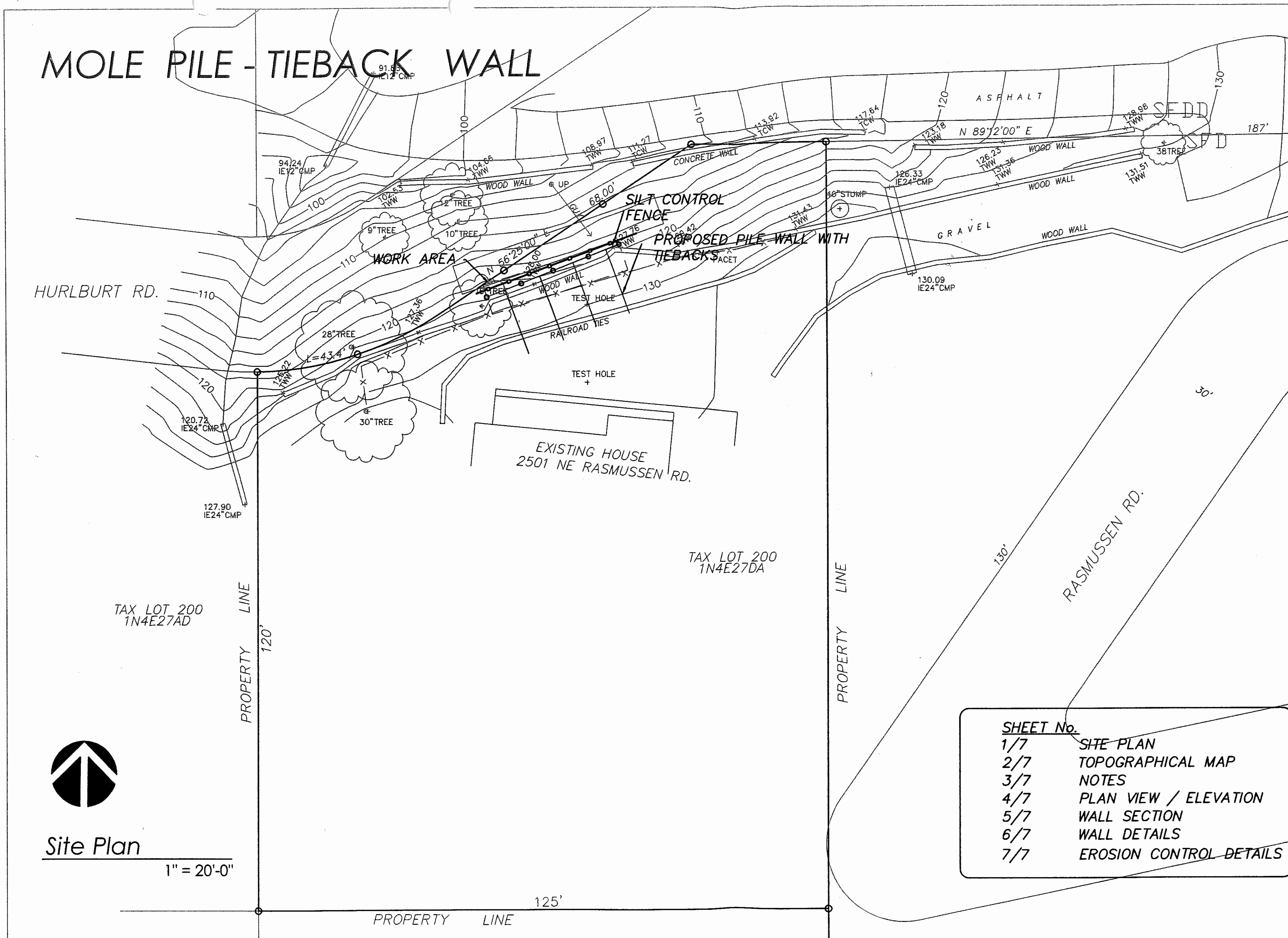
ZONING
CFR-5
Zoning District
HDP
Zoning Overlay.

6

KEY VIEWING AREAS: Check all the following sites from which your property can be seen.

- ☐ Cape Horn
- ☐ Crown Point
- ☐ Larch Mountain
- ☐ Multnomah Falls
- ☐ Columbia River
- ☐ Beacon Rock
- ☐ Historic Columbia River Highway
- ☐ Portland's Women's Forum State Park
- ☐ Highway I-84, including rest stops
- ☐ Rooster Rock State Park
- ☐ Bonneville Dam Visitor Centers
- ☐ Washington State Route 14
- ☐ Sandy River
- ☐ Pacific Crest Trail
- ☐ Larch Mountain Road (SMA only)
- ☐ Sherrard Point on Larch Mountain (if in SMA)

MOLE PILE - TIEBACK WALL



Site Plan

1" = 20'-0"

SHEET No.	
1/7	~~SITE PLAN~~
2/7	TOPOGRAPHICAL MAP
3/7	NOTES
4/7	PLAN VIEW / ELEVATION
5/7	WALL SECTION
6/7	WALL DETAILS
7/7	EROSION CONTROL DETAILS

PLI SYSTEMS
3045 SE 61st COURT
HILLSBORO, OR. 97123
503-649-8111

Mole Pile-Tieback Wall
2501 NE Rasmussen Rd, Corbett, OR 97019.

SHEET TITLE:

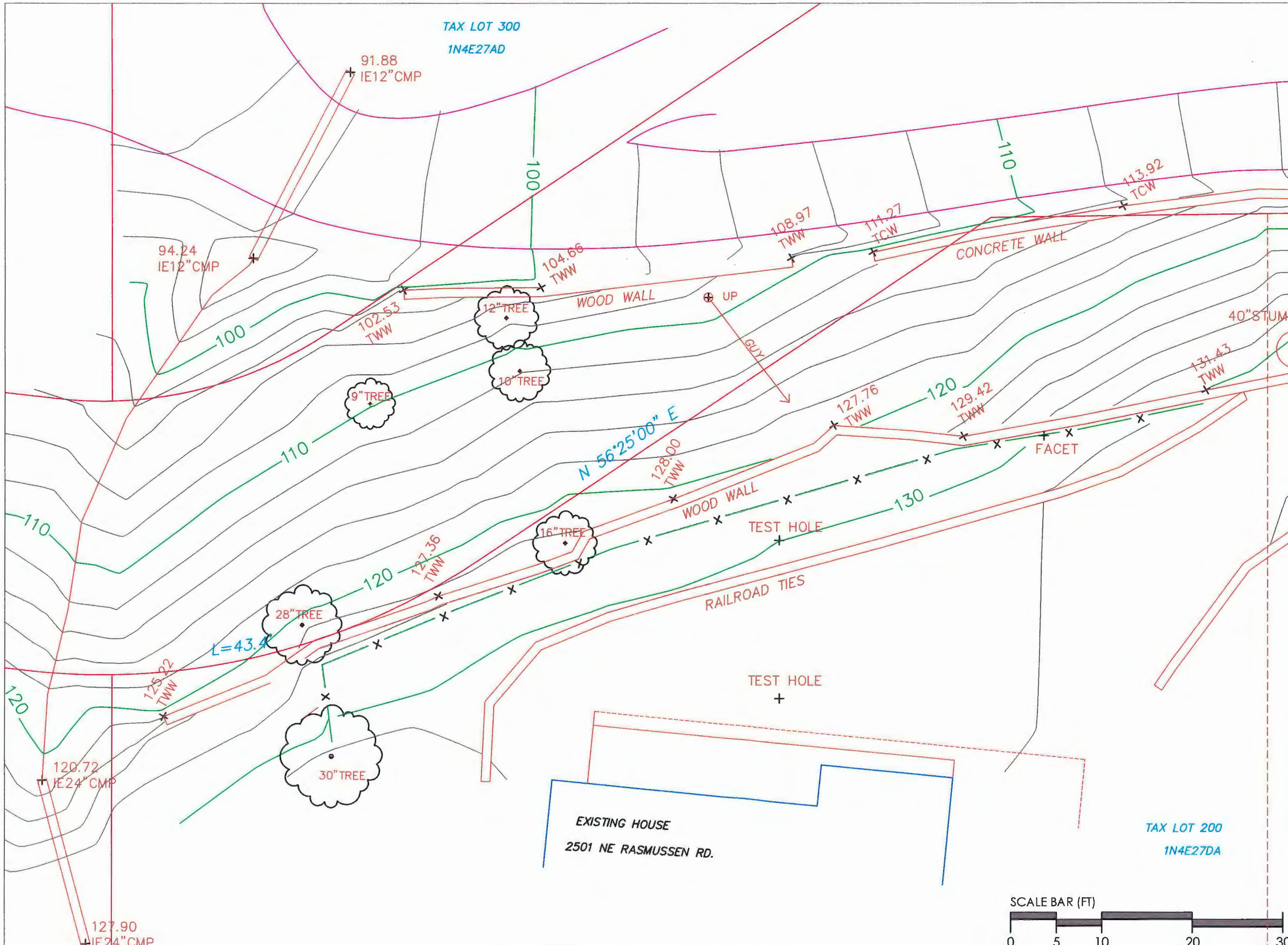
SITE PLAN

REVISIONS:

DATE:	08/24/17
SCALE:	AS SHOWN
DRAWN:	NPR
CHECKED BY:	MC
JOB NO.:	

1 / 7

Ref: Mole Pile & Tieback Wall (08-24-17)



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HILLSBORO, OR. 97123
503-649-8111

PLI SYSTEMS

Mole Pile-Tieback Wall

2501 NE Rasmussend Rd., Corbett OR 97019

SHEET TITLE:

TOPOGRAPHICAL MAP

REVISIONS:

DATE:	08/01/17
SCALE:	AS SHOWN
DRAWN:	NPR
CHECKED BY:	MC
JOB NO.:	

2 / 7

Ref: Mole Pile Wall (08-01-17)

TIE-BACK ANCHORS

1. THE WORK SHALL CONSIST OF CONSTRUCTING PERMANENT ANCHORS, AND INSTALLATION AS SPECIFIED HEREIN AND SHOWN ON THE PLANS.
2. THE TERM "ANCHOR" AS USED IN THESE SPECIFICATIONS IS INTENDED AS A GENERIC TERM AND REFERS TO A REINFORCING BAR GROUTED INTO A DRILLED HOLE INSTALLED IN ANY TYPE OF GROUND. BAR SHALL BE #14 William GRADE 75 BAR.
3. TIEBACK ANCHORS SHALL BE COMPOSED OF METALIZED STEEL WITH THE ADDITION OF PVC COVER TO FORM A NO-LOAD ZONE.
4. GROUT SHALL BE A NEAT OR SAND/CEMENT MIXTURE WITH A MINIMUM 3-DAY COMPRESSIVE STRENGTH OF 1500 PSI AND A MINIMUM 28-DAY COMPRESSIVE STRENGTH OF 5000 PSI PER ASTM C109.
5. CEMENT SHALL BE ASTM C150, TYPE I OR II.
6. FINE AGGREGATE SHALL BE CLEAN, NATURAL SAND, ASTM C33. ARTIFICIAL OR MANUFACTURED SAND IS ACCEPTABLE PROVIDED IT IS SUITABLE FOR PUMPING IN ACCORDANCE WITH ACI 304, 4.2.2.
7. BAR COUPLERS SHALL DEVELOP THE FULL ULTIMATE TENSILE STRENGTH OF THE BAR AS CERTIFIED BY THE MANUFACTURER.
8. BEARING PLATES SHALL BE AASHTO M183/ASTM A36.
9. NUTS SHALL MEET AASHTO M291, GRADE B, HEXAGONAL FITTED WITH BEVELED WASHER OR SPHERICAL SEAT TO PROVIDE UNIFORM BEARING.
10. ANCHORS SHALL BE INSTALLED WITH A GROUT AS YOU DRILL SYSTEM AT THE LOCATIONS AND TO THE LENGTHS INDICATED ON THE PLANS. THE ENGINEER MAY ADD, ELIMINATE, OR RELOCATE ANCHOR DOWELS TO ACCOMODATE ACTUAL FIELD CONDITIONS. MODIFICATIONS TO THE DESIGN RESULTING FROM ACTIONS OF THE CONTRACTOR SHALL BE DETERMINED BY THE ENGINEER.
11. THE CONTRACTOR SHALL IMMEDIATLY SUSPEND DRILLING OPERATIONS IF ADVERSE CONDITIONS ARE OBSERVED, OR IF ADJACENT STRUCTURES ARE DAMAGED AS A RESULT OF THE DRILLING OPERATION. THE ADVERSE CONDITIONS SHALL BE STABILIZED IMMEDIATELY AND THE ENGINEER SHALL BE NOTIFIED OF SUCH CONDITIONS WITHIN 24 HOURS.
12. GROUT EQUIPMENT SHALL PRODUCE A UNIFORMLY MIXED GROUT FREE OF LUMPY AND UNDISPERSED CEMENT. A POSITIVE DISPLACEMENT GROUT PUMP SHALL BE USED.
13. 100% OF PRODUCTION ANCHORS SHALL BE TESTED USING PROOF TESTING PROCEDURES. ALL RECORDED TEST DATA SHALL BE RECORDED BY THE OWNER'S REPRESENTATIVE, OR REQUESTED BY THE ENGINEER. PULLOUT TESTING OF ANCHOR DOWELS SHALL NOT BE PERFORMED UNTIL THE ANCHOR DOWEL GROUT HAS ATTAINED AT LEAST 33 PERCENT OF THEIR SPECIFIED 28-DAY COMPRESSIVE STRENGTHS.

PROOF TEST

AL	125% DTL
0.25 DTL	150% DTL (TEST LOAD)
0.50 DTL	(HOLD FOR CREEP TEST)
0.75 DTL	AL (OPTIONAL)
1.00 DTL	ADJUST TO LOCK-OFF LOAD

14. TESTING EQUIPMENT SHALL INCLUDE TWO DIAL GAUGES, A DIAL GAUGE SUPPORT, JACK AND PRESSURE GAUGE, A PUMP, AND A REACTION FRAME.

* A MINIMUM OF TWO DIAL GAUGES CAPABLE OF MEASURING TO 0.001-INCH SHALL BE AVAILABLE AT THE SITE TO MEASURE THE ANCHOR DOWEL MOVEMENT. THE DIAL GAUGES SHALL HAVE A MINIMUM TRAVEL SUFFICIENT TO ALLOW THE TEST TO BE PERFORMED WITHOUT RE-SETTING THE DIAL GAUGE. THE DIAL GAUGES SHALL BE ALIGNED WITHIN 5 DEGREES OF THE AXIS OF THE ANCHOR DOWEL AND SHALL BE SUPPORTED INDEPENDENT OF THE JACKING SET-UP AND THE WALL. A HIDRAULIC JACK, PRESSURE GAUGE, AND PUMP SHALL BE USED TO APPLY AND MEASURE THE TEST LOAD. THE CONTRACTOR SHALL PROVIDE RECENT CALIBRATION CURVES IN ACCORDANCE WITH THE SUBMITTALS SECTION.
* THE JACK AND PRESSURE GAUGE SHALL BE CALIBRATED BY AN INDEPENDENT TESTING LABORATORY AS A UNIT. THE PRESSURE GAUGE SHALL BE GRADUATED IN 100 PSI INCREMENTS OR LESS AND SHALL HAVE A RANGE NOT EXCEEDING TWICE THE ANTICIPATED MAXIMUM PRESSURE DURING TESTING UNLESS APPROVED OTHERWISE BY THE ENGINEER. THE RAM TRAVEL OF THE JACK SHALL BE SUFFICIENT TO ENABLE THE TEST TO BE PERFORMED WITHOUT RE-SETTING THE JACK. THE JACK SHALL BE CAPABLE OF APPLYING EACH TEST LOAD INCREMENT IN LESS THAN 1 MINUTE.
* THE JACK SHALL BE INDEPENDENTLY SUPPORTED AND CENTERED OVER THE ANCHOR DOWEL SO THAT THE ANCHOR DOWEL DOES NOT CARRY THE WEIGHT OF THE JACK. THE STRESSING EQUIPMENT EQUIPMENT SHALL BE PLACED OVER THE ANCHOR DOWEL IN SUCH A MANNER THAT THE JACK, BEARING PLATES, AND STRESING ANCHORAGE ARE IN ALIGNMENT. THE JACK SHALL BE POSITIONED AT THE BEGINNING OF THE TEST SUCH THAT UNLOADING AND REPOSITIONING OF THE JACK DURING THE TEST WILL NOT BE REQUIRED.
* THE REACTION FRAME SHALL BE SUFFICIENTLY RIGID AND OF ADEQUATE DIMENSION SUCH THAT EXCESSIVE DEFORMATION OF THE TEST APPARATUS REQUIRING REPOSITIONING OF ANY COMPONENTS DOES NOT OCCUR DURING TESTING. WHERE THE REACTION FRAME BEARS DIRECTLY ON THE CONCRETE FACING, THE REACTION FRAME SHALL BE DESIGNED TO PREVENT FRACTURE OF THE CONCRETE.

15. THE ALLOWABLE BAR LOAD DURING TESTING SHALL NOT EXCEED 80 PERCENT OF THE STEEL ULTIMATE STRENGTH FOR GRADE 150 BARS OR 90 PERCENT OF THE YIELD STRENGTH FOR GRADE 60 AND GRADE 75 BARS.

16. DURING THE CREEP TEST HOLD THE TEST LOAD AND TAKE MOVEMENT READINGS AT 0, 1, 2, 3, 4, 5, 6, AND 10 MINUTES. IF THE MOVEMENT BETWEEN 1 MINUTE AND 10 MINUTES EXCEEDS 0.04 INCHES (1 mm.) CONTINUE THE CREEP TEST FOR AN ADDITIONAL 50 MINUTES. TAKE MOVEMENT READINGS AT 20, 30, 40, 50, AND 60 MINUTES.

17. THE ENGINEER SHALL EVALUATE THE RESULTS OF EACH VERIFICATION TEST. INSTALLATION METHODS THAT DO NOT SATISFY THE ANCHOR DOWEL TESTING REQUIREMENTS SHALL BE CONSIDERED INADEQUATE. THE CONTRACTOR SHALL PROPOSE ALTERNATIVE METHODS AND INSTALL REPLACEMENT VERIFICATION TEST ANCHOR DOWELS. REPLACEMENT TEST ANCHOR DOWELS SHALL BE INSTALLED AND TESTED AT THE OWNER'S EXPENSE.

18. ANCHORS THAT ENCOUNTER UNANTICIPATED OBSTRUCTIONS DURING DRILLING SHALL BE RELOCATED BY THE ENGINEER AT THE OWNER'S COST. ANCHORS THAT DO NOT SATISFY THE SPECIFIED TOLERANCES DUE TO THE CONTRACTOR'S INSTALLATION METHODS SHALL BE REPLACED TO THE ENGINEER'S SATISFACTION AT NO ADDITIONAL COST TO THE OWNER.

19. SPECIAL INSPECTION IS REQUIRED IN ACCORDANCE WITH IBC SECTION 1704 ON THE FOLLOWING ITEMS:
- REINFORCING STEEL PLACEMENT
- CONCRETE CONSTRUCTION
- TIEBACK ANCHORS
- STRUCTURAL WELDING

METALS

* ALL STRUCTURAL AND MISCELLANEOUS STEEL: ASTM A36 (FY= 36,000 PSI) UNLESS NOTED OTHERWISE.
* ALL HP & WF STEEL SHAPES: ASTM A572, GRADE 50 (Fy= 50,000 PSI)
* HSS & TS SHAPES: ASTM A500, GRADE "B" (Fy= 46,000 PSI)
* LIGHT GAGE MATERIAL: ASTM A570 (Fy= 30,000 PSI) UNLESS NOTED OTHERWISE.
* ALL BOLTS: ASTM A307 UNLESS NOTED OTHERWISE.
* WELDING: PER AWS STANDARDS. E70XX ELECTRODE AND BY AWS QUALIFIED WELDERS.
* DESIGN, FABRICATION, AND ERECTION SHALL BE IN ACCORDANCE WITH THE "AISC SPECIFICATION FOR THE DESIGN, FABRICATION AND ERECTION OF STRUCTURAL STEEL FOR BUILDINGS".

WELDING SHALL CONFORM TO THE AWS CODES FOR ARC AND GAS WELDING IN BUILDING CONSTRUCTION AND SHALL BE 3/16" MINIMUM UNLESS OTHERWISE NOTED. WELDING SHALL BE BY AWS CERTIFIED WELDERS.
PREQUALIFIED WELDING PROCEDURES ARE TO BE USED, UNLESS AWS QUALIFICATION IS SUBMITTED TO THE ARCHITECT/ENGINEER PRIOR TO FABRICATION.

PLI SYSTEMS
3045 SE 61st COURT
HILLSBORO, OR. 97123
503-649-8111

MOLE PILE-TIEBACK WALL
2501 NE Rasmussend Rd, Corbett, OR 97019.

SHEET TITLE:

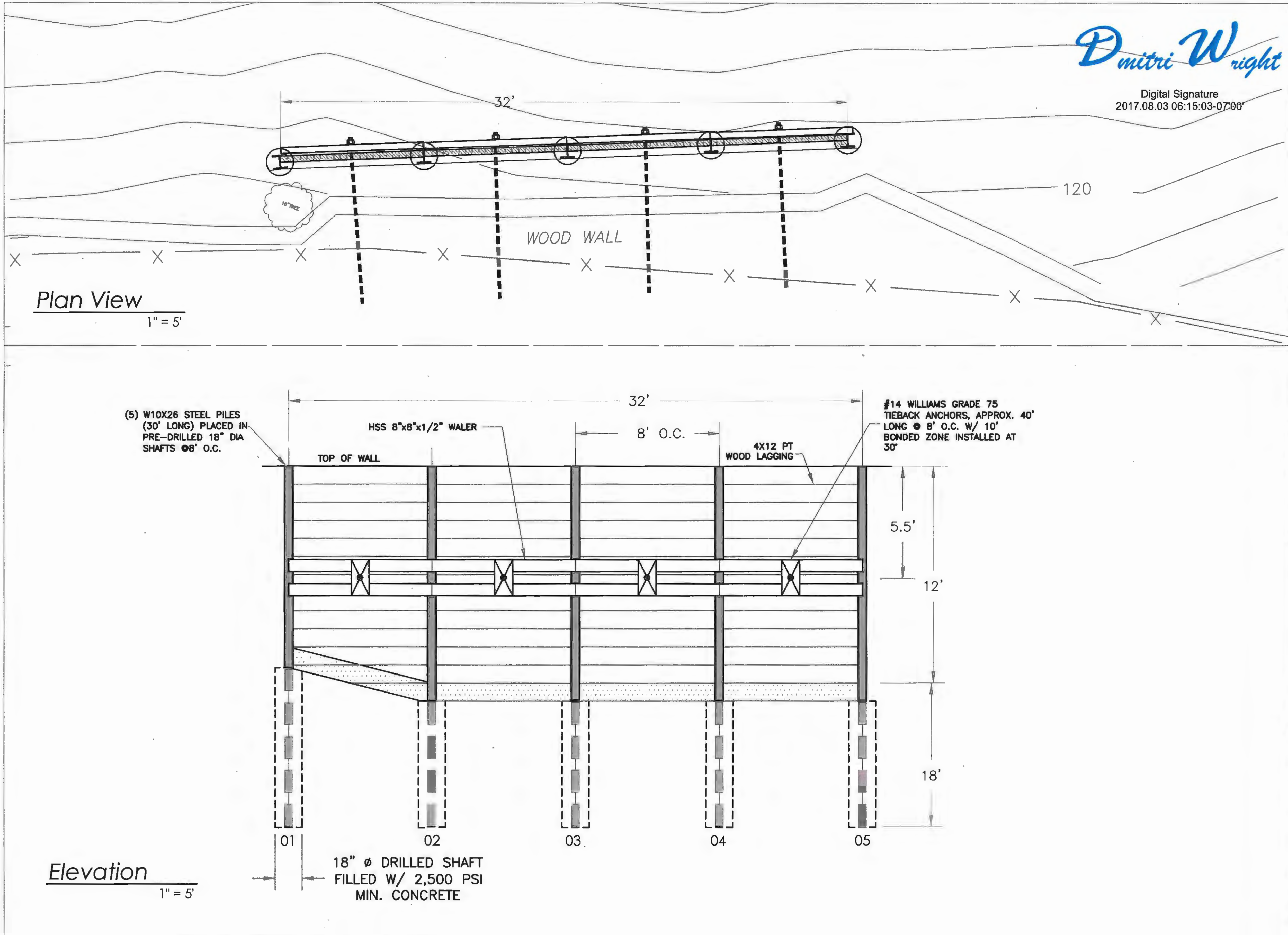
NOTES

REVISIONS:

DATE:	08/01/17
SCALE:	AS SHOWN
DRAWN:	NPR
CHECKED BY:	MC
JOB NO.:	

3 / 7

Ref: Mole Pile & Tieback Wall (08-01-17)



STRUCTURAL REGISTERED PROFESSIONAL ENGINEER
58926 PE
OREGON
JULY 9, 2001
DMITRI S. WRIGHT
EXPIRES: 6/30/18

3045 SE 61st COURT
HILLSBORO, OR. 97123
503-649-8111
PLI SYSTEMS

MOLE PILE-TIEBACK WALL

2501 NE Rasmussend Rd, Corbett, OR 97019.

SHEET TITLE:

PLAN VIEW / WALL ELEVATION

REVISIONS:

DATE:	06/21/17
SCALE:	AS SHOWN
DRAWN:	NPR
CHECKED BY:	MC
JOB NO.:	

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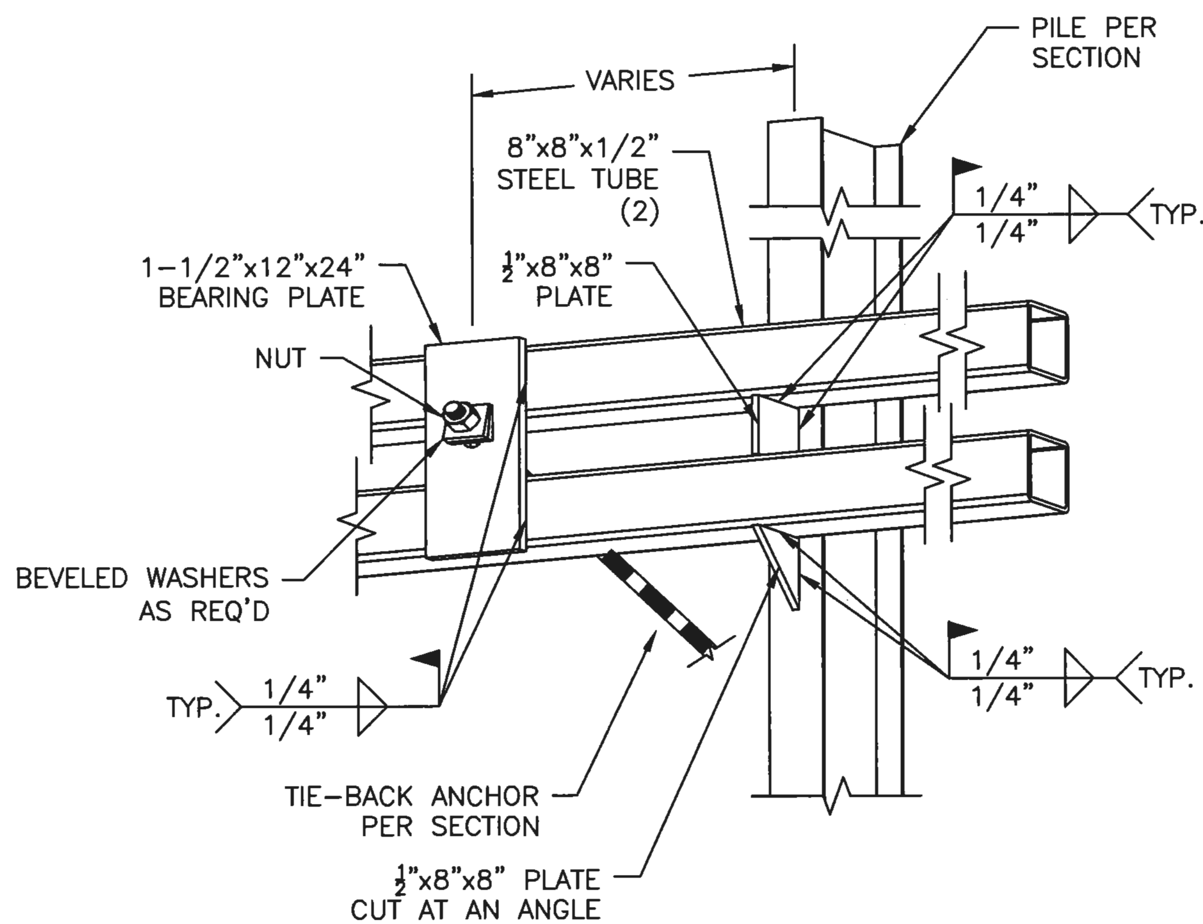
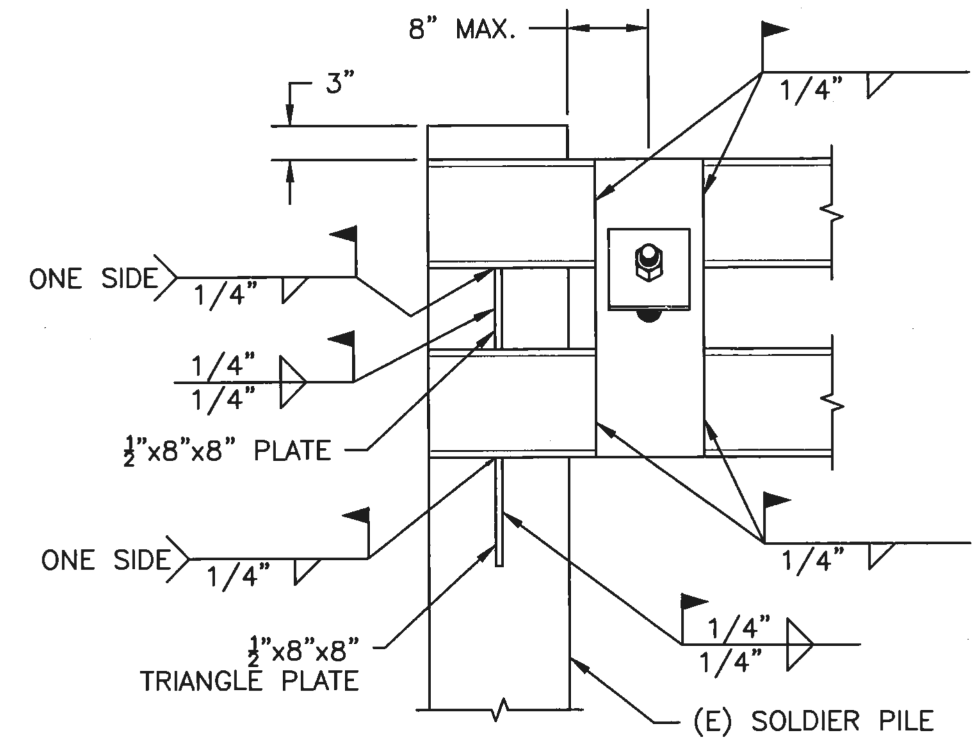
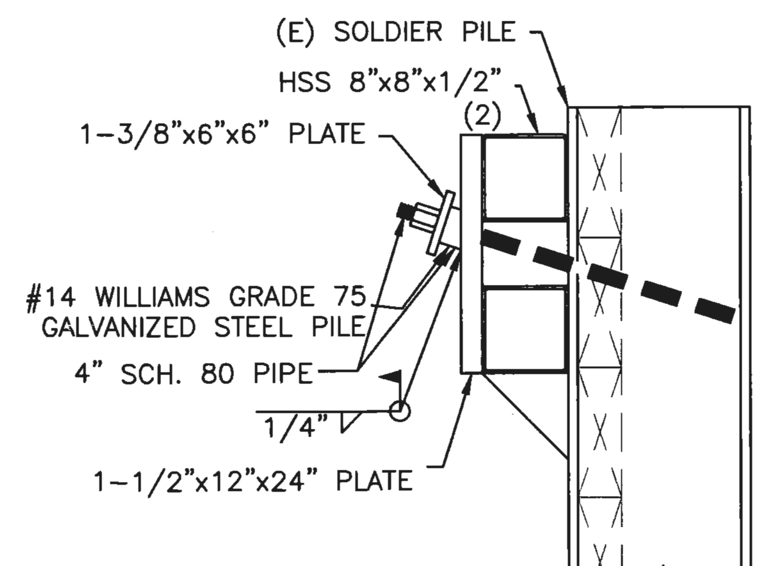
Ref: Mole Linda Pile & Tieback Wall (06-21-17)



DETAIL AT WALER
N.T.S.



DETAIL
N.T.S.



Section Typ.
1/2" = 1'

Dmitri Wright
Digital Signature
2017.08.03 06:14:10-07'00'

REGISTERED PROFESSIONAL ENGINEER
STRUCTURAL
58926 PE
OREGON
JULY 9, 2001
DMITRI S. WRIGHT
EXPIRES: 6/30/18

PLI SYSTEMS
3045 SE 61st COURT
HILLSBORO, OR. 97123
503-649-8111

MOLE PILE-TIEBACK WALL
2501 NE Rasmussend Rd, Corbett, OR 97019.

SHEET TITLE:

WALL DETAIL

REVISIONS:
00/00/17

DATE:	06/21/17
SCALE:	AS SHOWN
DRAWN:	SMW / NPR
CHECKED BY:	MC
JOB NO.:	

6 / 7

Ref: Mole Pile-Tieback wall (06-21-17)

CASCADE
ENGINEERING, INC
209 NE Lincoln Street, Suite. A
Hillsboro, OR 97124-3043
503-846-1131

Calculations For

Mole Linda Pile Wall
2501 NE Rasmussen Road
Corbett, Oregon 97019

Job No: 17143
Client: PLI Systems
8/2/2017

Index to Calculations

Item	Sheet
Design Criteria and Loads	D1 - D2
Steel Element Calculations	C1 - C10
Steel Connections	SC1 - SC3

STRUCTURAL
REGISTERED PROFESSIONAL
ENGINEER
58926 PE
OREGON
JULY 9, 2001
DMITRI S. WRIGHT
EXPIRES: 6/30/18

Dmitri Wright
Digital Signature
2017.08.03 06:16:25-07'00'

CASCADE ENGINEERING, INC
209 NE Lincoln Street, Suite. A
Hillsboro, OR 97124-3043
503-846-1131

Client	PLI Systems
Project	Mole Linda Pile Wall
Location	2501 NE Rasmussen Road, Corbett, OR

STRUCTURAL DESIGN CRITERIA

Location: City of Corbett, OR
Latitude: 45.540 Longitude: -122.292 Elevation: 150 ft

Occupancy: Residence Risk Category II

Applicable Codes:
2014 Oregon Structural Specialty Code
2012 International Building Code
ASCE7-10 Minimum Design Loads for Buildings and Other Structures
NDS-12 National Design Specification for Wood Construction
ACI 318-10 Building Code Requirements for Structural Concrete

Project Narrative:

Structural design of soldier piles, waler, and tieback wall elements.

Design Loads:

Design Loads Used are from the Goetechnical Report EEI Report No. **17-028-1** issued by Earth Engineers Inc. dated March 27, 2017, and sealed by Troy Hall, PE, GE
All driving loads are assummed to be working level, and no factors were applied for ASD design.

CASCADE
ENGINEERING, INC
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Client PLI Systems
Project Mole Linda Pile Wall
Location 2501 NE Rasmussen Road, Corbett, OR

MATERIALS SPECIFICATIONS

Concrete:

	fc' psi
Foundations, slabs:	2500

Structural Steel:

MATERIALS:		Fy psi	Fu psi
Structural Shapes	A992 (Seismic Load Resisting System)	50,000	
Structural Tubes	A500 Gr. B	46,000	58,000
Structural Pipe	A53 Gr. B	35,000	60,000
Structural Plates	A36	36,000	58,000
Bolts:	F1554 Grade 36	36,000	58,000
Welding:	E70 XX Electrodes	60,000	72,000

(All welding to be in conformance with AWS D1.1 and requires special inspection.)

Lumber: (NDS-12)	Fb psi	Fc psi	Fv psi	E psi
Lagging: Hem Fir No. 2, Preservative Treated	850	1300	150	1.30E+06

Statement of Special Inspections:

ITEM	DURATION	INSP. AGENCY
STRUCTURAL WELDING & HIGH-STRENGTH BOLTING:		
SINGLE PASS FILLET WELDS 5/16" AND SMALLER	PERIODIC	TESTING LAB
MULTIPASS OR FILLET WELDS LARGER THAN 5/16"	CONTINUOUS	TESTING LAB
PLUG AND SLOT WELDS	CONTINUOUS	TESTING LAB

CASCADE ENGINEERING, INC
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Client	PLI Systems
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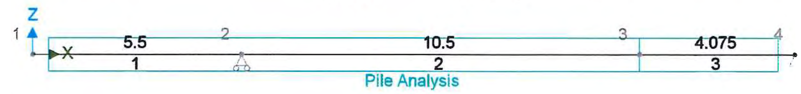
Location	2501 NE Rasmussen Road, Corbett, OR

PILE ANALYSIS 2.0.REVISED

ANALYSIS

Tedds calculation version 1.0.21

Geometry



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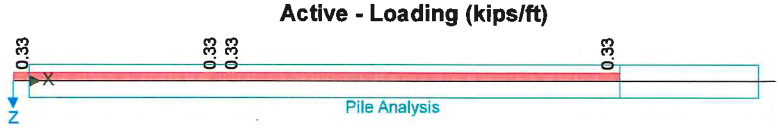

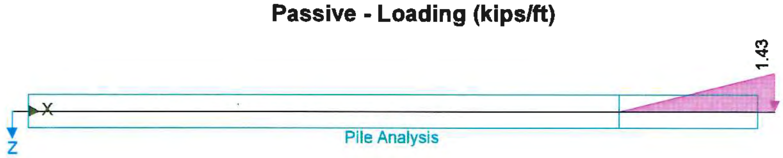

Load combination factors

Member Loads

Member	Load case	Load Type	Orientation	Description
Pile Analysis	Active	UDL	GlobalZ	0.33 kips/ft at 0 ft to 16 ft
Pile Analysis	Seismic	VDL	GlobalZ	0.14 kips/ft at 0 ft to 0.03 kips/ft at 16 ft
Pile Analysis	Passive	VDL	GlobalZ	0 kips/ft at 16 ft to 1.43 kips/ft at 20.075 ft

Results

Reactions

Load combination: Pile Loading (Service)

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
2	0	44.455	0
4	0	0.012	0

CASCADE ENGINEERING, INC
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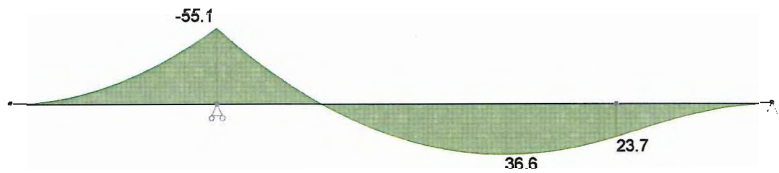
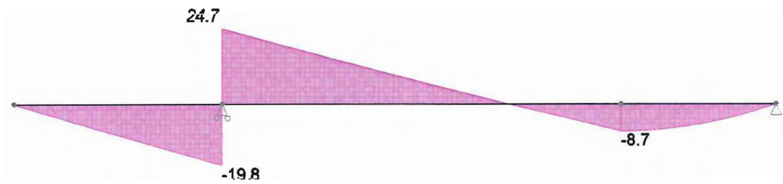
Client PLI Systems
Project Mole Linda Pile Wall
Location 2501 NE Rasmussen Road, Corbett, OR

Forces

All combinations - Moment envelope (kip_ft)



All combinations - Shear envelope (kips)



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STEEL PILE DESIGN (AISC360).REVISED

STEEL MEMBER DESIGN (AISC 360)

In accordance with AISC360 14th Edition published 2010 using the ASD method

Tedds calculation version 4.2.01

Safety factors

Shear; Ω_v = **1.50**
Flexure; Ω_b = **1.67**
Tensile yielding; $\Omega_{t,y}$ = **1.67**
Tensile rupture; $\Omega_{t,r}$ = **2.00**
Compression; Ω_c = **1.67**

Design section 1

Section details

Section type; W 10x26 (AISC 14th Edn (v14.1))
ASTM steel designation; A992
Steel yield stress; F_y = **50** ksi
Steel tensile stress; F_u = **65** ksi
Modulus of elasticity; E = **29000** ksi

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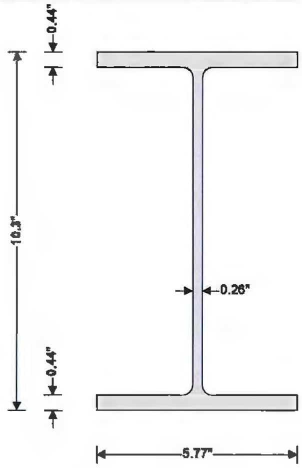
Location	2501 NE Rasmussen Road, Corbett, OR



W 10x26 (AISC 14th Edn (v14.1))
Section depth, d, 10.3 in
Section breadth, b_f, 5.77 in
Weight of section, Weight, 26 lbf/ft
Flange thickness, t_f, 0.44 in
Web thickness, t_w, 0.26 in
Area of section, A, 7.6 in^2
Radius of gyration about x-axis, r_x, 4.35 in
Radius of gyration about y-axis, r_y, 1.36 in
Elastic section modulus about x-axis, S_x, 27.9 in^3
Elastic section modulus about y-axis, S_y, 4.89 in^3
Plastic section modulus about x-axis, Z_x, 31.3 in^3
Plastic section modulus about y-axis, Z_y, 7.5 in^3
Second moment of area about x-axis, I_x, 144 in^4
Second moment of area about y-axis, I_y, 14.1 in^4

Analysis results

Required flexural strength - Major axis;	$M_{r,x}$ = **55.1** kips_ft	
Required shear strength - Major axis;	$V_{r,x}$ = **24.96** kips	
Required compressive strength;	P_r = **25.7** kips	

Restraint spacing

Major axis lateral restraint;	L_x = **16** ft	
Minor axis lateral restraint;	L_y = **8** ft	
Torsional restraint;	L_z = **16** ft	

Classification of sections for local buckling - Section B4

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio;	$b_f / (2 \times t_f)$ = **6.56**	
Limiting ratio for compact section;	$\lambda_{pff} = 0.38 \times \sqrt{[E / F_y]}$ = **9.15**	
Limiting ratio for non-compact section;	$\lambda_{rff} = 1.0 \times \sqrt{[E / F_y]}$ = **24.08**;	Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio;	$(d - 2 \times k) / t_w$ = **33.92**	
Limiting ratio for compact section;	$\lambda_{pwf} = 3.76 \times \sqrt{[E / F_y]}$ = **90.55**	
Limiting ratio for non-compact section;	$\lambda_{rwf} = 5.70 \times \sqrt{[E / F_y]}$ = **137.27**;	Compact

Section is compact in flexure

Classification of flanges in uniform compression - Table B4.1a (case 1)

Width to thickness ratio;	$b_f / (2 \times t_f)$ = **6.56**	
Limiting ratio for non-compact section;	$\lambda_{rfc} = 0.56 \times \sqrt{[E / F_y]}$ = **13.49**;	Nonslender

Classification of web in uniform compression - Table B4.1a (case 5)

Width to thickness ratio;	$(d - 2 \times k) / t_w$ = **33.92**	
Limiting ratio for non-compact section;	$\lambda_{rwc} = 1.49 \times \sqrt{[E / F_y]}$ = **35.88**;	Nonslender

Section is nonslender in compression

Design of members for compression - Chapter E

Required compressive strength;	P_r = **25.7** kips	

Slenderness limitations and effective length - Section E2

Unbraced length; $L_{b,x}$ = **16** ft

Effective length factor; K_x = **1.00**

Column slenderness; $\lambda_x = K_x \times L_{b,x} / r_x$ = **44.138**

Major axis column slenderness ratio does not exceed recommended limit of 200

Slenderness limitations and effective length - Section E2

Unbraced length; $L_{b,y}$ = **8** ft

Effective length factor; K_y = **1.00**

Column slenderness; $\lambda_y = K_y \times L_{b,y} / r_y$ = **70.588**

Minor axis column slenderness ratio does not exceed recommended limit of 200

Flexural buckling of members without slender elements - Section E3

Elastic critical buckling stress - eq E3-4; $F_{e,x} = \pi^2 \times E / (K_x \times L_{b,x} / r_x)^2$ = **146.9** ksi

Flexural buckling stress - eq E3-2; $F_{cr,x} = [0.658^{F_y / F_{e,x}}] \times F_y$ = **43.4** ksi

Nominal compressive strength for flexural buckling - eq E3-1;

$P_{n,fb,x} = F_{cr,x} \times A$ = **330** kips

Elastic critical buckling stress - eq E3-4; $F_{e,y} = \pi^2 \times E / (K_y \times L_{b,y} / r_y)^2$ = **57.4** ksi

Flexural buckling stress - eq E3-2; $F_{cr,y} = [0.658^{F_y / F_{e,y}}] \times F_y$ = **34.7** ksi

Nominal compressive strength for flexural buckling - eq E3-1;

$P_{n,fb,y} = F_{cr,y} \times A$ = **264.3** kips

Torsional and torsional-flexural buckling of members without slender elements - Section E4

Unbraced length; $L_{b,z}$ = **16** ft

Effective length factor; K_z = **1.00**

Flexural-torsional elastic buckling stress - eq E4-4; $F_e = [\pi^2 \times E \times C_w / (K_z \times L_{b,z})^2 + G \times J] / (I_x + I_y)$ = **45.3** ksi

Flexural-torsional buckling stress - eq E3-2; $F_{cr} = [0.658^{F_y / F_e}] \times F_y$ = **31.5** ksi

Nominal compressive strength for torsional and flexural-torsional buckling - eq E4-1;

$P_{n,ftb} = F_{cr} \times A$ = **239.7** kips

Allowable compressive strength - E1

Nominal compressive strength; $P_n = \min(P_{n,fb,x}, P_{n,fb,y}, P_{n,ftb})$ = **239.7** kips

Allowable compressive strength; $P_c = P_n / \Omega_c$ = **143.6** kips

P_r / P_c = **0.179**

PASS - Nominal compressive strength exceeds required compressive strength

Design of members for shear - Chapter G

Required shear strength; $V_{r,x}$ = **25** kips

Web area; $A_w = d \times t_w$ = **2.678** in^2

Web plate buckling coefficient; k_v = **5**

$(d - 2 \times k) / t_w <= 2.24 \times \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-2; C_v = **1.000**

Nominal shear strength - eq G2-1; $V_{n,x} = 0.6 \times F_y \times A_w \times C_v$ = **80.3** kips

Allowable shear strength; $V_{c,x} = V_{n,x} / \Omega_v$ = **53.6** kips

$V_{r,x} / V_{c,x}$ = **0.466**

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength; $M_{r,x}$ = **55.1** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1; $M_{n,yld,x} = M_{p,x} = F_y \times Z_x$ = **130.4** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length; $L_b = L_{y\_s1}$ = **8** ft

Limiting unbraced length for yielding - eq F2-5; $L_p = 1.76 \times r_y \times \sqrt{(E / F_y)}$ = **4.804** ft

Distance between flange centroids; h_o = **9.86** in

c = **1**

r_{ts} = **1.58** in

Limiting unbraced length for inelastic LTB - eq F2-6; $L_r = 1.95 \times r_{ts} \times E / (0.7 \times F_y) \times \sqrt{((J \times c / (S_x \times h_o)) + \sqrt{((J \times c / (S_x \times h_o))^2 + 6.76 \times (0.7 \times F_y / E)^2))}}$ = **14.926** ft

LTB modification factor; C_b = **1.000**

Nominal flexural strength for lateral-torsional buckling - eq F2-2

$M_{n,ltb,x} = \min(C_b \times (M_{p,x} - (M_{p,x} - 0.7 \times F_y \times S_x) \times (L_b - L_p) / (L_r - L_p)), M_{p,x})$ = **114.9** kips_ft

Allowable flexural strength - F1

Nominal flexural strength; $M_{n,x} = \min(M_{n,yld,x}, M_{n,ltb,x})$ = **114.9** kips_ft

Allowable flexural strength; $M_{c,x} = M_{n,x} / \Omega_b$ = **68.8** kips_ft

$M_{r,x} / M_{c,x}$ = **0.801**

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for combined forces - Chapter H

Combined flexure and axial force - eq H1-1b; $P_r / (2 \times P_c) + M_{r,x} / M_{c,x}$ = **0.890**

PASS - Combined flexure and axial force is within acceptable limits

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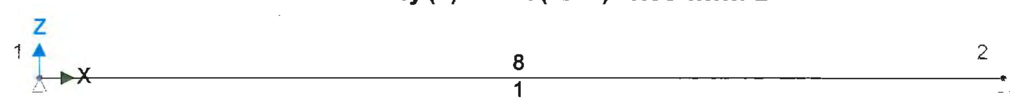
WALER ANALYSIS (T1-T2)

ANALYSIS

Tedds calculation version 1.0.21

Geometry

Geometry (ft) - Steel (AISC) - HSS 8x8x1/2



Loading

Self weight included

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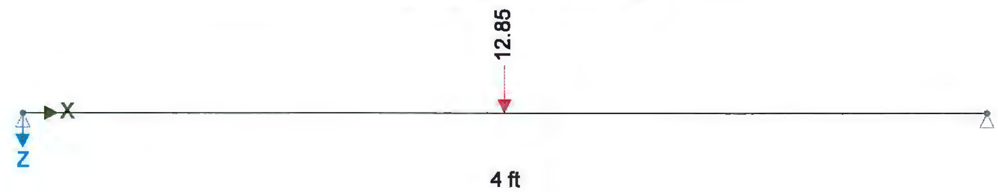
Client PLI Systems
Project Mole Linda Pile Wall
Location 2501 NE Rasmussen Road, Corbett, OR

Earth - Loading (kips)



Results

Reactions

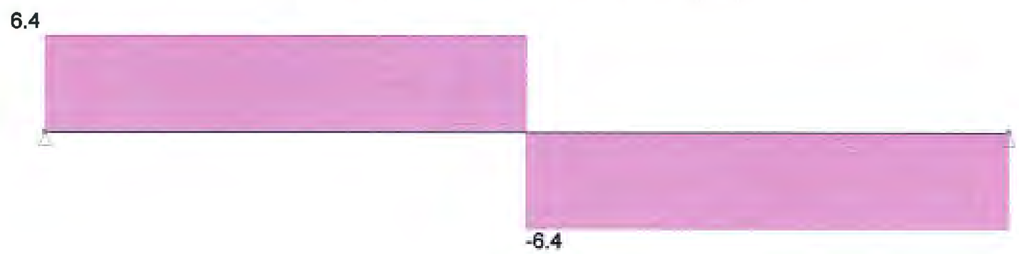
Load combination: 1.0D (Service)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	6.425	0
2	0	6.425	0

Forces

Service combinations - Moment envelope (kip_ft)

Service combinations - Shear envelope (kips)



Service combinations - Deflection envelope (in)

Job # 17143 By JJB Date 7/26/2017 Sheet # C6

Element results

Envelope - Service combinations

Element	Position (ft)	Shear force (kips)		Moment (kip_ft)	
1	0	6.425 (max abs)		0 (min)	
	4	6.425 (max abs)	-6.425	25.7 (max)	

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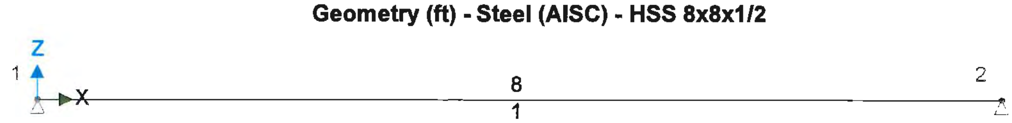
WALER ANALYSIS (T1-T2) MINOR

ANALYSIS

Tedds calculation version 1.0.21

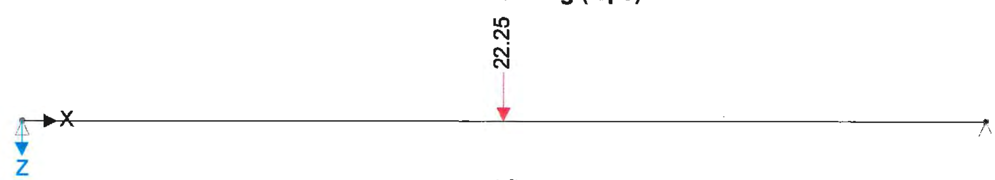
Geometry

Geometry (ft) - Steel (AISC) - HSS 8x8x1/2



Loading

Earth - Loading (kips)



Results

Reactions

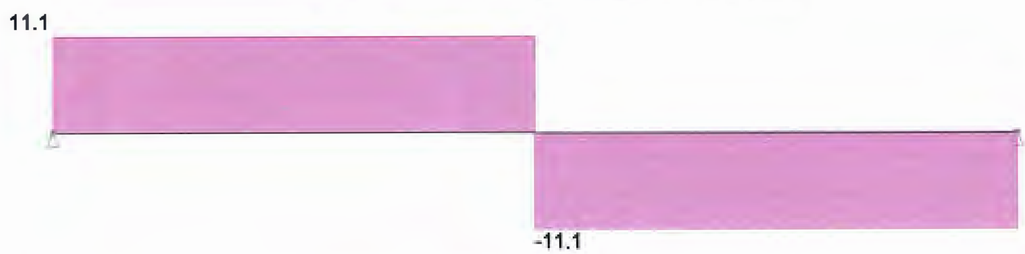
Load combination: 1.0D (Service)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	11.125	0
2	0	11.125	0

Forces

Service combinations - Moment envelope (kip_ft)

Service combinations - Shear envelope (kips)



Service combinations - Deflection envelope (in)

Element results

Envelope - Service combinations

Element	Position (ft)	Shear force (kips)		Moment (kip_ft)	
1	0	11.125 (max abs)		0	
	4	11.125 (max abs)	-11.125	44.5 (max)	

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WALER DESIGN (AISC360) 2.0

STEEL MEMBER DESIGN (AISC 360)

In accordance with AISC360 14th Edition published 2010 using the ASD method

Tedds calculation version 4.2.01

Safety factors

Shear; Ω_v = **1.67**

Flexure; Ω_b = **1.67**

Tensile yielding; $\Omega_{t,y}$ = **1.67**

Tensile rupture; $\Omega_{t,r}$ = **2.00**

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Compression; Ω_c = **1.67**

Design section 1

Section details

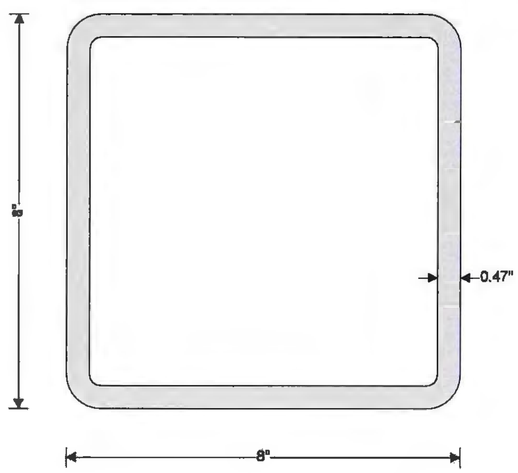
Section type; HSS 8x8x1/2 (AISC 14th Edn (v14.1))

ASTM steel designation; A500 Gr.B

Steel yield stress; F_y = **46** ksi

Steel tensile stress; F_u = **58** ksi

Modulus of elasticity; E = **29000** ksi



HSS 8x8x1/2 (AISC 14th Edn (v14.1))
Section depth, d, 8 in
Section breadth, b_f, 8 in
Weight of section, Weight, 48.9 lbf/ft
Section thickness, t, 0.465 in
Area of section, A, 13.5 in^2
Radius of gyration about x-axis, r_x, 3.04 in
Radius of gyration about y-axis, r_y, 3.04 in
Elastic section modulus about x-axis, S_x, 31.2 in^3
Elastic section modulus about y-axis, S_y, 31.2 in^3
Plastic section modulus about x-axis, Z_x, 37.5 in^3
Plastic section modulus about y-axis, Z_y, 37.5 in^3
Second moment of area about x-axis, I_x, 125 in^4
Second moment of area about y-axis, I_y, 125 in^4

Analysis results

Required flexural strength - Major axis; $M_{r,x}$ = **25.7** kips_ft

Required flexural strength - Minor axis; $M_{r,y}$ = **44.5** kips_ft

Required shear strength - Major axis; $V_{r,x}$ = **6.4** kips

Required shear strength - Minor axis; $V_{r,y}$ = **11.1** kips

Restraint spacing

Major axis lateral restraint; L_x = **8** ft

Minor axis lateral restraint; L_y = **8** ft

Torsional restraint; L_z = **8** ft

Classification of sections for local buckling - Section B4

Classification of flanges in flexure - Table B4.1b (case 17)

Width to thickness ratio; $\max(d - 3 \times t, b_f - 3 \times t) / t$ = **14.20**

Limiting ratio for compact section; $\lambda_{pff} = 1.12 \times \sqrt{[E / F_y]}$ = **28.12**

Limiting ratio for non-compact section; $\lambda_{rff} = 1.40 \times \sqrt{[E / F_y]}$ = **35.15**; Compact

Classification of web in flexure - Table B4.1b (case 19)

Width to thickness ratio; $\max(d - 3 \times t, b_f - 3 \times t) / t$ = **14.20**

Limiting ratio for compact section; $\lambda_{pwf} = 2.42 \times \sqrt{[E / F_y]}$ = **60.76**

Limiting ratio for non-compact section; $\lambda_{rwf} = 5.70 \times \sqrt{[E / F_y]}$ = **143.12**; Compact

Section is compact in flexure

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Design of members for shear - Chapter G

Required shear strength;	$V_{r,x}$ = **6.4** kips
Web area;	$A_w = 2 \times (d - 3 \times t) \times t$ = **6.143** in^2
Web plate buckling coefficient;	k_v = **1.2**
	$(d - 3 \times t) / t <= 1.10 \times \sqrt{(k_v \times E / F_y)}$
Web shear coefficient - eq G2-3;	C_v = **1.000**
Nominal shear strength - eq G2-1;	$V_{n,x} = 0.6 \times F_y \times A_w \times C_v$ = **169.5** kips
Allowable shear strength;	$V_{c,x} = V_{n,x} / \Omega_v$ = **101.5** kips
	$V_{r,x} / V_{c,x}$ = **0.063**
	PASS - Allowable shear strength exceeds required shear strength
Required shear strength;	$V_{r,y}$ = **11.1** kips
Web area;	$A_w = 2 \times (b_f - 3 \times t) \times t$ = **6.143** in^2
Web plate buckling coefficient;	k_v = **1.2**
	$(b_f - 3 \times t) / t <= 1.10 \times \sqrt{(k_v \times E / F_y)}$
Web shear coefficient - eq G2-3;	C_v = **1.000**
Nominal shear strength - eq G2-1;	$V_{n,y} = 0.6 \times F_y \times A_w \times C_v$ = **169.5** kips
Allowable shear strength;	$V_{c,y} = V_{n,y} / \Omega_v$ = **101.5** kips
	$V_{r,y} / V_{c,y}$ = **0.109**
	PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength;	$M_{r,x}$ = **25.7** kips_ft
Yielding - Section F7.1	
Nominal flexural strength for yielding - eq F7-1;	$M_{n,yld,x} = M_{p,x} = F_y \times Z_x$ = **143.7** kips_ft
Allowable flexural strength - F1	
Nominal flexural strength;	$M_{n,x} = M_{n,yld,x}$ = **143.7** kips_ft
Allowable flexural strength;	$M_{c,x} = M_{n,x} / \Omega_b$ = **86.1** kips_ft
	$M_{r,x} / M_{c,x}$ = **0.299**
	PASS - Allowable flexural strength exceeds required flexural strength

Design of members for flexure - Chapter F

Required flexural strength;	$M_{r,y}$ = **44.5** kips_ft
Yielding - Section F7.1	
Nominal flexural strength for yielding - eq F7-1;	$M_{n,yld,y} = M_{p,y} = F_y \times Z_y$ = **143.7** kips_ft
Allowable flexural strength - F1	
Nominal flexural strength;	$M_{n,y} = M_{n,yld,y}$ = **143.7** kips_ft
Allowable flexural strength;	$M_{c,y} = M_{n,y} / \Omega_b$ = **86.1** kips_ft
	$M_{r,y} / M_{c,y}$ = **0.517**
	PASS - Allowable flexural strength exceeds required flexural strength

Design of members for combined forces - Chapter H

Combined flexure and axial force - eq H1-1b;	$M_{r,x} / M_{c,x} + M_{r,y} / M_{c,y}$ = **0.816**
	PASS - Combined flexure and axial force is within acceptable limits

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Client	PLI Systems
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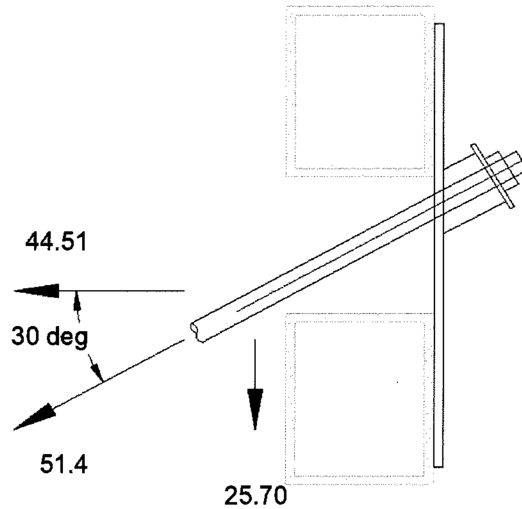
Tieback Connection to Waler

Tieback Tension **T** (k)	51.4
Angle with Horizontal (deg)	30
Horizontal Component **Th** (k)	44.51
Vertical Component **Tv** (k)	25.70



<u>Check Bending in Head Bearing Plate</u>

Point Load on bearing plate (k)	51.40
Plate span for bending **a** (in)	4
Bending in plate **Mpl** (k-in)	51.400
Steel plate yield strength **Fy** (ksi)	36
Plate thickness **t** (in)	1.375
Plate width **b** (in)	6
Safety Factor Ω (ASD)	1.67
Plate Bending Strength (AISC Eq. F11-1)	
Mp=1/Ω*Fy*Zx ≤ 1.6Fy*Sx (k-in)	61.256
Stress Ratio:	0.84

<u>Use 1-3/8" thick x 6" x 6" Head Bearing Plate</u>

<u>Check Fillet Welds:</u>	Ring	Plate	
Load on Connection (k)	25.70	12.85	
Type of Weld	Fillet Weld	Fillet Weld	
Nominal Tensile, **Fexx** (ksi)	70	70	
Thinnest Welded Part (in)	0.3125	0.3125	
Minimum Nominal Weld Size (in)	0.1875	0.1875	
Maximum Nominal Weld Size (in)	NA	1.3125	
Nominal Weld Leg Size (in)	0.25	0.25	
Effective Throat t_E (in)	0.177	0.177	
Weld Length **Lw** (in)	14.13717	12	
Effective Weld Area **Aw** (in2)	0.177	0.177	
Safety Factor Ω (ASD)	2	2	
Allowable Stress **Fw=1/Ω*0.6Fexx** (ksi)	21	21	*AISC Table J2.5*
Allowable Unit Load **Fw*Aw** (k/in)	3.712	3.712	*Eq. J2-3*
Maximum Applied Unit Load (k/in)	1.818	1.071	
Stress Ratio:	0.49	0.29	

<u>Use 1/4" fillet weld all around on bearing ring</u>
<u>Use (2) 4" long min. 1/4" fillet welds bearing plate to waler</u>

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Client	PLI Systems
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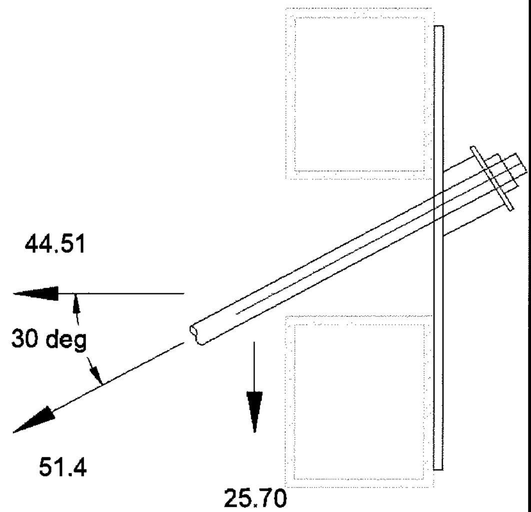
Tieback Connection to Waler

Tieback Tension **T** (k)	51.4
Angle with Horizontal (deg)	30
Horizontal Component **Th** (k)	44.51
Vertical Component **Tv** (k)	25.70



<u>Check Bending in Waler Bearing Plate</u>	
Point Load on bearing plate (k)	44.51
Plate span for bending **a** (in)	8
Bending in plate **Mpl** (k-in)	89.027
Steel plate yield strength **Fy** (ksi)	36
Plate thickness **t** (in)	1.5
Plate width **b** (in)	12
Safety Factor Ω (ASD)	1.67
Plate Bending Strength (AISC Eq. F11-1)	
Mp=1/Ω*Fy*Zx ≤ 1.6Fy*Sx (k-in)	145.800
Stress Ratio:	0.61

<u>Use 1-1/2" thick x 12" x 24" Waler Bearing Plate</u>

Job # 17143 By JJB Date 8/2/17 Sheet# SC1

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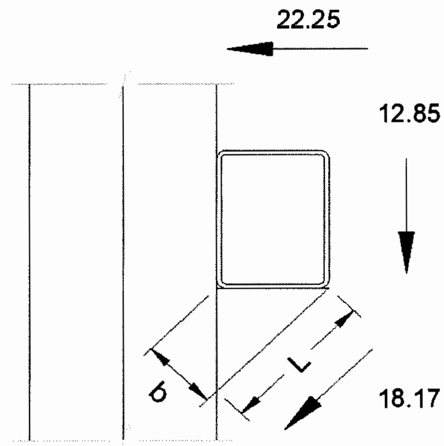
Client	PLI Systems
Project	Mole Linda Pile Wall
Location	2501 NE Rasmussen Road, Corbett, OR

Waler Connection to Pile

Horizontal Component **Th** (k)	22.25
Vertical Component **Tv** (k)	12.85
Tieback Spacing (ft)	8
Pile Spacing (ft)	8
Waler Reactions on Pile:	
Horizontal Reaction **Rh** (k)	22.25
Vertical Reaction **Rv** (k)	12.85



<u>Check Axial Load in Stiffener Plate:</u>	
Load on Stiffener (k)	18.17
Stiffener thickness **t** (in)	0.5
Stiffener width **b** (in)	5.657
Stiffener length **L** (in)	11.314
Steel plate yield strength **Fy** (ksi)	36
Element slenderness ratio **b/t**	11.31
Slenderness limit **λr** (AISC Table B4.1, #8)	21.29
Effective length factor **K**	1
Radius of gyration **r** (in) (t/12^1/2)	0.144
Column slenderness **KL/r**	78.4
Elastic buckling stress **Fe** (ksi)	46.585
Flexural buckling stress **Fcr** (ksi)	26.051
Safety Factor **Ω** (ASD)	1.67
Allowable compression (AISC Eq. E3-1) **Pa=1/Ω*Fcr*Ag** (k)	44.211
Stress Ratio:	0.41

<u>Use 8" x 8" x 1/2" thick triangular stiffener plate</u>

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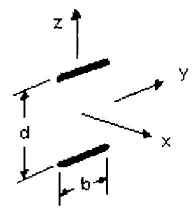
Client	PLI Systems
Project	Mole Linda Pile Wall
Location	2501 NE Rasmussen Road, Corbett, OR

Weld Group - 2 Lines

Check Stiffener Plate Fillet Line Weld:

Shear Load on Connection (k)	12.85
Bending on Connection (k-in)	51.4

Type of Weld	Fillet Weld
Nominal Tensile, **Fexx** (ksi)	70
Thinnest Welded Part (in)	0.375
Minimum Nominal Weld Size (in)	0.1875
Maximum Nominal Weld Size (in)	0.3125
Nominal Weld Leg Size (in)	0.25
Effective Throat t_E (in)	0.177
Weld Pattern Depth, **d** (in)	0.5
Weld Pattern Width, **b** (in)	8



Properties			
Length, **Lw** (in)	16.000	NA Distance, **cy** (in)	4
Moment of Inertia, **Iy** (in3)	1	NA Distance, **cz** (in)	0.25
Moment of Inertia, **Iz** (in3)	85.33333	Section Modulus, **Sy** (in2)	4.000
Torsional Moment **Jx** (in3)	86.33333	Section Modulus, **Sz** (in2)	21.333

Loads

Px (k)	Py (k)	Pz (k)	Tx (k-in)	My (k-in)	Mz (k-in)
0	12.85	0	0	0	51.4

Stresses

	Px	Py	Pz	Tx	My	Mz	Total
fx (pli)	0	-	-	-	0	2.409375	2.409
fy (pli)	-	0.803125	-	0	-	-	0.803
fz (pli)	-	-	0	0	-	-	0.000
					Total - SRSS (pli)		2.540

Effective Weld Area **Aw** (in2)	0.177	
Safety Factor **Ω** (ASD)	2	
Allowable Stress **Fw=1/Ω*0.6Fexx** (ksi)	21	*AISC Table J2.5*
Allowable Unit Load **Fw*Aw** (k/in)	3.712	*Eq. J2-3*
Maximum Applied Unit Load (k/in)	2.540	
Stress Ratio:	0.68	

Use 1/4" Fillet Both Sides

Earth Engineers, Inc.

2411 Southeast 8th Avenue • Camas • WA 98607
Phone: 360-567-1806 • Fax: 360-253-8624
www.earth-engineers.com

March 27, 2017

Linda K. Moles
2501 Northeast Rasmussen Road
Corbett, Oregon 97019

Phone: 503-380-4928
E-mail: lindakay8@gmail.com

Subject: **Geotechnical Investigation Report**
Retaining Wall Replacement
2501 Northeast Rasmussen Road
Corbett, Multnomah County, Oregon 97019
EEI Report No. 17-028-1

Dear Linda Moles:

Per your request, **Earth Engineers, Inc. (EEI)** is providing geotechnical engineering recommendations for the replacement of the existing, failing retaining wall on your property. Our services have been conducted in accordance with EEI Proposal No. 17-P049-R1 dated February 20, 2017 which you authorized on February 20, 2017.

PROJECT BACKGROUND

Our initial understanding of the project was based on a brief site meeting with you on February 10, 2017. Based on our site visit by EEI Principal Geotechnical Engineer Troy Hull, an existing timber retaining wall immediately north of the home is failing. There was a large crack in the ground behind the wall and the soil had dropped 1 to 2 inches between the ground crack and the wall.

A survey of the property titled "Moles Exhibit Map" by Griffin Land Surveying, Inc, dated March 15, 2017, was provided to us. The survey plan shows 2-foot topographic contours, the existing home, Rasmussen Road, as well as existing trees and culverts. It is noted that Hurlburt Road is also shown on the survey, however this roadway is not current constructed. In addition, the Rasmussen Road right-of-way is not where the actual road has been constructed. The topographic site plan is included in our Site Exploration Plan (Appendix B), attached to the end of this report.

SCOPE OF SERVICES

We have been requested to provide geotechnical engineering recommendations for the design of a retaining structure to replace the existing, retaining wall along the north property line. In order to evaluate the existing failing retaining wall and the property, we performed a subsurface investigation to determine the subsurface soil and/or rock conditions present. Two Standard Penetration Test (SPT) borings (B-1 and B-2) were performed using a subcontracted Beretta T-46 drill from PLI Systems (PLi) of Hillsboro, Oregon. Our borings extended to a depth of 20 and 28 feet below, respectively. Note that B-1 was rock cored from 25 to 30 feet, after the 25 to 26.5 foot SPT sample was obtained.

This report includes the following:

- A discussion of subsurface conditions encountered including pertinent soil and groundwater conditions.
- Recommendations for stabilizing the retained slope north of the house.
- Other discussion on geotechnical issues that may impact the project.

Note that our scope does not include any structural engineering for the waler that will transfer the loading from the tiebacks to the piles. A structural engineering consultant will need to be hired to provide that portion of the replacement wall design. This report will need to be provided to the structural engineer so that they can complete their design.

Our scope also does not include stabilizing any other areas of the property. It is limited to the area noted with dashed lines shown in Appendix B

SITE LOCATION AND DESCRIPTION

The property is located in the sparsely populated town of Corbett, Oregon. The irregularly shaped lot is approximately 0.55 acres in size (according to www.portlandmaps.com), and is bordered by Northeast Rasmussen Road at the north and east. Vacant lots, both privately and publically owned, border the property to the south and west. The lot sits on a hillside that is steeply sloping down to the north.

The subject lot includes an existing 2-story single family residence located in the central portion, nearly level portion of the property. According to www.portlandmaps.com, the house was built in 1928.

Various retaining structures are present at the site. Three timber walls, running east-west, are at the site. The two upper walls are within the confines of the subject property and lines the north side of the subject property. The east end of the lower wall appears to at the property border with Northeast Rasmussen Road. See Photo 1 below.

PHOTO 1: Viewing west at 2 existing wood retaining walls between NE Rasmussen Road and the private driveway, viewing from the northeast property corner.

A existing retaining wall along the north edge of the driveway tapers from about 1 foot in height at its east end (see Photo 1 above) up to about 11 feet at its west end. This retaining wall is covered with overhanging brush for the west approximate 40 feet (see Photo 2 below). The soils behind this retaining wall are generally level for about 50 feet (creating the house pad and driveway). The wall appears to be constructed using wood timbers as both the vertical posts and horizontal wall facing. It was noted that some of the wood posts and facing had deteriorated, and the tallest section of the wall appeared to be tilting outward. We also observed occasional round metal vertical posts in place in front of the wall facing. We presume these posts were added to provide additional support after the timber wall had been constructed. See Photo 3 below.

PHOTO 2: Viewing at the vegetation overhanging the west end of the upper timber wall lining the north side of the driveway.

PHOTO 3: Viewing at the east end of the timber wall lining the north side of the driveway. Note the intermediate metal round posts between the timber posts.

The lower timber retaining wall shown in Photo 1 above lines the south edge of Northeast Rasmussen Road. The wall height ranges from a couple feet up to about 7 feet, and includes sloping backfill that ranges from about 2H:1V to 1/2H:1V (Horizontal:Vertical). It is noted the alignment of this lower wall is broken into 2 sections with about 70 lateral feet in this break of the timber wall alignment. Some regions of this alignment break do not include any retaining structure, only sloping soils upward from the edge of the roadway shoulder. A cement masonry unit (CMU) structure is retaining about 3 to 5 feet of soil for about 25 lateral feet of this break in the wall alignment. This CMU structure is battered (i.e. leaning) into the slope at about 20 to 25 degrees. See Photo 4 below. During each of our site visits, there was a lot of water observed flowing on the slope.

PHOTO 4: Viewing west at a CMU retaining structure along the south side of Northeast Rasmussen Road.

The existing home sits about 25 feet behind the upper timber retaining wall. This region of the property is generally level and covered with either gravel or grasses. It is noted that the west end of the driveway appeared to have sunken relative to the surrounding grades. The tilting section of the upper timber wall is located near these possible sunken soils. Surface cracking was also observed in the soils behind the upper wall, in the section where the wall height is about 8 feet. Finally, existing stormwater collection systems in the driveway as well downspouts appear to be routed to flexible pipe which discharges downslope, below the lower wall into a publically maintained catch basin.

AREA GEOLOGY

The subject property is located on a terrace above the south side of the Columbia River, in the Western Cascade lowlands just east of where the Columbia River Gorge opens into the Portland Basin. The mapped lithology in the area of the subject property is Tgn_2 magnestratographic subunit of the Grand Ronde Basalt of the middle Miocene Columbia River Basalt Group (unit Tgn_2)[1]. This unit is made up of grey to black, fine to medium grained, basaltic-andesite flows.

The surface soils on the subject property are mapped by the US Soil Survey as Aschoff-Rock outcrop-Wahkeena association, very steep and Burlington fine sandy loam on 8 to 15 percent slopes[2]. Aschoff-Rock outcrop soils are well-drained and form on mountain slopes from colluvium derived from andesite and basalt mixed with volcanic ash. Burlington fine sandy loam is considered excessively drained and is derived by alluvial depositing.

A review of the Oregon Department of Geology and Mineral Industries' (DOGAMI's) Statewide Landslide Inventory Database (SLIDO), version 3.2, indicates that the property is centrally located within a mapped ancient (i.e. greater than 150 years ago) landslide area. SLIDO further defines the ancient slide to be deep seated and of a rock and earth flow movement. Figure 1 below shows the subject property relative to the mapped landslides. It is noted that all of the properties along Northeast Rasmussen Road are located inside the mapped landslide by SLIDO.

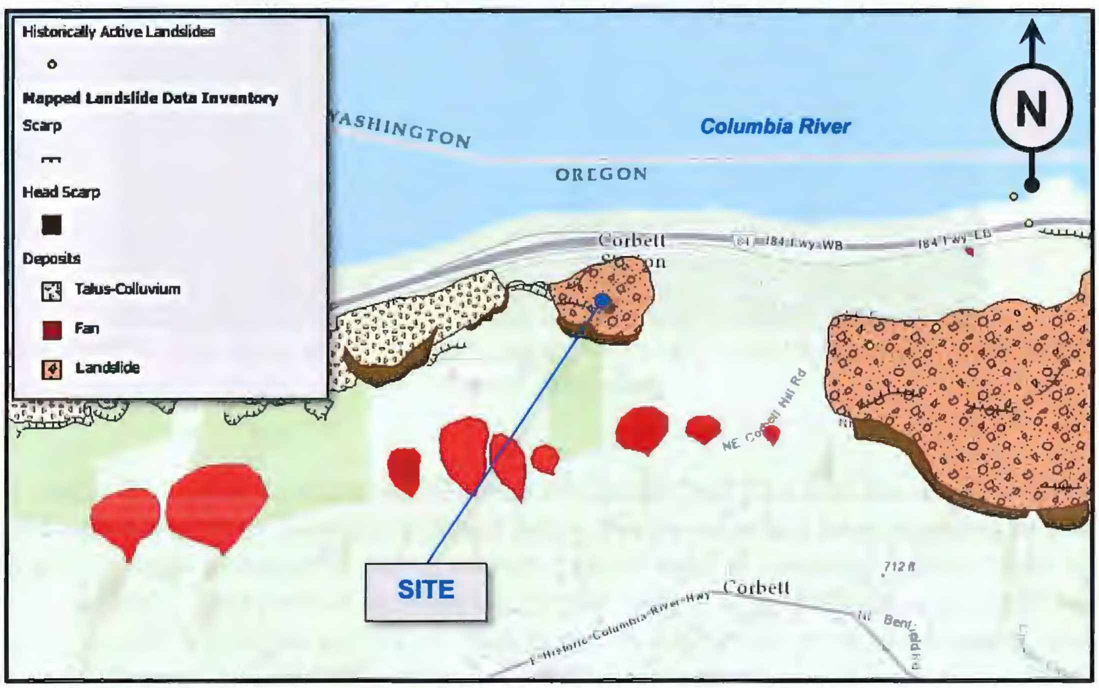


FIGURE 1: SLIDO version 3.2 image of the area around the subject property.

[1] Phillips, W.M., 1987, Geologic map of the Vancouver quadrangle, Washington: Washington Division of Geology and Earth Resources, Open File Report 87-10, scale 1:100,000

[2] Soil Survey Staff, Natural Resources Conservation Service, United States Department of Agriculture. Web Soil Survey. Available online at http://websoilsurvey.nrcs.usda.gov/ accessed 3/20/2017.

SEISMICITY

In accordance with ASCE 7-10, we recommend a Site Class D (stiff soil profile) for this site when considering the average of the upper 100 feet of bearing material beneath the foundations. Inputting our recommended site class as well as the site latitude and longitude into the United States Geological Survey (USGS) website[3], we obtained the seismic design parameters shown in Table 1 below. The return interval for these ground motions is 2 percent probability of exceedance in 50 years.

TABLE 1: Seismic Design Parameter Recommendations (ASCE 7-10)

PARAMETER	RECOMMENDATION
S_s	0.805g
S_1	0.347g
F_a	1.178
F_v	1.706
S_{MS} (=S_s x F_a)	0.948g
S_{M1} (=S_1 x F_v)	0.592g
S_{DS} (=2/3 x S_s x F_a)	0.632g
Design PGA (=S_{DS} / 2.5)	0.253g
MCE_G PGA	0.340g
F_{PGA}	1.160
PGA_M=(MCE_G PGA x F_{PGA})	0.394g

Note: Site latitude = 45.540352, longitude = -122.292092

In regard to earthquake faulting affecting the retaining wall, there are no faults mapped on the property (the closest mapped fault is located approximately 1 mile northeast of the south end of the Lacamas Lake Fault[4]).

SUBSURFACE EXPLORATION

In order to evaluate the subsurface conditions behind the failing retaining wall, we performed 2 SPT soil borings, B-1 and B-2. B-1 was located just behind the failing retaining wall, while B-2 was located next to the home. B-1 and B-2 were advanced to about 28 and 21 feet below the existing ground surface, respectively.

In general, both soil borings encountered what we interpreted to be a native sandy silt soil stratum overlying decomposed bedrock overlying highly fractured and moderately weathered basalt bedrock. The exception to this was that in B-1 (located closest to the retaining wall), there was a 10-foot thick fill stratum first encountered at the ground surface that consisted of silty sand with trace gravel. Each stratum is described in more detail below.

[3] USGS, Earthquakes Hazard Program, http://earthquake.usgs.gov/designmaps/us/application.php
[4] Personius, S.F., compiler, 2002, Fault number 880, Lacamas Lake fault, in Quaternary fault and fold database of the United States: U.S. Geological Survey website, http://earthquakes.usgs.gov/hazards/qfaults, accessed 09/9/2016.

The apparent fill consisted of silty sand with trace gravel. SPT N-values ranged from 5 to 6, indicating it was loose. Moisture contents ranged from 24 to 29 percent. The one sample tested for fines content (i.e. material passing the #200 sieve resulted in 49 percent fines. Based on our observations of the samples and the drilling resistance, it does not appear that this fill was properly compacted when it was placed.

Underlying the fill in B-1 and encountered at the ground surface of B-2, was sandy silt what we interpret to be native soil. SPT N-values ranged from 5 to 27, indicating it was loose to medium dense. Moisture contents of the samples tested ranged from 22 to 35 percent. Fines content of the samples tested ranged from 63 to 68 percent fines.

Underlying the native sandy silt stratum in both borings was a layer of silty sand with gravel. SPT N-values ranged from 25 to greater than 50, indicating it was medium to very dense. Moisture contents of the samples tested ranged from 22 to 35 percent. Fines content of the samples tested ranged from 36 to 37 percent.

The bedrock stratum was encountered beneath the silty sand with gravel. The top portion of the bedrock was decomposed to a soil with some rock fragments. It was encountered at a depth of about 20.5 feet in B-1 and 20 feet in B-2. The thickness of the decomposed bedrock was about 5 feet. Harder, fractured bedrock was encountered beneath the decomposed layer—at a depth of about 25 feet in B-1. The quality of the bedrock was very low, with core recover of about 50 percent and an RQD value of 0 percent. We did not drill deep enough in B-2 to encounter the harder, fractured bedrock layer.

The above subsurface description is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The boring logs included in the Appendix should be reviewed for specific information at specific locations. These records include soil and rock descriptions, stratifications, and locations of the samples. The stratifications shown on the logs represent the conditions only at the actual exploration locations. The stratifications represent the approximate boundary between subsurface materials and the actual transition may be gradual. Water level information obtained during field operations is also shown on these logs. The samples that were not altered by laboratory testing will be retained for 60 days from the date of this report and then will be discarded.

GEOTECHNICAL DISCUSSION

Based on our site observations the structural integrity of the existing timber retaining wall supporting the driveway and home is comprised. It is rotting and tilting, and its retained ground is shifting. We recommend that the existing retaining wall be replaced. In order to reduce construction costs, it would be acceptable to leave the existing wall in place and construct the new wall in front of the old wall.

We did preliminarily evaluate whether a global, deep-seated slope stability issue could possibly be present, and causing the retaining wall to fail. We ruled this out as we understand the house is not exhibiting any distress (i.e. cracks in the drywall, doors and windows that aren't closing properly, etc.). If there was a larger, global slope stability problem, the house would be expected to show some distress. In addition, bedrock was encountered relatively shallow. In B-1, we encountered it at about 25 to 26 feet below grade. In B-2, it appears that bedrock is about 21 feet below grade. We did not observe any signs of a slip plane in the SPT samples obtained at the transition between soil and bedrock. Finally, it appears that the fill immediately behind the wall is uncompacted and the loose fill soil fill allows rain water to saturate the soils and exert greater than normal lateral earth pressure on the wall. It is our professional opinion that this deep, loose fill soil is the primary cause of the failing retaining wall.

We considered mitigating the failing wall by replacing the uncompacted fill with properly compacted structural fill. However, given the significant depth of the fill, the existing deteriorating timber retaining wall elements, and the close proximity to the house and underground utilities, we decided it would not be prudent. In addition, the excavation would require its own shoring wall to protect the home, which would defeat the purpose of doing the overexcavation in the first place.

Therefore, we are recommending the existing wall be replaced with a new wall. Based on the subsurface conditions encountered and the design height of the new wall, we are providing design recommendations for a solder pile and tieback retaining wall.

We considered replacing the entire existing wall with a new wall. Given budget limitations, we are only providing recommendations for replacing a 64-foot long section of the wall located immediately in front of the house (i.e. to protect the home).

SOLDIER PILE RETAINING WALL DESIGN RECOMMENDATIONS

We recommend the existing timber retaining wall be replaced by constructing a soldier pile and tieback retaining wall. We performed our retaining wall engineering analysis using Shoring Suite, Version 8.17a computer software by CivilTech Software. The following design parameters were used in our analysis:

- A maximum retaining wall height of 16 feet was assumed. As previously mentioned, the exposed height of the current wall is only about 11 feet. We have assumed a design height of 16 feet for the new wall to account for soft soils that extend down to that depth. As such, some soil will need to be removed from the front of the wall in order to place the wall facing (i.e. wood lagging).
- The retaining wall design is based on a minimum Factor of Safety (FOS) against overturning of 1.5 for static conditions and 1.1 for seismic conditions. The FOS was applied to the passive earth pressure, or the resisting force. Note after analyzing both the static and seismic conditions, we determined that the seismic design controls.
- The maximum deflection at any point along the wall height was limited to less than 1 inch.
- In accordance with NAVFAC's earth pressure diagram for tied-back walls and dense granular soils, a rectangular shaped at-rest earth pressure of 480 pcf was applied. This value was based on a constant of 0.5, a K_o of 0.5, a total soil unit weight (γ) of 120 pcf, and a wall height (H) of 16 feet (see Figure 2 below).

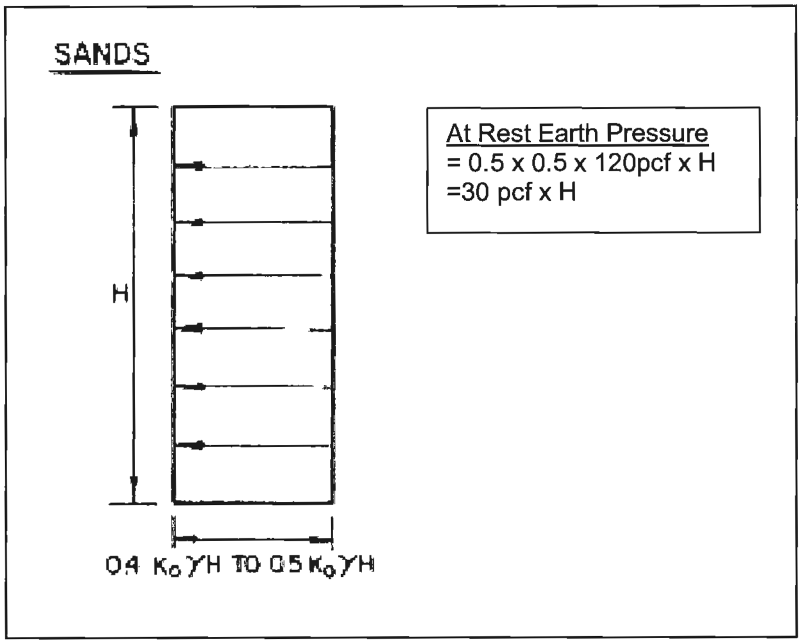


FIGURE 2: Tied-back at-rest earth pressure diagram (source: NAVFAC DM-7.2, Page 106).

- An earthquake load was applied. We used ½ of the Design Peak Horizontal Ground Acceleration (Design PGA) of 0.253g in accordance with Section 1803.5.12 of the 2014 OSSC.
- The vertical piles will be set into 16-inch minimum diameter predrilled boreholes and then backfilled with concrete.
- The vertical pile elements will develop passive arching resistance equivalent to 3 times the pile diameter, with an ultimate passive earth pressure value of 350 pcf based on most of the passive resistance being derived from the subsurface sand and silt mixtures.
- The vertical pile steel was assumed to have a yield strength (Fy) of at least 50 ksi.

Based on the above input parameters, we developed the following retaining wall design.

- The retaining wall should be supported vertically by W10x26 (or equivalent) steel piles with a length of 30 feet each. Note that if difficult drilling is encountered on basalt bedrock, it would be acceptable to shorten the pile length such that the piles are embedded at least 3 feet into the fractured bedrock stratum.
- The piles should be laterally spaced no further than 8 feet measured center to center.
- The pile locations should be predrilled followed by pile placement into the drilled shaft. The voids surrounding the pile should then be filled with structural concrete that has a minimum compressive strength of at least 2,500 psi for the vertical distance below the planned bottom of wall elevation. The concrete should be briefly consolidated with a concrete vibrator to ensure complete concrete coverage around each pile. The concrete does not need to be consolidated with a vibrator if it has a design slump of at least 6 inches.

- The portion of the piles that will have a wood lagging facing should be backfilled with lean concrete (i.e. CDF) having a minimum compressive strength of at least 150 psi. The lean concrete will then have to be chipped away in order to place the wood lagging between the pile flanges.
- The retaining wall should be supported laterally by a single row of permanent, drilled and grouted **solid bar** tieback anchors as outlined below. We considered hollow bar tiebacks however, we have concern that they would not be able to drill deep enough into the hard rock to achieve the desired tension load. The tiebacks should be metalized for corrosion protection because they are permanent. The diameter of the grouted columns was assumed to be at least 6 inches. The tieback bond length should be completely in bedrock and the unbonded length should be completely in soil. In no case should the bonded and unbonded lengths be less than shown in Table 2 below.

TABLE 2: Tieback Anchor Design Recommendations

Tieback Anchor Row #	Depth Below Top of Wall (feet)	Angle Below Horizontal (degrees)	Anchor Lateral Spacing (feet)	Design Maximum Anchor Tension Force (kips)	Estimated Anchor Unbonded Length (feet)	Estimated Anchor Bonded Length (feet)[1]	Total Estimated Anchor Length (feet)
1	5½	30	8	69	26	10	36

Notes:
1. Grouted length based on an assumed design rock-grout bond strength of 5 kips per square foot in bedrock. Actual required grouted tieback length will be based on tieback pull testing. The tiebacks may actually be shorter or longer than reported in Table 2, depending upon tieback pull test results.

- The tieback grout compressive strength (f'g) should be no less than 5,000 psi at 28 days.
- Centralizers should be used with the tieback bars at a spacing not to exceed 7 feet. The first centralizer should be installed within 18 inches of the end of the bar.
- All anchors will need to be proof tested to 150 percent of the design load (AL= alignment load, DL=design load):

  AL, 0.25 DL, 0.50 DL, 0.75 DL, 1.00 DL, 1.25 DL, 1.33/2.0 DL, Lockoff Load (0.8 DL)

  Proof test readings shall be taken immediately after reading each load increment, except at 1.25 DL and 1.5 DL. At 1.25 and 1.5 DL, readings shall be taken at 1, 2, 3, 4, 5, 6 and 10 minutes. If the total creep movement exceeds 0.040 inches between 1 and 10 minutes (i.e. 1 log cycle), then the test load shall be maintained for an additional 50 minutes, with recordings at 20, 30, 40 50 and 60 minutes. The total creep movement should not exceed 0.080 inches between 6 and 60 minutes (1 log cycle).

  The Geotechnical Engineer should evaluate the proof test results to verify the anchors will achieve their designed capacity without excessive movement.

- Tieback pull testing shall not occur until the grout has reached a minimum compressive strength of 3,500 psi.
- Because some of the tiebacks will cross each other (see Appendix F), it is acceptable to modify the tieback installation by up to 5 degrees. In other words, it will be acceptable to install tiebacks anywhere from 25 to 35 degrees below horizontal, for the purposes of avoiding hitting other tiebacks.
- We recommend the wall facing consist of 4-inch thick pressure treated wood lagging.
- A 4-inch diameter, perforated plastic drain pipe should be placed behind the entire length of the base of the retaining wall for drainage. The drain pipe should outlet into the drainage ditch next to the road.
- The gap between the new wall and the old wall shall be backfilled with drain rock.
- The wall should be vertical or battered slightly into the slope. If, during pile installation, any of the piles start to develop a negative batter so that the top of the pile is leaning out away from the slope, they should be immediately corrected.
- The tiebacks will need to be structurally connected to the piles using a waler (i.e. steel beam) designed by a Structural Engineer. To be clear, the waler has not been designed yet as structural engineering is not included in our scope of services.

CONSTRUCTION INSPECTION

Due to the complex nature of this project, it is important that EEI be retained to provide geotechnical observation services during the construction phase to mitigate the items discussed above. EEI cannot accept responsibility for any conditions that deviate from those described in this report, if not engaged to also provide construction observation for this project.

A representative of the Geotechnical Engineer should perform the following construction inspections.

- Drilling of the boreholes for the retaining wall piles (continuous).
- Placement of the retaining wall piles (periodic).
- Placement of the concrete to backfill around the piles (continuous).
- Installation, pull testing, and lockoff of retaining wall tiebacks (continuous).
- Installation of the wood lagging (periodic).
- Wall drain pipe and drain rock backfill installation (periodic).

LIMITATIONS

As is standard practice in the geotechnical industry, the conclusions contained in our report are considered preliminary because they are based on assumptions made about the soil, rock, and groundwater conditions exposed/interpreted at the site during our subsurface investigation. A more complete extent of the actual subsurface conditions can only be identified when they are exposed/interpreted during construction. Therefore, EEI should be retained as your consultant during construction to observe the actual conditions and to provide our final conclusions. If a different geotechnical consultant is retained to perform geotechnical inspection during construction then they should be relied upon to provide final design conclusions and recommendations, and should assume the role of geotechnical engineer of record.

Our scope of services is limited to providing geotechnical engineering recommendations for replacing the existing the existing retaining wall in order to stabilize the ground immediately behind it. We have not evaluated any other portions of your property for stability.

The geotechnical conclusions and recommendations presented in this report are based on the available project information described in this report. If any of the noted information is incorrect, please inform EEI in writing so that we may amend the recommendations presented in this report if appropriate. EEI will not be responsible for the implementation of its recommendations when it is not notified of changes in the project.

This report has been prepared for the exclusive use of Linda K. Moles, owner of the property located at 2501 Northeast Corbett Hill Road in Corbett, Oregon. EEI does not authorize the use of the advice herein nor the reliance upon the report by third parties without prior written authorization by EEI.

If you have any questions pertaining to this report, please contact our office at 360-567-1806.

Sincerely,
Earth Engineers, Inc.

REGISTERED PROFESSIONAL ENGINEER
19206PE
OREGON
JULY 15, 1997
TROY M. HULL
EXPIRES: 6/30 17

Troy Hull, P.E., G.E.
Principal Geotechnical Engineer

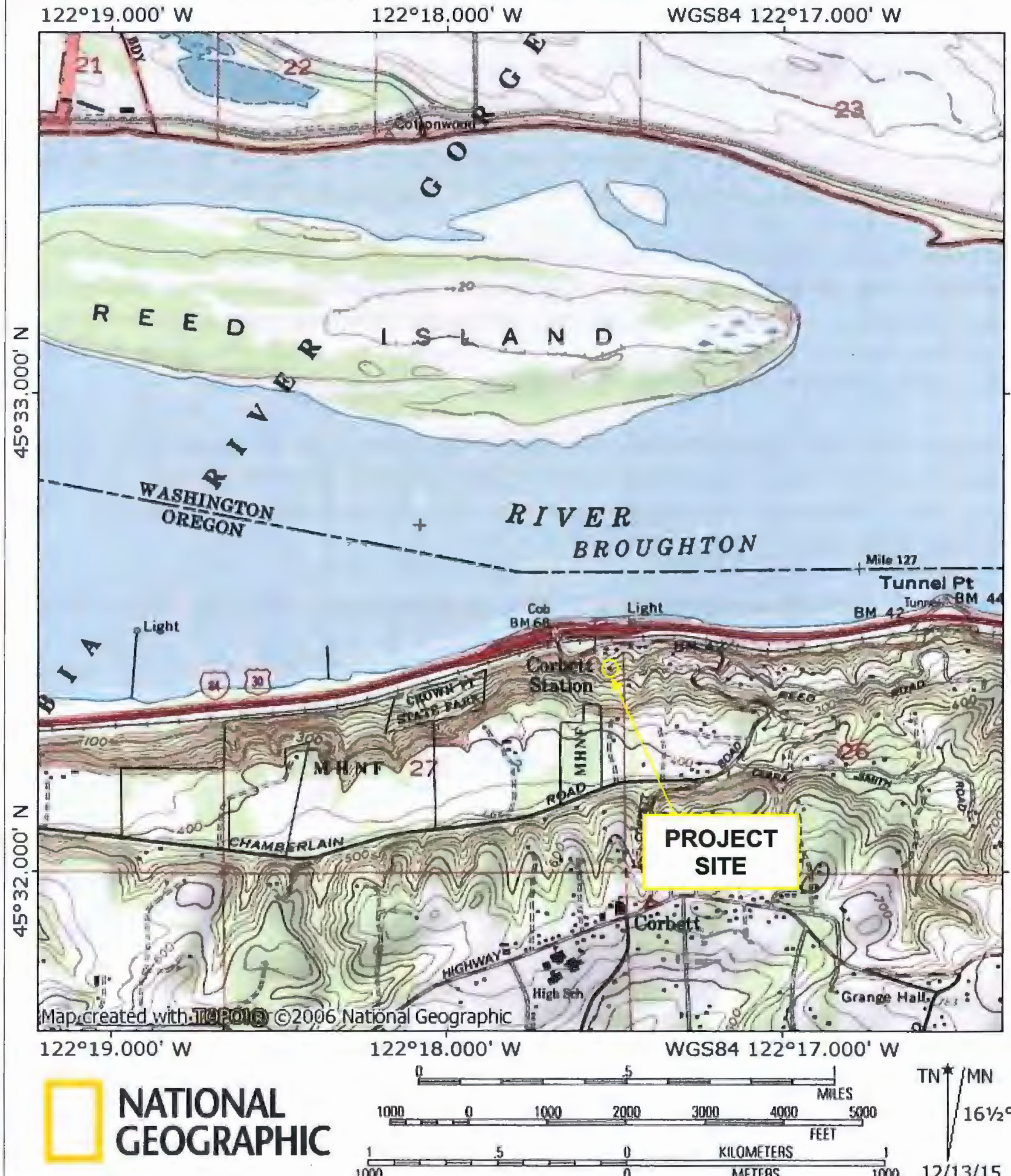
Reviewed by:

Jeremy Fissel, P.E.
Geotechnical Engineer

Attachments: Appendix A, Site Location Plan
Appendix B, Boring Location Plan
Appendix C, Boring Logs
Appendix D, Soil and Rock Classification Legends
Appendix E, Shoring Suite Calculations
Appendix F, New Retaining Wall Site Plan
Appendix G, New Retaining Wall Cross-Section

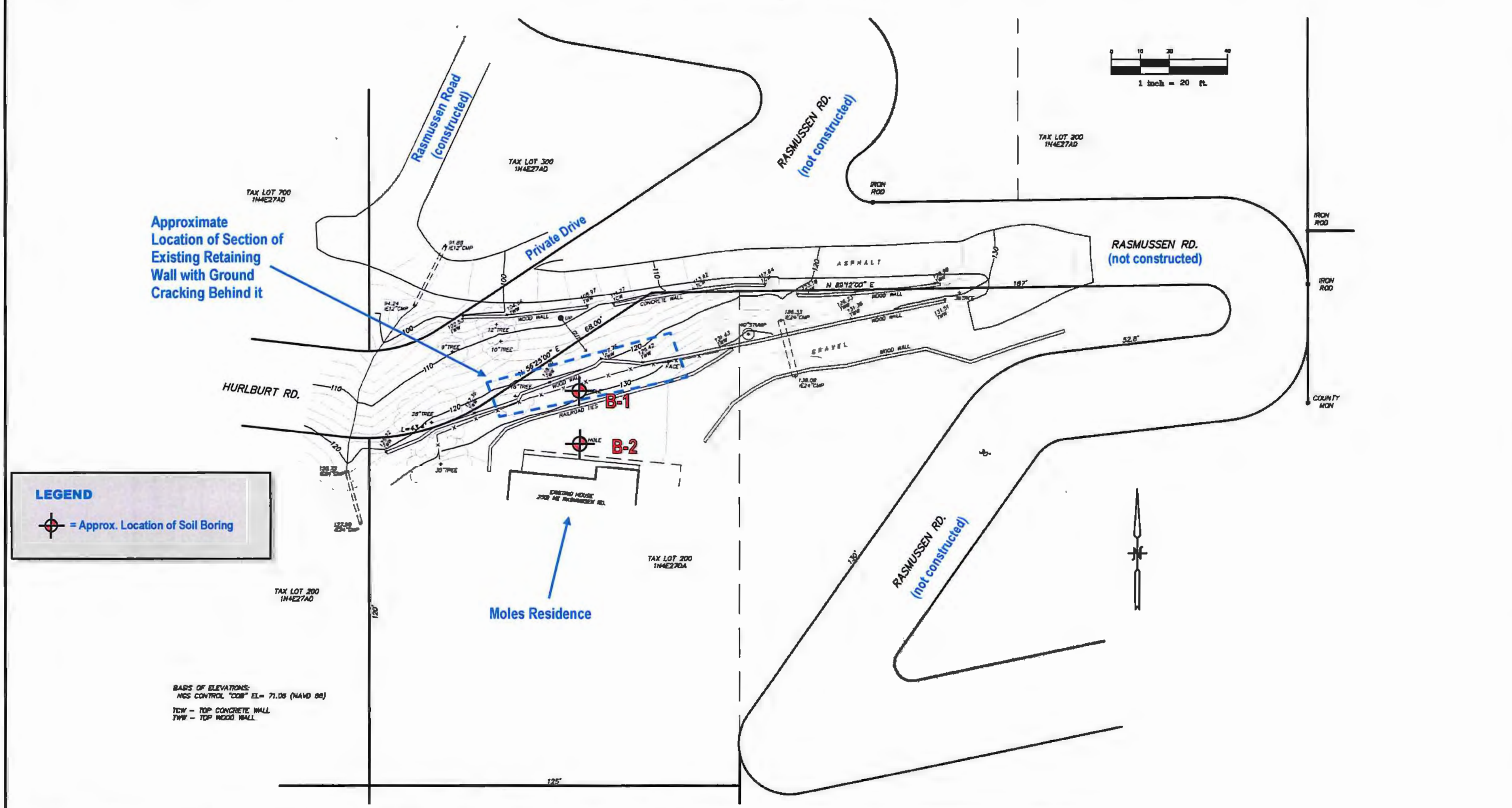
APPENDIX A – SITE LOCATION PLAN

TOPO! map printed on 12/13/15 from "Untitled.tpo"



Earth Engineers, Inc.	**Failing Retaining Wall Evaluation** **Moles Single Family Residence** **2501 NE Rasmussen Road** **Corbett, Multnomah County, Oregon**	**Report No. 17-028-1** **March 27, 2017**

APPENDIX B – BORING LOCATION PLAN



Base Drawing Source: March 15, 2017 drawing titled "Moles Exhibit Map, Tax Lot 200 Map 1N4E27DA, in the SE 1/4 of Section 27, T.1N., R.4E., W.M., Multnomah County, Oregon," by Griffin Land Surveying Inc. of Beaverton, Oregon.

Earth Engineers, Inc.	Failing Retaining Wall Evaluation Moles Single Family Residence 2501 NE Rasmussen Road Corbett, Multnomah County, Oregon	EEI Report No. 17-028-1	March 27, 2017

APPENDIX C: BORING B-1

CLIENT: Linda K. Moles	EARTH ENGINEERS, INC. REPORT NO.: 17-028-1
PROJECT: Failing Retaining Wall Evaluation	EQUIPMENT: PLi's Beretta T-46 Drill, HSA, Auto SPT Hammer
LOCATION: 2501 NE Rasmussen Road, Corbett, Oregon	APPROXIMATE ELEVATION: 130 feet
DATE DRILLED: February 20, 2017	LOGGED BY: Troy Hull, PE, GE

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS / 6 IN.	N_{60} VALUE	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT, %	REMARKS
0			**LANDSCAPING BARK:** wet, some silt and sand, approximately 2 inches thick							
3–4.5	SPT-1		Silty Sand, trace round gravel, wet, brown, loose (PROBABLE FILL)	3 2 2	5				28	
5.5–7	SPT-2			2 2 2	5				29	
8–9.5	SPT-3		*no trace gravel*	2 2 3	6	49			24	
10.5–12	SPT-4		Sandy Silt, trace round gravel, moist, brown, loose	2 2 3	6	63			25	
13–14.5	SPT-5		*apparent density becomes medium dense, no trace gravel*	2 2 9	14				27	
15			*(continued on next page)*							

EARTH ENGINEERS, Inc.

APPENDIX C: SOIL BORING B-1

CLIENT: Linda K. Moles	EARTH ENGINEERS, INC. REPORT NO.: 17-028-1
PROJECT: Failing Retaining Wall Evaluation	EQUIPMENT: PLi's Beretta T-46 Drill, HSA, Auto SPT Hammer
LOCATION: 2501 NE Rasmussen Road, Corbett, Oregon	APPROXIMATE ELEVATION: 130 feet
DATE DRILLED: February 20, 2017	LOGGED BY: Troy Hull, PE, GE

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS/6 IN.	N-VALUE	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT, %	REMARKS
			(continued from previous page)							SPT sampler fell 1 foot under weight of hammer
	SPT-6		Sandy Silt, wet, brown, very loose	0						
16				0		66			30	
				3						
17	SPT-7		*trace semi-rounded gravel*	14	25				30	
				6						
18										drill grinding from 18 to 19 feet
19										easy drilling from 19 to 20 feet
20				8						
21	SPT-8		Silty Sand with Gravel, brown, wet, dense to very dense *(inferred as intensely weathered rock)*	16	46	36			30	hard drilling from 20 to 22½ feet
				20						
22										
23										very hard drilling from 22½ to 25 feet
24										
25				15						switched from Hollow Stem Auger to Rock Coring at 25 feet
26	SPT-9			35	>50				35	
			highly fractured basalt bedrock with silt soil infill	50/4"						
27										very slow rock coring recover=50% RQD=0
28										
29			Soil boring/rock coring terminated at 28 feet below the surface. Borehole was backfilled with bentonite chips after completion. Groundwater was not encountered at the time of the exploration. Boring located 20 feet from NE house corner & 24 feet from front porch slab & 9' from existing retaining wall. Auto-hammer had an efficiency of 76.9% by GeoDesign Report of SPT Hammer Energies dated 5/27/2015. Therefore the blow counts have been multiplied by 1.28 (i.e. 76.9/60 = 1.28). Reported elevation approximated using provided survey by Griffen Land Surveying dated 3/15/2017.							
30										

EARTH ENGINEERS, Inc.

APPENDIX C: SOIL BORING B-2

CLIENT: Linda K. Moles	EARTH ENGINEERS, INC. REPORT NO.: 17-028-1
PROJECT: Failing Retaining Wall Evaluation	EQUIPMENT: PLi's Beretta T-46 Drill, HSA, Auto SPT Hammer
LOCATION: 2501 NE Rasmussen Road, Corbett, Oregon	APPROXIMATE ELEVATION: 130 feet
DATE DRILLED: February 20, 2017	LOGGED BY: Troy Hull, PE, GE

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS/6 IN.	N-VALUE	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT (%)	REMARKS
			CRUSHED GRAVEL DRIVEWAY SURFACING: About 4 inches thick, moist,							
1										
2										
3	SPT-1		Sandy Silt, trace round gravel, moist, very loose to medium dense transitioning to loose	4 4 4	10				24	
4										
5										
6	SPT-2			3 11 10	27	68			35	
7										
8										
9										
10										
11	SPT-3		*apparent density becomes loose, no trace gravel*	2 2 2	5	66			35	
12										hard drilling at 12 feet, possible gravels
13										
14										
15			*(continued on next page)*							

EARTH ENGINEERS, Inc.

APPENDIX C: SOIL BORING B-2

CLIENT: Linda K. Moles	EARTH ENGINEERS, INC. REPORT NO.: 17-028-1
PROJECT: Failing Retaining Wall Evaluation	EQUIPMENT: PLi's Beretta T-46 Drill, HSA, Auto SPT Hammer
LOCATION: 2501 NE Rasmussen Road, Corbett, Oregon	APPROXIMATE ELEVATION: 130 feet
DATE DRILLED: February 20, 2017	LOGGED BY: Troy Hull, PE, GE

DEPTH (ft)	SAMPLE NO.	SAMPLE	SOIL DESCRIPTION	BLOWS/6 IN.	N-VALUE	% PASSING #200 SIEVE	LIQUID LIMIT	PLASTIC LIMIT	MOISTURE CONTENT, %	REMARKS
			(continued from previous page)							
16	SPT-4		Silty Sand with Gravel, moist to wet, medium to very dense	9 16 4	25	37			22	harder drilling at 16 feet
17										
18										
19										
20										
21	SPT-5		Silty Sand with Gravel, brown, wet, very dense *(inferred as intensely weathered rock)*	31 50/5"	>50	63			41	
22										
23										
24										
25										
26										
27										
28										
29			Soil boring terminated at 21 feet below the surface. Borehole was backfilled with bentonite chips after completion. Groundwater was not encountered at the time of the exploration. Boring located approximately 4.5 feet from front porch concrete slab & 20' from NE house corner. Auto-hammer had an efficiency of 76.9% by GeoDesign Report of SPT Hammer Energies dated 5/27/2015. Therefore the blow counts have been multiplied by 1.28 (i.e. 76.9/60 = 1.28). Reported elevation approximated using provided survey by Griffen Land Surveying dated 3/15/2017.							
30										

EARTH ENGINEERS, Inc.

APPENDIX D: SOIL CLASSIFICATION LEGEND

APPARENT CONSISTENCY OF COHESIVE SOILS (PECK, HANSON & THORNBURN 1974, AASHTO 1988)				
Descriptor	**SPT N_{60} (blows/foot)***	**Pocket Penetrometer, Qp (tsf)**	**Torvane (tsf)**	**Field Approximation**
Very Soft	< 2	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	2 – 4	0.25 – 0.50	0.12 – 0.25	Easily penetrated several inches by thumb
Medium Stiff	5 – 8	0.50 – 1.0	0.25 – 0.50	Penetrated several inches by thumb w/moderate effort
Stiff	9 – 15	1.0 – 2.0	0.50 – 1.0	Readily indented by thumbnail
Very Stiff	16 – 30	2.0 – 4.0	1.0 – 2.0	Indented by thumb but penetrated only with great effort
Hard	> 30	> 4.0	> 2.0	Indented by thumbnail with difficulty

* Using SPT N_{60} is considered a crude approximation for cohesive soils.

APPARENT DENSITY OF COHESIONLESS SOILS (AASHTO 1988)	
Descriptor	**SPT N_{60} Value (blows/foot)**
Very Loose	0 – 4
Loose	5 – 10
Medium Dense	11 – 30
Dense	31 – 50
Very Dense	> 50

MOISTURE (ASTM D2488-06)	
Descriptor	**Criteria**
Dry	Absence of moisture, dusty, dry to the touch, well below optimum moisture content (per ASTM D698 or D1557)
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table, well above optimum moisture content (per ASTM D698 or D1557)

PERCENT OR PROPORTION OF SOILS (ASTM D2488-06)	
Descriptor	**Criteria**
Trace	Particles are present but estimated < 5%
Few	5 – 10%
Little	15 – 25%
Some	30 – 45%
Mostly	50 – 100%
Percentages are estimated to nearest 5% in the field. Use "about" unless percentages are based on laboratory testing.	

SOIL PARTICLE SIZE (ASTM D2488-06)	
Descriptor	**Size**
Boulder	> 12 inches
Cobble	3 to 12 inches
Gravel - Coarse Fine	¾ inch to 3 inches No. 4 sieve to ¾ inch
Sand - Coarse Medium Fine	No. 10 to No. 4 sieve (4.75mm) No. 40 to No. 10 sieve (2mm) No. 200 to No. 40 sieve (.425mm)
Silt and Clay ("fines")	Passing No. 200 sieve (0.075mm)

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2488)				
Major Division			**Group Symbol**	**Description**
Coarse Grained Soils (more than 50% retained on #200 sieve)	**Gravel** (50% or more retained on No. 4 sieve)	Clean Gravel	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravel with fines	GM	Silty gravels and gravel-sand-silt mixtures
			GC	Clayey gravels and gravel-sand-clay mixtures
	Sand (> 50% passing No. 4 sieve)	Clean sand	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly-graded sands and gravelly sands, little or no fines
		Sand with fines	SM	Silty sands and sand-silt mixtures
			SC	Clayey sands and sand-clay mixtures
Fine Grained Soils (50% or more passing #200 sieve)	**Silt and Clay** (liquid limit < 50)		ML	Inorganic silts, rock flour and clayey silts
			CL	Inorganic clays of low-medium plasticity, gravelly, sandy & lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silt and Clay (liquid limit > 50)		MH	Inorganic silts and clayey silts
			CH	Inorganic clays or high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck and other highly organic soils

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GRAPHIC SYMBOL LEGEND		
GRAB		Grab sample
SPT		Standard Penetration Test (2" OD), ASTM D1586
ST		Shelby Tube, ASTM D1587 (pushed)
DM		Dames and Moore ring sampler (3.25" OD and 140-pound hammer)
CORE		Rock coring

APPENDIX D: ROCK CLASSIFICATION LEGEND

WEATHERING DESCRIPTORS FOR INTACT ROCK (USBR, 2001)						
Descriptor	**Chemical Weathering-Discoloration-Oxidation**		**Mechanical Weathering and Grain Boundary Conditions**	**Texture and Solutioning**		**General Characteristics**
	Body of Rock	**Fracture Surfaces**		**Texture**	**Solutioning**	
Fresh	No discoloration, not oxidized	No discoloration or oxidation	No separation, intact (tight)	No change	No solutioning	Hammer rings when crystalline rocks are struck
Slightly Weathered	Discoloration or oxidation limited to surface or short distance from fractures; some feldspar crystals are dull	Minor or complete discoloration or oxidation of most surfaces	No visible separation, intact (tight)	Preserved	Minor leaching of some soluble minerals may be noted	Hammer rings when crystalline rocks are struck; body of rock not weakened
Moderately Weathered	Discoloration or oxidation extends from fractures usually throughout; Fe-Mg minerals are "rusty," feldspar crystals are "cloudy"	All fracture surfaces are discolored or oxidized	Partial separation of boundaries visible	Generally preserved	Soluble minerals may be mostly leached	Hammer does not ring when rock is struck; body of rock is slightly weakened
Intensely Weathered	Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent or chemical alteration produces in-situ disaggregation	All fracture surfaces are discolored or oxidized; surfaces are friable	Partial separation; rock is friable; granitics are disaggregated in semi-arid conditions	Altered by chemical disaggregation such as via hydration or argillation	Leaching of soluble minerals may be complete	Dull sound when struck with hammer; usually can be broken with moderate to heavy manual pressure or by light hammer blow; rock is significantly weakened
Decomposed	Discolored or oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay		Complete separation of grain boundaries (disaggregation)	Resembles a soil; partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete		Can be granulated by hand; resistant minerals such as quartz may be present as "stringers" or "dikes"

RELATIVE STRENGTH OF INTACT ROCK	
Descriptor	**Uniaxial Compressive Strength (psi)**
Extremely Strong	> 30,000
Very Strong	14,500 – 30,000
Strong	7,000 – 14,500
Medium Strong	3,500 – 7,000
Weak	700 – 3,500
Very Weak	150 – 700
Extremely Weak	< 150

BEDDING SPACING (modified USBR, 2001)	
Descriptor	**Thickness or Spacing**
Massive.	> 10 feet
Very thickly bedded	3 to 10 feet
Thickly bedded	1 to 3 feet
Moderately bedded	3-5/8 inches to 1 foot
Thinly Bedded	1-1/4 inches to 3-5/8 inches
Very thinly bedded	3/8 inch to 1-1/4 inches
Laminated	< 3/8 inch

CORE RECOVERY CALCULATION (%)

$$= \frac{\text{length of recovered core pieces}}{\text{total length of core run}} \times 100\%$$

RQD CALCULATION (%)

$$= \frac{\text{length of intact core pieces} > 4\text{ in}}{\text{total length of core run (inches)}} \times 100\%$$

ROCK HARDNESS (modified USBR, 2001)	
Descriptor	**Criteria**
Extremely hard	Cannot be scratched with pocket knife or sharp pick; can only be chipped with repeated heavy hammer blows
Very hard	Cannot be scratched with pocket knife or sharp pick; breaks with repeated heavy hammer blows
Hard	Can be scratched with pocket knife or sharp pick with heavy pressure, heavy hammer blows required to break specimen
Moderately hard	Can be scratched with pocket knife or sharp pick with light or moderate pressure; breaks with moderate hammer blows
Moderately soft	Can be grooved 1/16 inch with pocket knife or sharp pick with moderate or heavy pressure; breaks with light hammer blow or heavy hand pressure
Soft	Can be grooved or gouged with pocket knife or sharp pick with light pressure; breaks with light to moderate hand pressure
Very soft	Can be readily indented, grooved, or gouged with fingernail, or carved with pocket knife; breaks with light hand pressure

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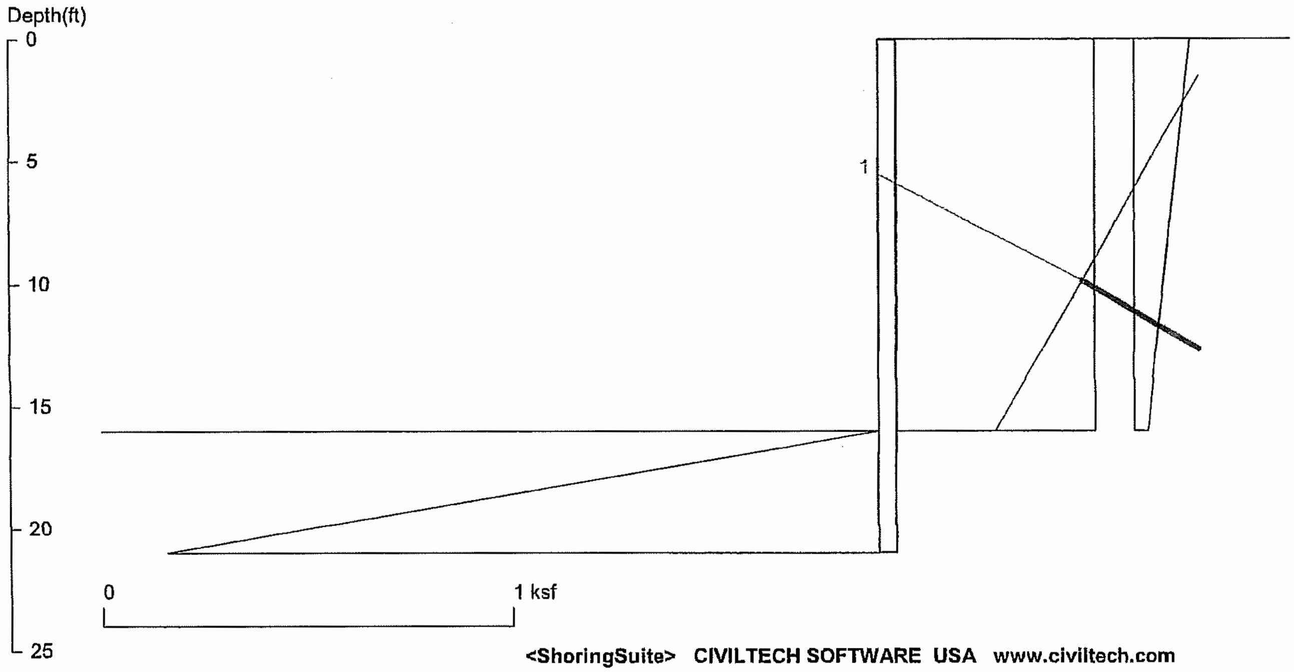
APPENDIX E

SHORING SUITE CALCULATIONS

Moles SFR Pile & Tieback Wall

16' tall, Seismic Analysis



<ShoringSuite> CIVILTECH SOFTWARE USA www.civiltech.com

Licensed to 4324324234 3424343 Date: 3/26/2017

File: P:\Projects\2017 Projects\17-028 (Moles SFR Failing Wall, 2501 NE Rasmussen Road, Corbett, OR) (TH)\Shoring St

Wall Height=16.0 Pile Diameter=1.0 Pile Spacing=8.0 Wall Type: 3. Soldier Pile, Driving

PILE LENGTH: Min. Embedment=4.98 (5~10ft is recommended!!!) Min. Pile Length=20.98
MOMENT IN PILE: Max. Moment=72.95 per Pile Spacing=8.0 at Depth=5.51

PILE SELECTION:
Request Min. Section Modulus = 26.5 in3/pile=434.71 cm3/pile, Fy= 50 ksi = 345 MPa, Fb/Fy=0.66
-> Piles meet Min. Section Requirements: Top Deflection is shown in (in)
HP8X36 (0.00) W8X31 (0.00) HP10X42 (0.00) W10X26 (0.00) HP12X53 (0.00)
W12X26 (0.00) HP13X60 (0.00) HP14X73 (0.00) W14X22 (0.00) W16X26 (0.00)
HP16X88 (0.00) W16X89 (0.00) HP16X101 (0.00) W16X100 (0.00)

BRACE FORCE: Strut, Tieback, Plate Anchor, Deadman, Sheet Pile as Anchor

Bedrock

No. & Type	Depth	Angle	Space	Total F.	Horiz. F.	Vert. F.	L_free	Fixed Length
1. Tieback	5.5	30.0	8.0	69.9	60.6	35.0	8.7	8.9

UNITS: Width,Diameter,Spacing,Length,Depth,and Height - ft; Force - kip; Bond Strength and Pressure - ksf

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

Z1	P1	Z2	P2	Slope
0	.480	16	0.480	0
*				
* EQ	Load			
0	0.137	16	0.034	-0.006437

PASSIVE PRESSURES: Pressures below will be divided by a Factor of Safety =1.1

Z1	P1	Z2	P2	Slope
16	0	30	4.900	.350

ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	8.00
2	16.00	1.00

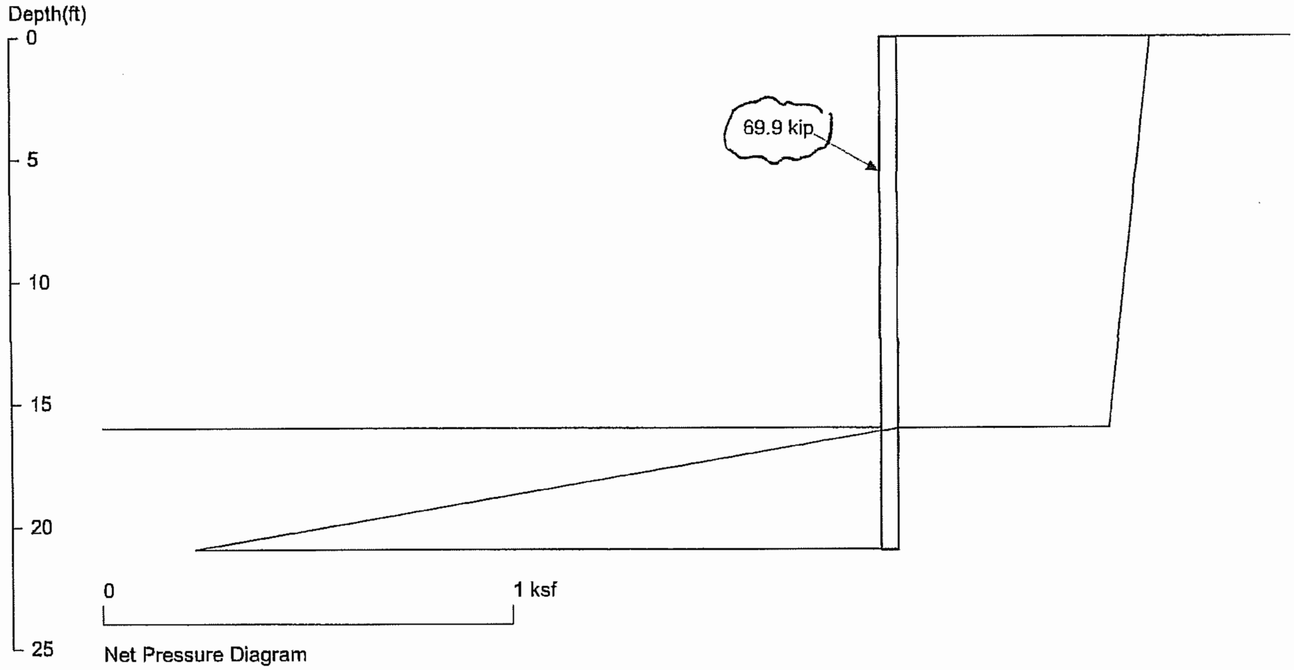
PASSIVE SPACING:

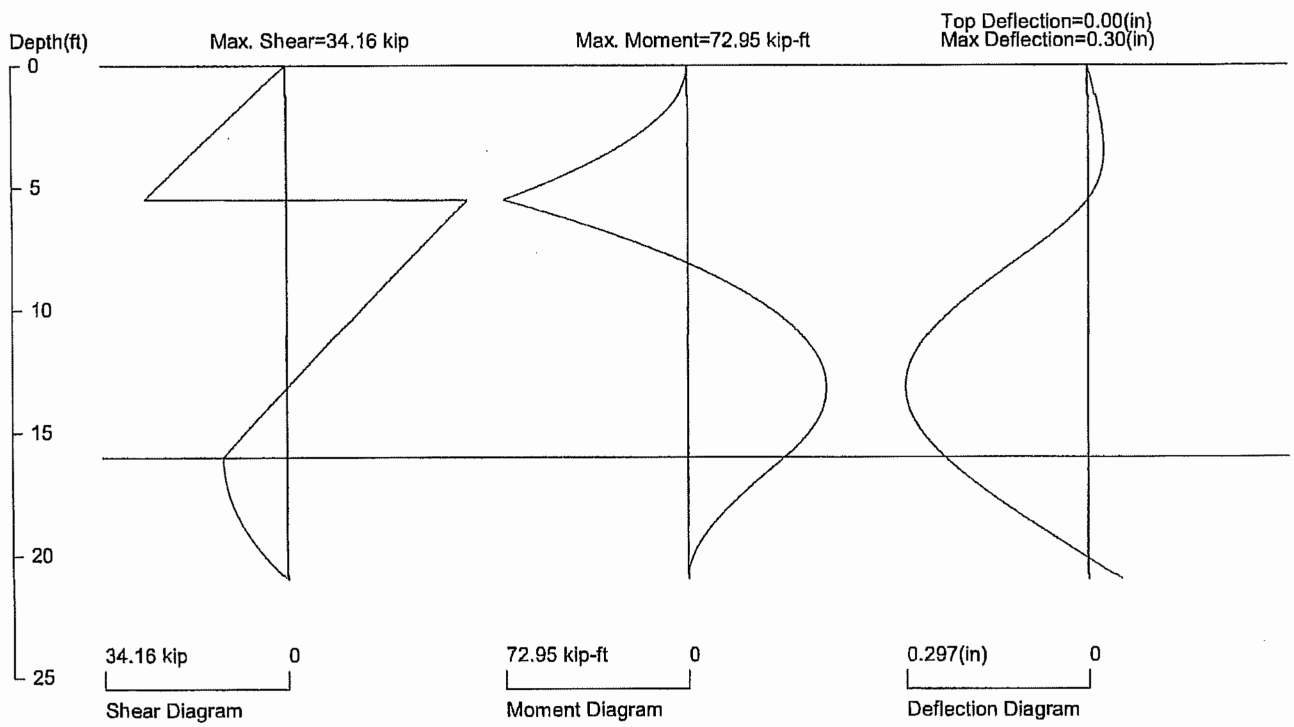
No.	Z depth	Spacing
1	16.00	3.00

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft3; Deflection - in

Moles SFR Pile & Tieback Wall

16' tall, Seismic Analysis





PRESSURE, SHEAR, MOMENT, AND DEFLECTION DIAGRAMS

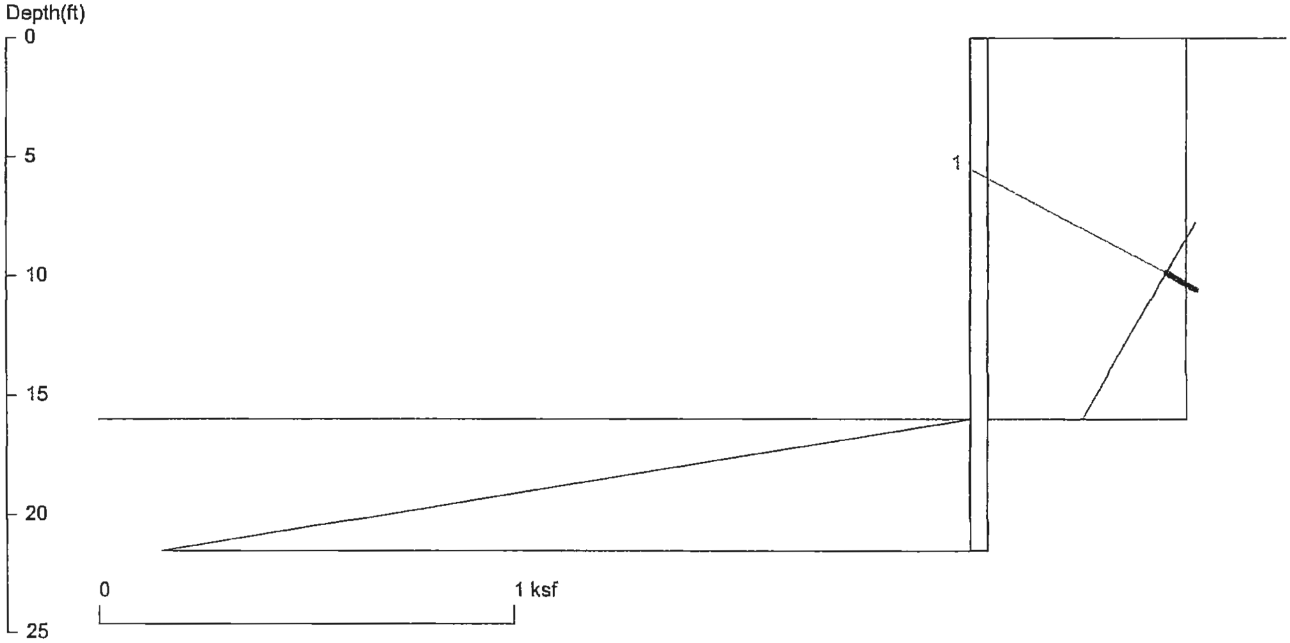
Based on pile spacing: 8.0 foot or meter

User Input Pile, W12x26: E (ksi)=29000.0, I (in4)/pile=204.0

File: P:\Projects\2017 Projects\17-028 (Moles SFR Failing Wall, 2501 NE Rasmussen Road, Corbett, OR) (TH)\Shoring Suite Stuff\16' wall (seismic).sh8

Moles SFR Pile & Tieback Wall

16' tall, Static Analysis



<ShoringSuite> CIVILTECH SOFTWARE USA www.civiltech.com

Licensed to 4324324234 3424343 Date: 3/26/2017
File: P:\Projects\2017 Projects\17-028 (Moles SFR Failing Wall, 2501 NE Rasmussen Road, Corbett, OR) (TH)\Shoring St

Wall Height=16.0 Pile Diameter=1.0 Pile Spacing=8.0 Wall Type: 3. Soldier Pile, Driving

PILE LENGTH: Min. Embedment=5.56 Min. Pile Length=21.56
MOMENT IN PILE: Max. Moment=57.93 per Pile Spacing=8.0 at Depth=5.49

PILE SELECTION:
Request Min. Section Modulus = 21.1 in3/pile=345.18 cm3/pile, Fy= 50 ksi = 345 MPa, Fb/Fy=0.66
-> Piles meet Min. Section Requirements: Top Deflection is shown in (in)
HP8X36 (-0.16) W8X28 (-0.19) HP10X42 (-0.09) W10X22 (-0.16) HP12X53 (-0.05)
W12X19 (-0.15) HP13X60 (-0.04) HP14X73 (-0.03) W14X22 (-0.09) W16X26 (-0.06)
HP16X88 (-0.02) W16X89 (-0.01) HP16X101 (-0.01) W16X100 (-0.01)

BRACE FORCE: Strut, Tieback, Plate Anchor, Deadman, Sheet Pile as Anchor

No. & Type	Depth	Angle	Space	Total F.	Horiz. F.	Vert. F.	L_free	Bedrock Fixed Length
1. Tieback	5.5	30.0	8.0	58.5	50.6	29.2	8.7	7.4

UNITS: Width,Diameter,Spacing,Length,Depth,and Height - ft; Force - kip; Bond Strength and Pressure - ksf

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

Z1	P1	Z2	P2	Slope
0	.480	16	0.480	0

PASSIVE PRESSURES: Pressures below will be divided by a Factor of Safety =1.5

Z1	P1	Z2	P2	Slope
16	0	30	4.900	.350

ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	8.00
2	16.00	1.00

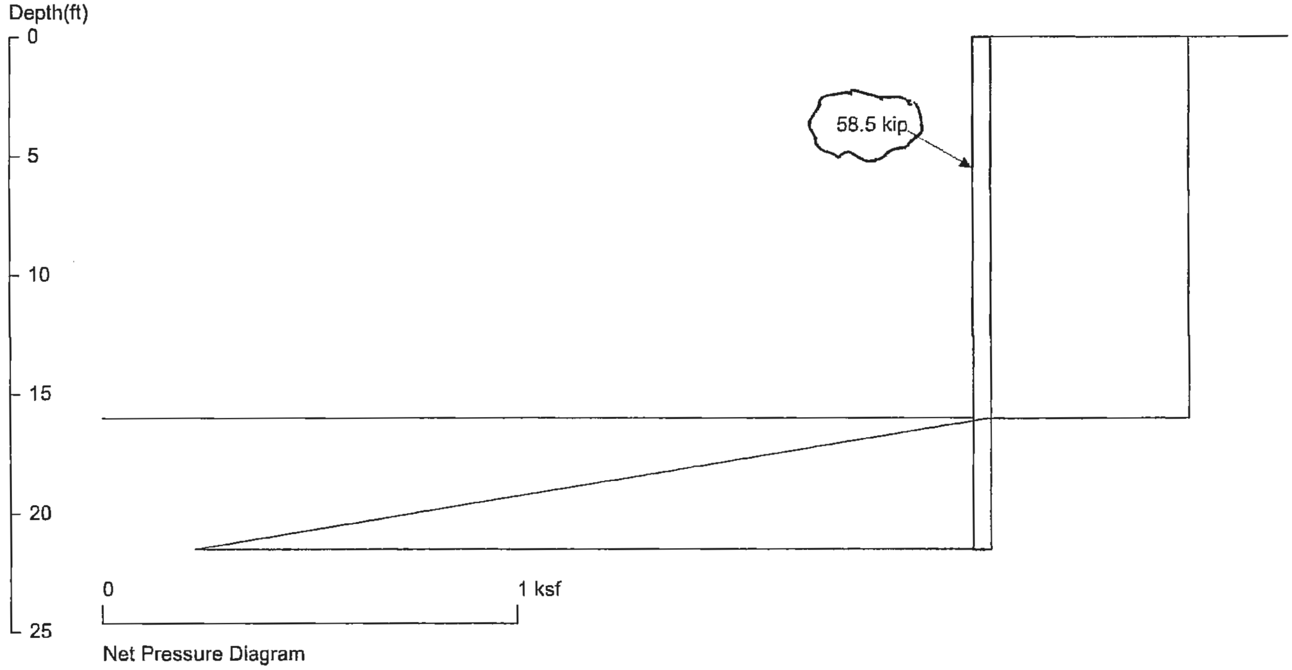
PASSIVE SPACING:

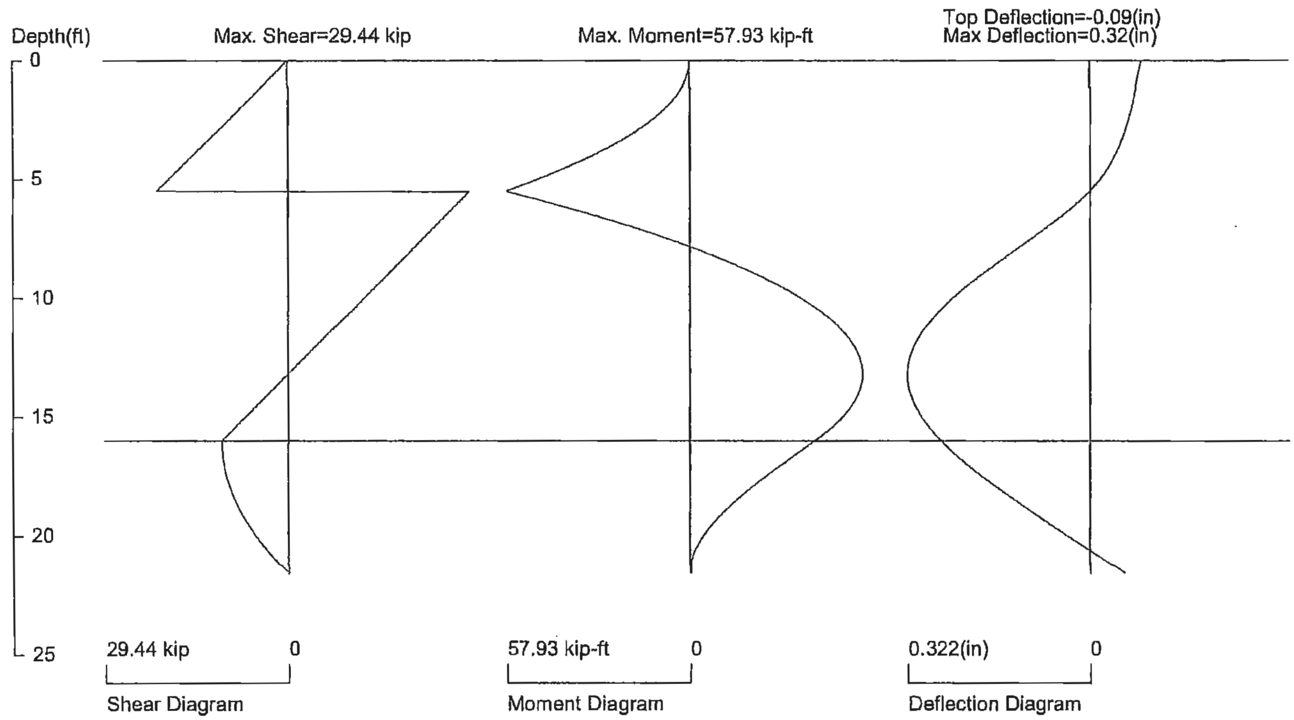
No.	Z depth	Spacing
1	16.00	3.00

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft3; Deflection - in

Moles SFR Pile & Tieback Wall

16' tall, Static Analysis





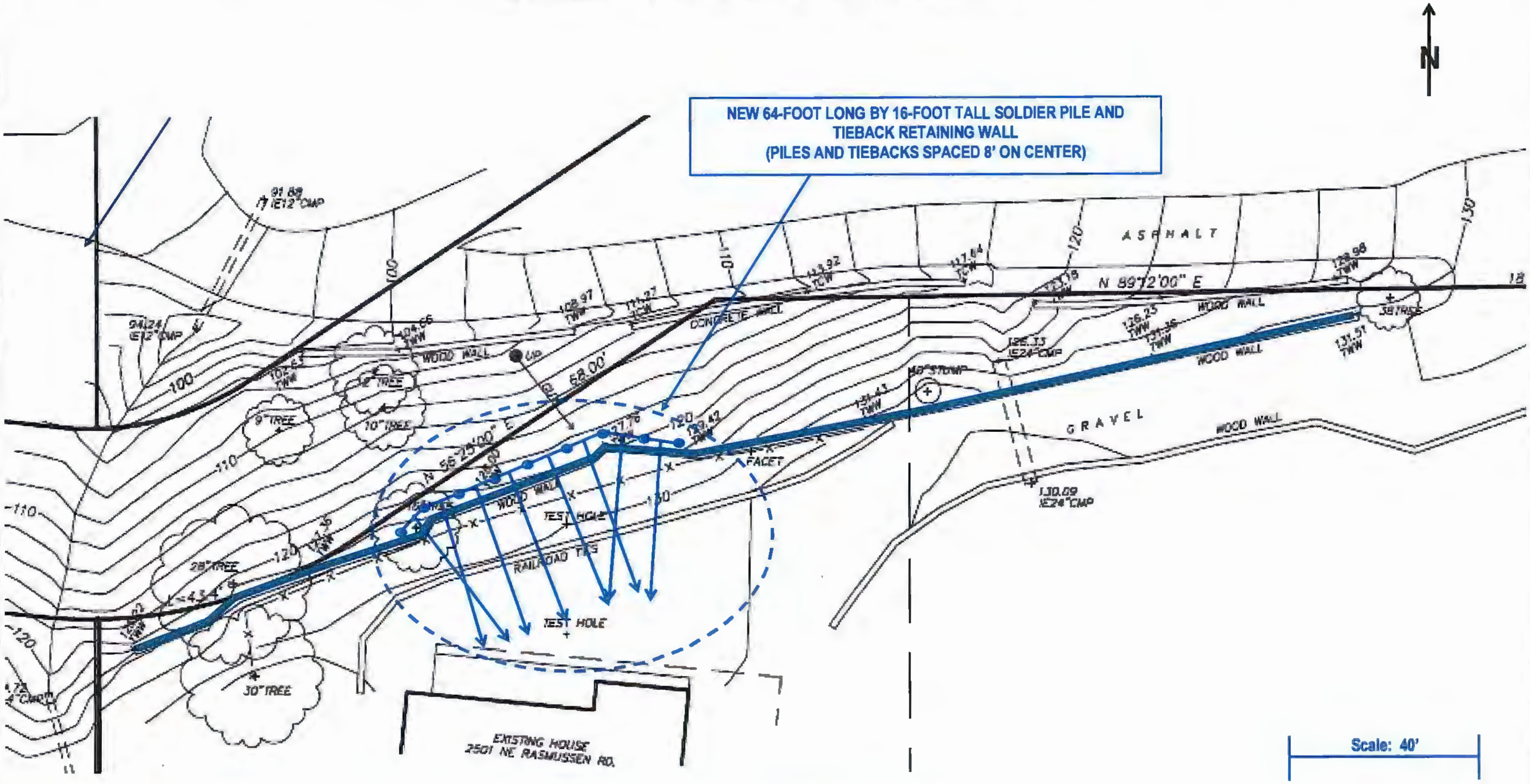
PRESSURE, SHEAR, MOMENT, AND DEFLECTION DIAGRAMS

Based on pile spacing: 8.0 foot or meter

User Input Pile, w12x26: E (ksi)=29000.0, I (in4)/pile=204.0

File: P:\Projects\2017 Projects\17-028 (Moles SFR Failing Wall, 2501 NE Rasmussen Road, Corbett, OR) (TH)\Shoring Suite Stuff\16' wall (static).sh8

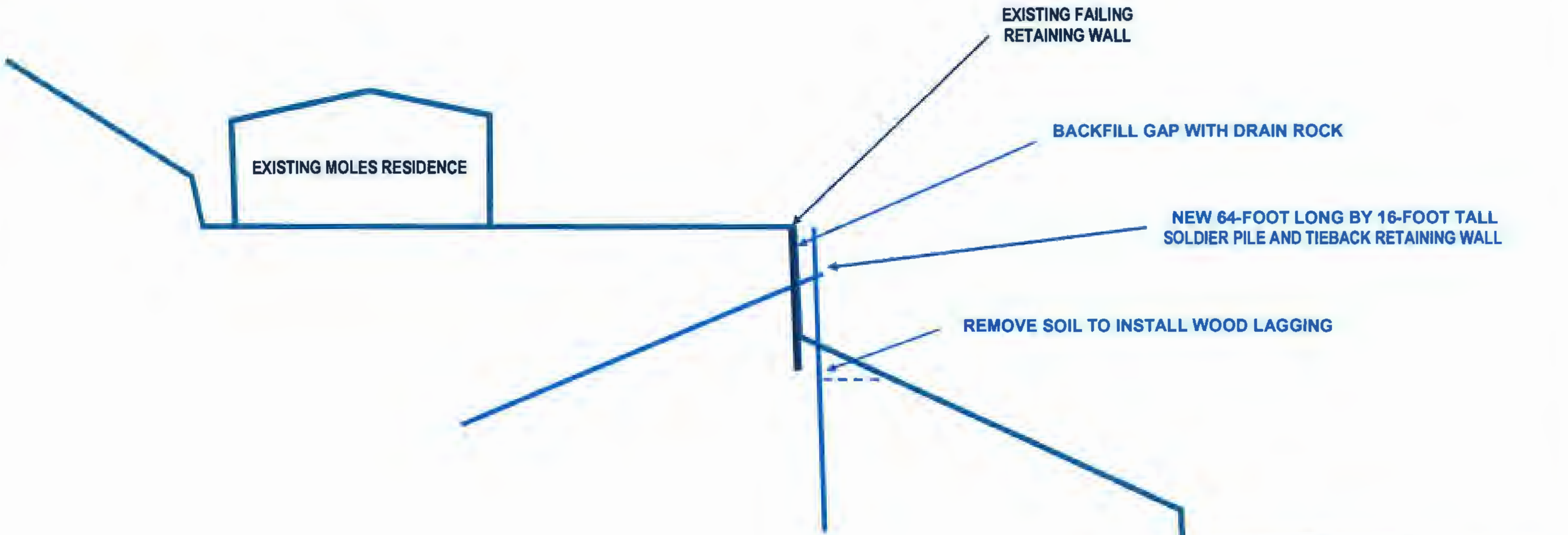
APPENDIX F – NEW RETAINING WALL LOCATION PLAN



Base drawing source: March 15, 2017 drawing titled "Moles Exhibit Map, Tax Lot 200 Map 1N4E27DA, in the SE 1/4 of Section 27, T.1N., R.4E., W.M., Multnomah County, Oregon," by Griffin Land Surveying Inc. of Beaverton, Oregon.

Earth Engineers, Inc.	Failing Retaining Wall Evaluation Moles Single Family Residence 2501 NE Rasmussen Road Corbett, Multnomah County, Oregon	EEI Report No. 17-028-1	March 27, 2017

APPENDIX G – NEW RETAINING WALL CROSS-SECTION



Not to scale.

Earth Engineers, Inc.	Failing Retaining Wall Evaluation Moles Single Family Residence 2501 NE Rasmussen Road Corbett, Multnomah County, Oregon	EEI Report No. 17-028-1	March 27, 2017