October 5, 2017

Project No. 5-917-17942-0

City of Bainbridge Island
280 Madison Avenue North
Bainbridge Island, Washington 98110

Attention: Mr. Peter Corelis, P.E.

Subject: Geotechnical Review of Documents
Dufresne - PLN50287 SVAR
11143 Rolling Bay Walk NE
Bainbridge Island, Washington 98110

Dear Mr. Corelis:

Amec Foster Wheeler Environment & Infrastructure, Inc. (Amec Foster Wheeler) has reviewed the geotechnical engineering reports prepared by Aspect Consulting, Perrone Consulting, Inc., P.S. and GeoEngineers. Specifically, we reviewed:

- *Geotechnical Report, Dufresne Residence*, by Aspect Consulting, dated July 20, 2017.
- *Geotechnical Engineering Consultation, Proposed Single Family Residence,* by Perrone Consulting, Inc., P.S., dated February 14, 2016.
- *Summary Letter, Geotechnical Construction Observation Services, Slide Debris Catchment Wall*, by GeoEngineers, dated September 8, 2006
- *Geotechnical Construction Observation Reports, No's.1-19, Slide Debris Catchment Wall,* by GeoEngineers, dated April 11 through June 6, 2006.
- *City of Bainbridge Island Permit Issuance Forms for 11129 and 11139 Rolling Bay Walk*, dated August 4, 2006.
- *Design Consultation, Proposed Slide Debris Catchment Wall,* by GeoEngineers, dated June 13, 2005.

Amec Foster Wheeler was requested to provide a third-party review of the reports to verify that they complied with the current Shoreline Master Program, Chapter 16.12 of the Bainbridge Island Municipal Code (BIMC), as detailed in your Notice to Proceed letter dated September 7, 2015. The review was requested due to the location of the subject site within a Geologically Hazardous Areas as defined by Subsection 16.20.060(K).

Amec Foster Wheeler Environment & Infrastructure, Inc.
600 University Street, Suite 600
Seattle, Washington 98101
(425) 368-1000 Phone
(425) 368-1001 Facsimile
www.amecfw.com

SITE AND PROJECT DESCRIPTION

The site is a half-acre lot located on the south side of Rolling Bay Walk NE. Dimensions are roughly 120 feet wide along the shoreline and extend roughly 165 feet upslope. The west half of the parcel was previously developed, but the residence at that time was destroyed by a landslide in 1997. The site slopes up from Rolling Bay Walk NE at an elevation of 12 feet to the southwest at a 2H:1V (Horizontal:Vertical) slope to the base of the first of two soldier pile tieback retaining walls at elevation 34 feet constructed in 2006. At the top of the first retaining wall (elevation ~40 feet), there is a relatively flat area for 16 feet to the toe of the second retaining wall also constructed in 2006 (elevation of 40 to 42 feet). This retaining wall was constructed to catch debris from a shallow landslide that could develop on the steep slope above the wall. Behind the catchment wall (elevation 50 feet), the inclination of the slope is 1H:1V to the top of slope (elevation 135 feet).

The proposed project is to construct a new single family house on the lower portion of the lot between Rolling Bay Walk NE and the lower soldier pile retaining wall. The structure will have a garage on the lower level with two-stories of living space above and auxiliary parking in front of the residence. A 12 to 15 foot cut will have to be made into the slope in order to construct the proposed residence. A septic drain field is proposed between the two existing soldier pile retaining walls. Stormwater from impervious surfaces will be collected into a tightline and conveyed to the proper collection system.

Soil conditions described by Aspect from their field work generally consisted of 7 to 15 feet of loose landslide debris, mantling medium dense beach sands to depths of 14 to 15 feet below the ground surface. Beneath the landslide and beach deposits are dense advance outwash sands to the full depth of their explorations. Aspect did not notice any recent signs (post 1997) of slope movement above the upper soldier pile retaining wall during their site visits, but did indicate they observed leaning and trunk curvature of trees in the downslope direction suggesting slope creep in the upper 1 or 2 feet of the slope. Slope stability analyses indicated the retaining walls and the proposed development will maintain sufficient factors of safety for deep-seated stability of the slope.

REVIEW COMMENTS

Amec Foster Wheeler reviewed the geotechnical engineering reports and associated documents to determine if they meet the criteria specified within Shoreline Master Program, Chapter 16.12 Subsection 16.20.060(K). This subsection details the requirements for a critical areas assessment and geotechnical reports to be submitted for proposed developments in the Geologically Hazardous Areas. Although Aspect's report and supporting historical documents provided most of the information needed, we have a few comments and requests for additional information.

1. BIMC 16.12.060(K)(4)(a)(v)A: Aspect describes the basic development concepts, but they should also review the construction plans. As part of the building permit review process, we recommend that Aspect review all pertinent plans (site grading, drainage, and foundations) and provide a plan review letter confirming that the plans incorporate all of the recommendations presented in the geotechnical engineering report.
2. BIMC 16.12.060(K)(4)(a)(v)A: We recommend that Aspect comment on the current status of the freeboard height and length of the catchment wall. They should also provide recommendations for monitoring and maintenance of the catchment wall.
3. BIMC 16.12.060(K)(4)(a)(v)D: Aspect's Geotechnical Report did not discuss critical area buffers or building setbacks. We request they provide buffer and setback recommendations.
4. BIMC 16.12.060(K)(4)(b)(ii)(B)3: Aspect's Geotechnical Report discusses the location of the drain field between the two existing soldier pile walls. They indicate that the effluent water will be handled by, "the wood lagging facing of the lower catchment wall is free-draining and will prevent the buildup of unbalanced hydrostatic pressures on the wall". Please clarify if the design intends to have sewage effluent seep through the retaining wall facing to the ground surface. Please describe where the sewage effluent will go after it seeps through the face of the wall. If the design is not intended to allow sewage effluent to seep through the wall to the surface, then please explain the drainage design. Also note that heavy equipment will utilize the bench between the two retaining walls to remove debris from the upper catchment wall, so the drainfield must be designed to accommodate heavy vehicle loads.
5. BIMC 16.12.060(K)(4)(d)ii: Although Aspect's report states the IBC Seismic Site Class is C, from our review of the boring logs it appears to be Class D. We recommend Aspect revise the seismic recommendations to Class D, or provide calculations supporting the Class C determination.
6. BIMC 16.12.060(K)(4)e: Aspect's Geotechnical Report should include comments on the impact of tsunami hazards.
7. BIMC 16.12.060(K)(5)(a)iv: Aspect predicts liquefaction and lateral spreading to occur during the design earthquake, and they have provided foundation recommendations that will prevent collapse of the structure, but will allow structural damage to the house. They stated that recommendations could be provided to fully mitigate lateral spreading but said that may be cost prohibitive. We recommend that Aspect clarify whether their foundation recommendations provide the "highest standard of safety feasible," as stated in the code.

8. We recommend that Aspect provide construction monitoring of geotechnical related tasks such as: erosion control; excavation; shoring; pin pile installation, and drainage.

CLOSURE

It should be noted that our scope of work for this letter was limited to a review of the documents supplied to us. Our scope did *not* include a site visit, exploration of actual subsurface conditions, nor does our review purport to verify the accuracy of the geotechnical engineering results presented within the documents.

We hope this letter meets your current needs. If you have any questions, please do not hesitate to contact us at your convenience.

Sincerely,

Amec Foster Wheeler Environment & Infrastructure, Inc.

Todd D. Wentworth, P.E., L.G.
Associate Geotechnical Engineer

Henry W, Brenniman, L.E.G.
Senior Geologist


11096
11090
11184
003-02
003-00
005-03
11172
004-01
004-07
11149
Rolling Bay Walk NE
004-05
004-08
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Addressed renumbered
to reflex actual
physical add
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11016
007-01
008-06
006-02
11010
11004
11008
008-07
Mountain View Rd NE
006-01
004-01
11025
003-00
004-03
001-01

#B101
BUILDING PERMIT APPLICATION

BLD PERMIT #	VALUATION	RELATED PROJECT NUMBERS

Type of Work:

☑ New ☐ Addition ☐ Alteration ☐ Tenant ☐ Other______________

Section 1 – Project Information

Description of Work: Build New Single Family Residence

Enter the square footage (sq. ft.) for areas where work is to be done. The determination of building permit fees for projects reviewed by the City of Bainbridge Island Building Division will be based on valuation computed from these figures.

Area	Sq.Ft. - New	Sq.Ft. - Remodel
1st floor	1048	
2nd floor	1200	
Basement		
Garage/Carport	1080	
Garage 2nd fl – storage		
Deck	152	
Other:		
Total All Sq Ft Areas	3480	
Total New Impervious Surface within last 5 years.	1536	If over 800 square feet, drainage review is required.

# of Bedrooms		#of Dwelling Units	Roofing material: Asphalt Shingles
# of Bathrooms		Method of Heat (ie: electric/propane):	Mini Split

Check if installing any of the below:

☐ Sprinkler System ☐ Alarm ☐ Elevator

Section 2 - Property Information

Site Address: 11143 Rolling Bay Walk Assessor Tax Parcel Number(s): 4156 001 004 1006

Present Zoning: R-2 Present Use of Property: Undeveloped land

Lot Size: 19,116 Lot Coverage: 6.28% Impervious Surface: 1536

Amount of Proposed Grading/Fill: 457 cubic yards

Section 3 - Lender Information

Lender information required if construction financing cost exceeds $5,000.

Lender: NONE

Lender's address: ______________________

Lender's Phone: ______________ Lender's email: ______________

Section 4 – Applicant/Property Owner Information

Property Owner:

Name: Margaret Dufresne Address: 3866 NE HWY 104, Poulsbo ct 98370

Contact Phone #: 206 491-3917 Email Address: margdof@yahoo.com

Applicant: Note: For projects with multiple owners, attach a separate sheet with each owner(s) information and signatures.

☐ Owner ☐ Applicant (other than owner) ☐ Authorized Agent/Representative*

Name: ______ Address: ______

Contact Phone #: ______ Email Address: ______

Contractor Washington State allows homeowners to be their own general contractor. However, when choosing a contractor or subcontractor to perform work they are required to be registered with the Washington State Department of Labor and Industries. For more information about choosing and hiring a contractor visit http://www.lni.wa.gov/tradeslicensing/.

☐ Check if this is the Authorized Agent/Representative* for this project.

Name: ______ Title: ______

License Number: ______ Liability Certificate: ______

Address: ______

Contact Phone #: ______ Email Address: ______

*I authorize the listed contractor to perform those inspections the City has identified in the self-certification program. (Residential projects only)

Margaret Dufresne ______

Owner Signature Date

*The authorized agent/representative is the primary contact for all project-related questions and correspondence. The City will email requests and information about the application to the authorized agent/representative and will 'copy' (Cc) the owner noted below. The authorized agent/representative is responsible for communicating information to all parties involved with the application. It is the responsibility of the authorized agent/representative and owner to ensure their mailbox accepts City email (i.e., City email is not blocked or sent to "junk mail"). There may be instances where regular USPS or courier mail is used.

I affirm, under penalty of perjury, that all answers, statements, and information submitted with this application are correct and accurate to the best of my knowledge. I also affirm that I am the owner of the subject site. Further, as owner, I grant permission to any and all employees and representative of the City of Bainbridge Island and other governmental agencies to enter upon and inspect said property as reasonably necessary to process this application.

Margaret Dufresne Margaret Dufresne ______

Print Name (Owner) Signature (Owner) Date

______ ______ ______

Print Name (Owner) Signature (Owner) Date

Approvals	Initials	Date
Planning		
Building		
Drainage		
Other		

#B101

BUILDING PERMIT APPLICATION

BLD PERMIT #	VALUATION	RELATED PROJECT NUMBERS

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# of Bedrooms		#of Dwelling Units		Roofing material:	Asphalt Shingles
# of Bathrooms		Method of Heat (ie; electric/propane):			Mini Split

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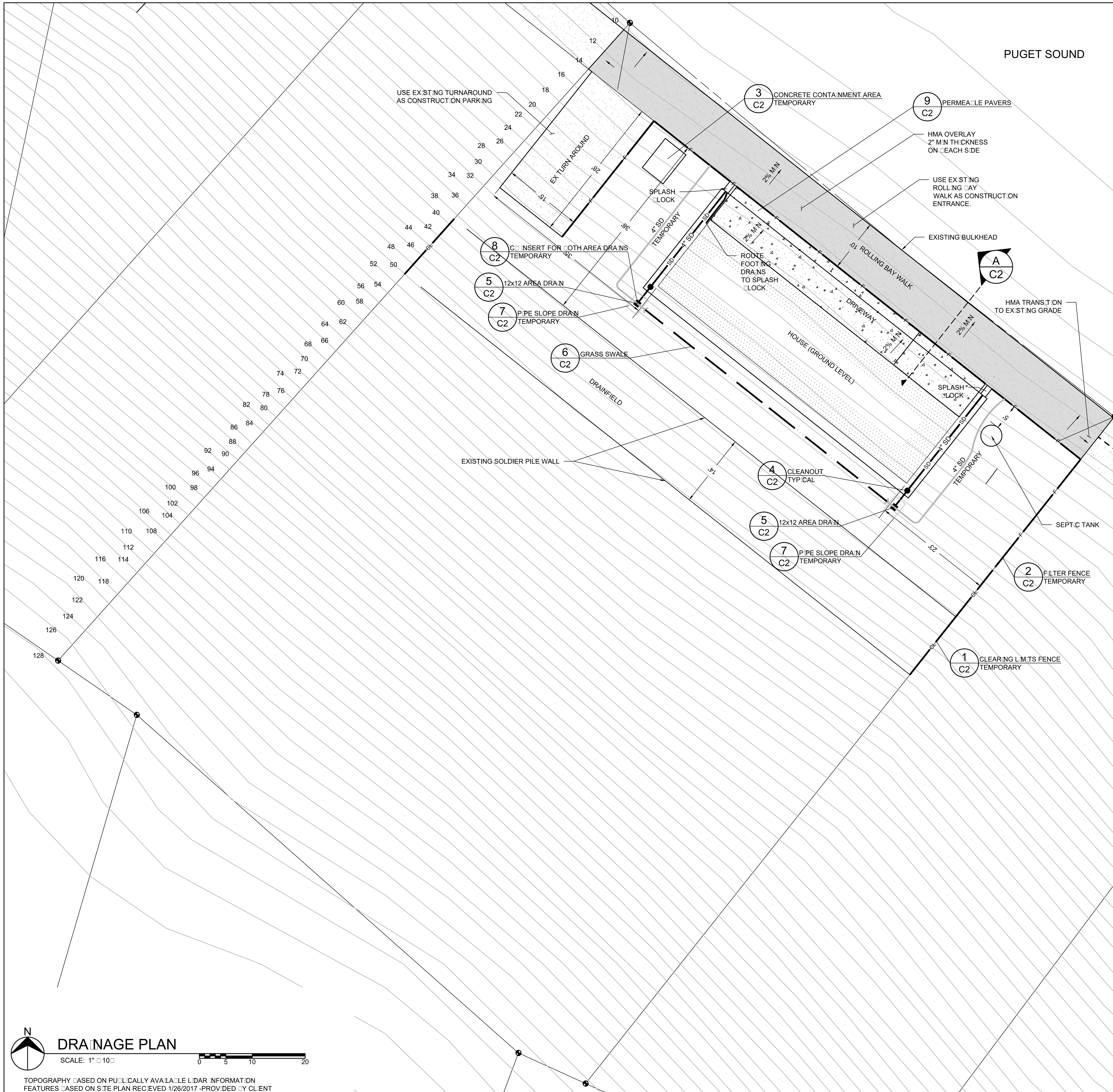


GENERAL NOTES

1. PROPERTY LINES AND EASEMENTS SHOWN ARE BASED ON INFORMATION PROVIDED AND OTHER EASEMENTS NOT SHOWN MAY EXIST. TOPOGRAPHY AND LOCATION OF FEATURES IS APPROXIMATE AND NOT BASED ON DETAILED SURVEY.

2. UNLESS OTHERWISE SPECIFIED, STORM DRAIN PIPE SHALL BE 4" DIAMETER, EITHER RIGID PVC (ASTM D3034) WITH THREE FEET OF COVER, OR SMOOTH INTERIOR CORRUGATED POLYETHYLENE (ADS N12 OR EQUIVALENT) WITH ONE FOOT OF COVER. OTHER PIPE MATERIALS SPECIFIED MAY NOT BE SUBSTITUTED. PIPE SHALL BE LAID AT A MINIMUM SLOPE OF 2% (1/4" PER FOOT UNLESS OTHERWISE NOTED). CARE SHALL BE TAKEN TO MAINTAIN MINIMUM SLOPE AND AVOID LOW POINTS ALONG THE PIPE.

3. ROOF DOWNSPOUTS SHALL BE CONNECTED TO ROOF DRAIN LINES SHOWN ACCORDING TO ARCHITECTURAL DETAILS.

4. CURTAIN AND FOUNDATION DRAINS MAY BE CONNECTED TO THE STORM DRAIN SYSTEM AT A POINT AT LEAST 1 FOOT BELOW THE LOWEST FOOTING.

5. GRADE DRIVEWAY AND PAVED AREAS TO DRAIN AWAY FROM BUILDINGS AND TOWARD CATCH BASINS AS SHOWN ON THE PLANS.

6. UNLESS OTHERWISE SPECIFIED, MATERIALS AND WORKMANSHIP SHALL BE IN ACCORDANCE WITH THE REQUIREMENTS OF THE CURRENTLY ADOPTED EDITION OF "STANDARD SPECIFICATIONS FOR ROAD, BRIDGE AND MUNICIPAL CONSTRUCTION", APWA/WSDOT EXCEPT AS MODIFIED BY THE CITY OF BAINBRIDGE ISLAND IN CURRENT EDITION OF "DESIGN AND CONSTRUCTION STANDARDS AND SPECIFICATIONS".

7. CONTRACTOR SHALL OBTAIN AN UNDERGROUND UTILITIES LOCATE PRIOR TO BEGINNING CONSTRUCTION (UNDERGROUND UTILITIES LOCATION SERVICE, 811).THE LOCATIONS OF EXISTING UNDERGROUND UTILITIES AND OTHER FEATURES ON THE PLAN ARE APPROXIMATE AND MAY NOT BE COMPLETE. ACTUAL LOCATIONS SHALL BE FIELD VERIFIED BY THE CONTRACTOR AS REQUIRED.

8. CONTRACTOR SHALL BE SOLELY RESPONSIBLE FOR WORKER SAFETY. ALL TRENCHING AND OTHER ACTIVITIES SHALL BE IN ACCORDANCE WITH STATE AND LOCAL SAFETY REGULATIONS AND REQUIREMENTS.

EROSION CONTROL NOTES

1. THE CONTRACTOR SHALL APPLY ALL MEASURES NECESSARY TO PREVENT THE DISCHARGE OF SEDIMENT-LADEN WATER OFF THE PROJECT SITE. FACILITIES SHOWN ON THE PLANS ARE THE MINIMUM REQUIREMENTS FOR ANTICIPATED SITE CONDITIONS.

2. THE CONTRACTOR SHALL INSPECT AND MAINTAIN ALL EROSION CONTROL FACILITIES REGULARLY, PARTICULARLY DURING AND FOLLOWING LARGE STORMS.

3. ALL STREETS ADJACENT TO THIS PROJECT SHALL BE KEPT CLEAN OF ALL MATERIAL DEPOSITS RESULTING FROM CONSTRUCTION.

4. SITE WORK SHALL BE SCHEDULED TO MINIMIZE THE EXPOSURE OF DISTURBED SOILS. ALL DISTURBED AREAS SHALL BE STABILIZED (CLEAR PLASTIC, MULCHING (SEE 5/C2) , ETC.) AS QUICKLY AS POSSIBLE AFTER COMPLETION OF WORK IN THE AREA. FROM OCTOBER 1 THROUGH APRIL 30, NO SOILS SHALL REMAIN EXPOSED FOR MORE THAN 2 DAYS. FROM MAY 1 THROUGH SEPTEMBER 30, NO SOILS SHALL REMAIN EXPOSED FOR MORE THAN 7 DAYS.

5. CARE SHALL BE TAKEN TO PREVENT ANY DISCHARGE OF SEDIMENT-LADEN WATER INTO THE STORMWATER SYSTEMS. ANY INLETS WHICH RECEIVE RUNOFF SHOULD BE PROTECTED WITH SEDIMENT FILTERS.

6. PETROLEUM PRODUCTS AND OTHER POTENTIAL POLLUTANTS SHALL BE PROTECTED TO PREVENT THEIR INTRODUCTION INTO SITE RUNOFF OR THE STORM DRAINAGE SYSTEM.

7. THE CONTRACTOR SHALL DEVELOP A DEWATERING PLAN TO PREVENT SEDIMENT LADEN WATER FROM DISCHARGING TO THE BEACH.

8. CLEARING LIMITS, IF SHOWN, SHALL BE CLEARLY FLAGGED PRIOR TO ANY CLEARING OR CONSTRUCTION ON THE SITE. DURING CONSTRUCTION, NO DISTURBANCE BEYOND THE FLAGGED CLEARING LIMITS SHALL BE PERMITTED. THE FLAGGING SHALL BE MAINTAINED BY THE CONTRACTOR UNTIL ALL CONSTRUCTION IS APPROVED.

9. ALL TEMPORARY EROSION CONTROL FACILITIES, INCLUDING PERIMETER CONTROLS, SHALL REMAIN IN PLACE UNTIL FINAL SITE CONSTRUCTION IS COMPLETE AND APPROVAL HAS BEEN RECEIVED FROM THE CITY.

SOIL AMENDMENT NOTES

AREAS TO BE DISTURBED SHALL HAVE THE TOPSOIL STRIPPED AND STOCKPILED. ALL DISTURBED AREAS THAT ARE TO BE LANDSCAPED SHALL HAVE THE STOCKPILED SOIL REPLACED OR SHALL BE AMENDED WITH COMPOST.

AREAS WHERE THE TOPSOIL WILL BE REPLACED SHALL HAVE THE SUBSOILS SCARIFIED TO 4 INCHES AND A MINIMUM DEPTH OF 8 INCHES OF TOPSOIL SHALL BE PLACED ON TOP OF SUBSOILS. 1-INCH OF COMPOSTED MATERIAL SHALL BE TILLED INTO THE TOP 4 INCHES OF TOPSOIL.

AMENDING THE SUBSOIL SHALL CONSIST OF SCARIFYING THE TOP 8-INCHES OF SOIL AND TILLING IN THE AMOUNT OF COMPOST DESCRIBED BELOW.

LANDSCAPED AREAS (10% ORGANIC CONTENT): PLACE AND TILL 3-INCHES OF COMPOSTED MATERIAL INTO TOP 5-INCHES OF SOIL. RAKE BEDS SMOOTH, REMOVE ROCKS LARGER THAN 2-INCHES IN DIAMETER AND MULCH AREAS WITH 2-INCHES OF ORGANIC MULCH.

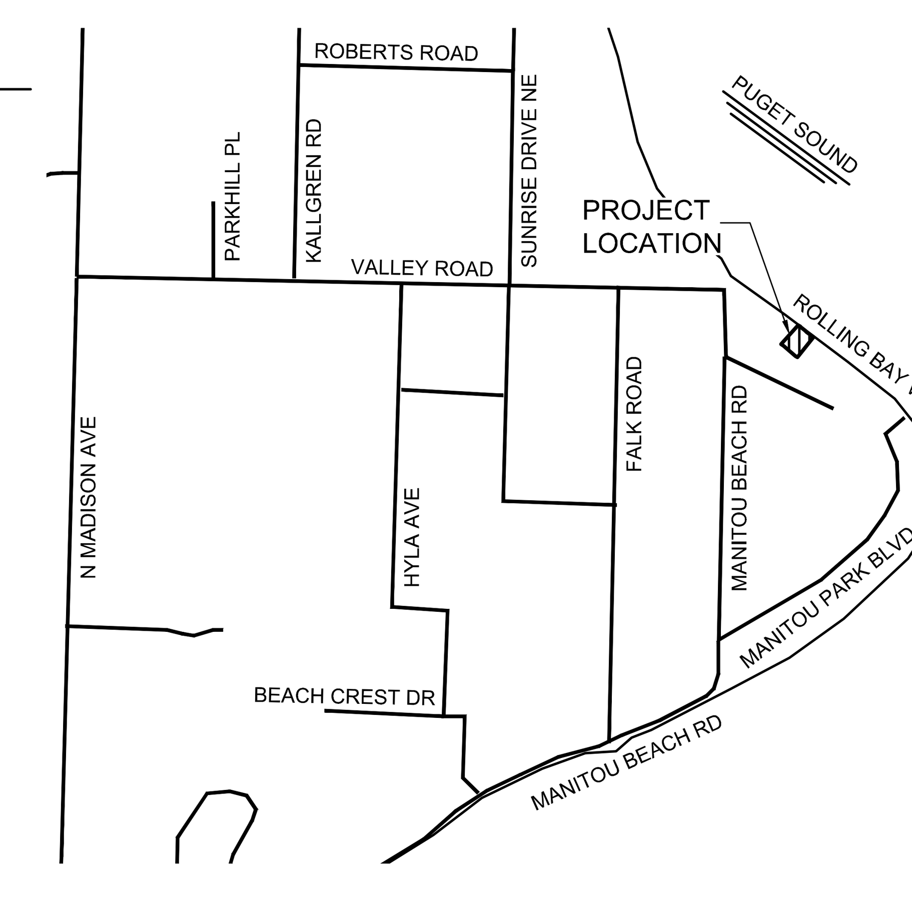
TURF AREAS (5% ORGANIC CONTENT) : PLACE AND TILL 1.75-INCHES OF COMPOSTED MATERIAL INTO 6.25-INCHES OF SOIL . WATER OR ROLL TO COMPACT SOIL TO 85% OF MAXIMUM. RAKE TO LEVEL, AND REMOVE SURFACE WOODY DEBRIS AND ROCKS LARGER THAN 1-INCH IN DIAMETER.

DO NOT SCARIFY SOIL WITHIN THE DRIP-LINE OF EXISTING TREES TO BE RETAINED. WITHIN 3-FEET OF THE TREE DRIP-LINE, AMENDMENT SHOULD BE INCORPORATED NO DEEPER THAN 3 TO 4-INCHES TO REDUCE DAMAGE TO ROOTS.

MATURE COMPOST SHALL BE GRADE A COMPOST MEETING WSDOT STD SPEC 9-14.4(8).

LEGEND

	EXISTING HOUSE/DRIVEWAY
	EXISTING MAJOR CONTOUR
	EXISTING MINOR CONTOUR
	PROPERTY LINE
	PROPERTY CORNER/STAKE
CL	PROPOSED CLEARING LIMITS FENCE
F	PROPOSED SILT FENCE
	PROPOSED CLEANOUT
SD	PROPOSED STORM DRAIN
	PROPOSED DITCH
	PROPOSED BUILDING

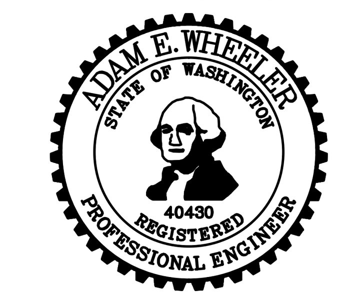


VICINITY MAP

SCALE: 1" = 1000'

0 500 1000 2000

PRELIMINARY
NOT FOR CONSTRUCTION



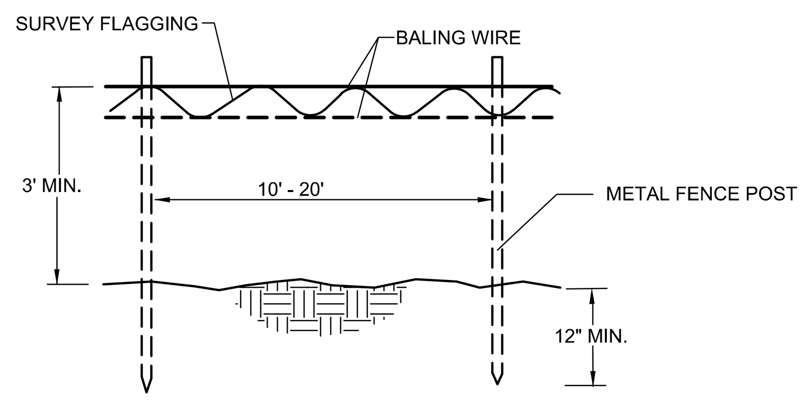
DRAINAGE PLAN
DUFRESNE RESIDENCE
BUILDING PERMIT SUBMITTAL

BROWNE • WHEELER ENGINEERS, INC
241 ERICKSEN AVENUE NE
BAINBRIDGE ISLAND, WA 98110
P 206.842.0605 INFO@BrowneWheeler.COM

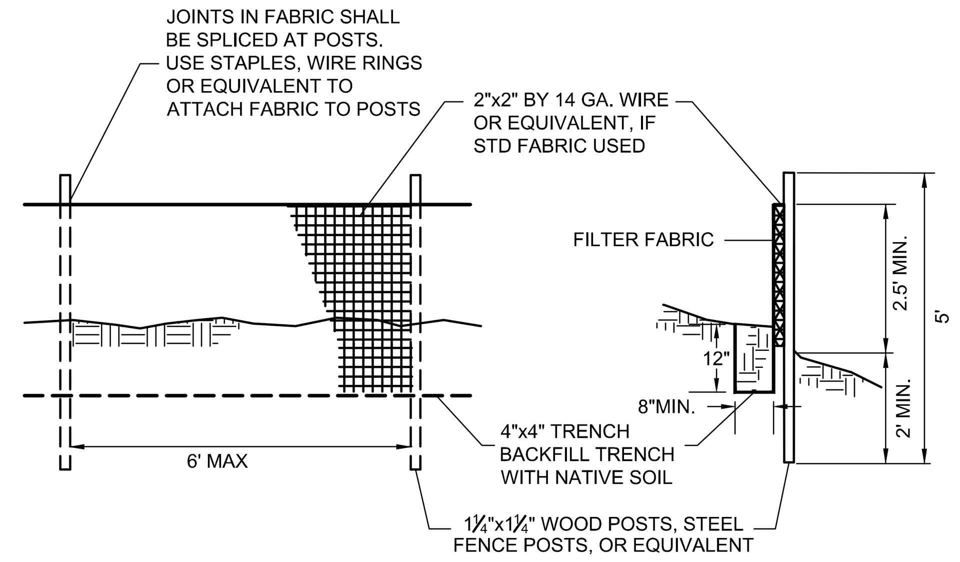
MARGARET DUFRESNE
3912 NE STATE HIGHWAY 104
POULSBO WA 98370

DATE 2/15/2017
DESIGNED AEW
DRAWN NDW
CHECKED AEW
PROJECT BDU05-001

C1

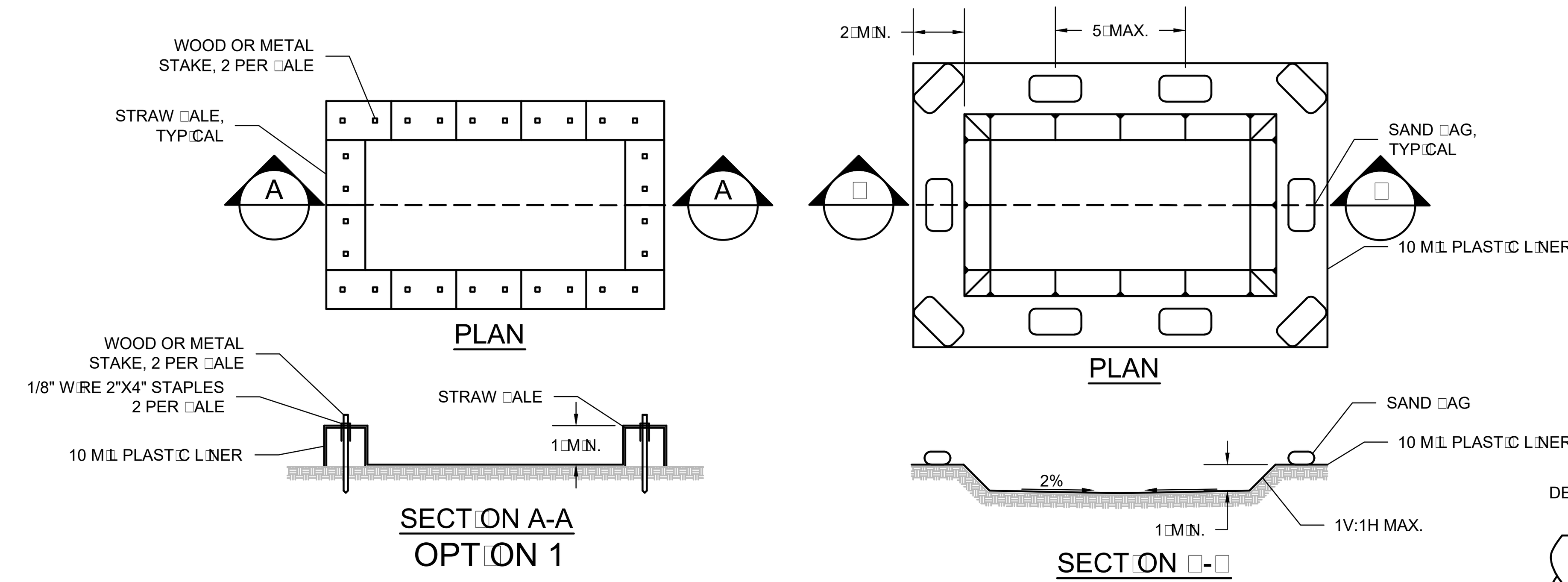


1 / C2 CLEARING LIMITS FENCE
NO SCALE

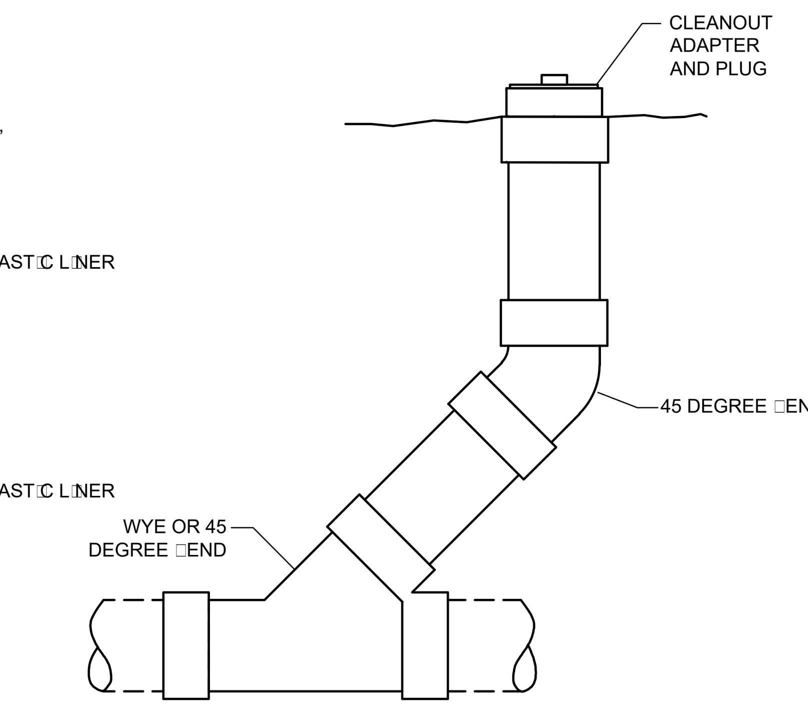


NOTES:
1. MAXIMIZE DETENTION OF STORMWATER BY PLACING FENCE AS FAR AWAY FROM TOE OF SLOPE AS POSSIBLE WITHOUT ENCROACHING ON SENSITIVE AREAS OR OUTSIDE CLEARING BOUNDARIES.
2. INSTALL SILT FENCING ALONG CONTOURS.
3. INSTALL THE ENDS OF THE SILT FENCE TO POINT SLIGHTLY UP-SLOPE TO PREVENT SEDIMENT FROM FLOWING AROUND THE ENDS OF THE FENCE.
4. PERFORM MAINTENANCE IN ACCORDANCE WITH STANDARD SPECIFICATIONS 8.01.3(9)A AND 8.01.3(15).
5. FILTER FABRIC SHALL MEET WSDOT 9-33.2(1) TABLE 6.

2 / C2 SILT FENCE
NO SCALE



3 / C2 CONCRETE CONTAINMENT AREA
NO SCALE

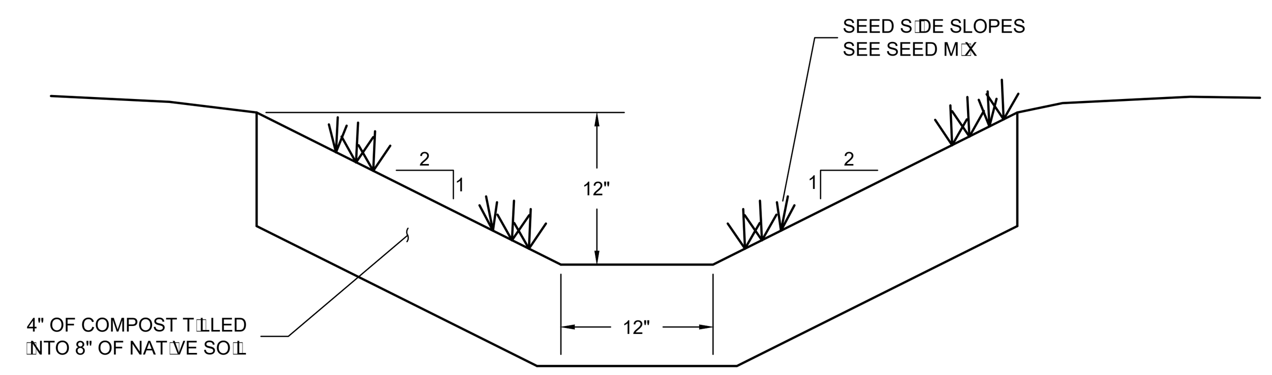


4 / C2 CLEANOUT
NO SCALE

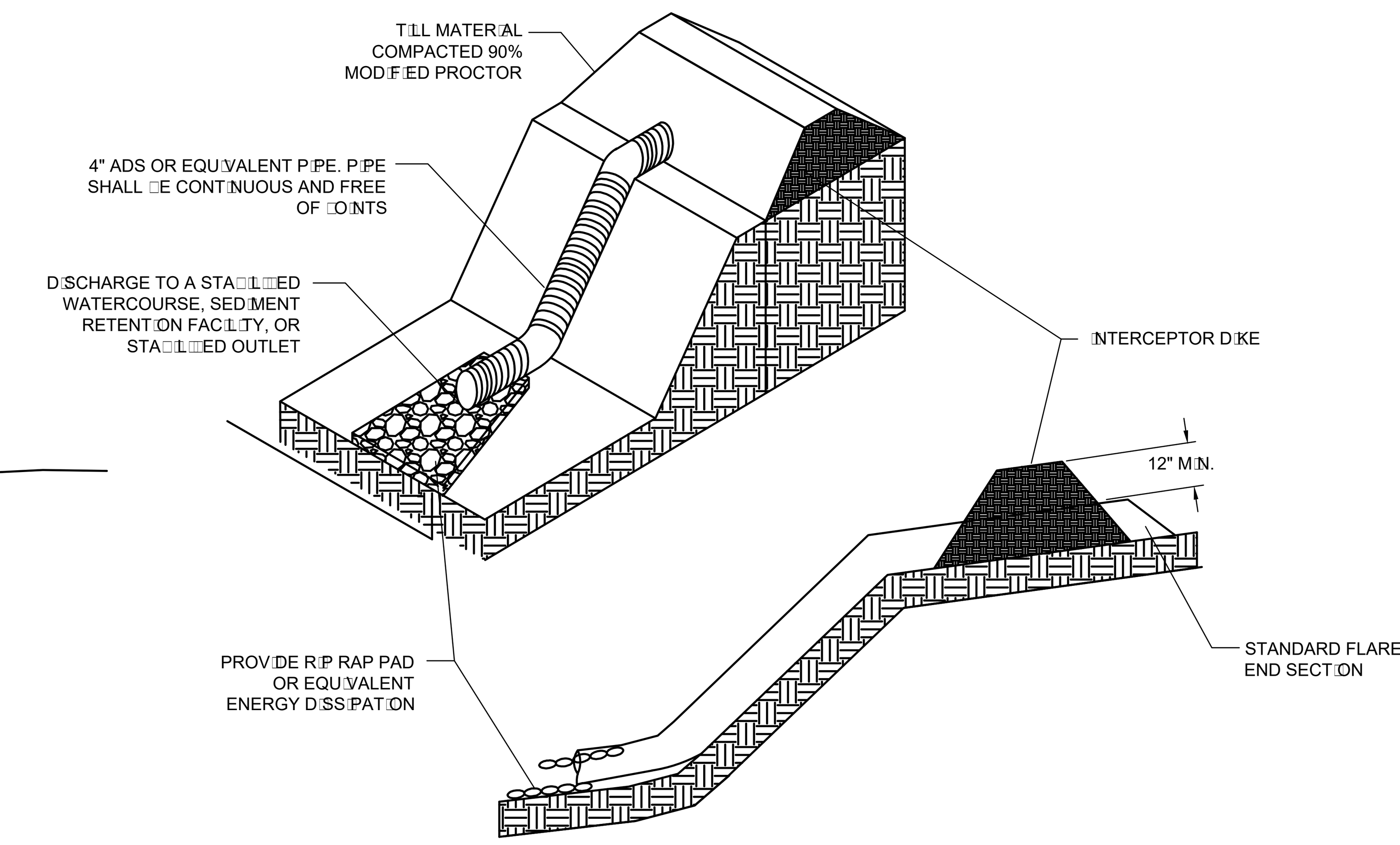
MULCH REQUIREMENTS		
MULCH MATERIAL	QUALITY STANDARDS	APPLICATION RATES
STRAW	AIR-DRIED FREE FROM UNDESIRABLE SEED AND COARSE MATERIAL.	2"-3" THICK 5 BALES PER 1000 SF OR 2-3 TONS PER ACRE
HYDROMULCH	NO GROWTH INHIBITING FACTORS.	APPROX. 25-30 LBS PER 1000 SF OR 1500-2000 LBS PER ACRE
COMPOSTED MULCH AND COMPOST	NO VISIBLE WATER OR DUST DURING HANDLING. MUST BE PURCHASED FROM SUPPLIER WITH SOLID WASTE HANDLING PERMIT (UNLESS EXEMPT).	2" THICK MIN. APPROX. 100 TONS PER ACRE (APPROX 800 LBS PER YARD)
CHIPPED SITE VEGETATION	AVERAGE SIZE SHALL BE SEVERAL INCHES. GRADATIONS FROM FINES TO 6 INCHES IN LENGTH FOR TEXTURE, VARIATION, AND INTERLOCKING PROPERTIES.	2" MINIMUM THICKNESS
WOOD-BASED MULCH	NO VISIBLE WATER OR DUST DURING HANDLING. MUST BE PURCHASED FROM A SUPPLIER WITH A SOLID WASTE HANDLING PERMIT OR ONE EXEMPT FROM SOLID WASTE REGULATIONS.	2" THICK APPROX. 100 TONS PER ACRE (APPROX. 800 LBS PER CUBIC YARD)

5 / C2 MULCHING
NO SCALE

SEED MIX			
SEED TYPE	% WEIGHT	% PURITY	% GERMINATION
TALL OR MEADOW FESCUE FESTUCA ARUNDINACEA OR FESTUCA ELATIOR	75-80	98	90
SEASIDE/CREEPING BENTGRASS AGROSTIS PALUSTRIS	10-15	92	85
REDTOP BENTGRASS AGROSTIS ALBA OR AGROSTIS GIGANTEA	5-10	90	80

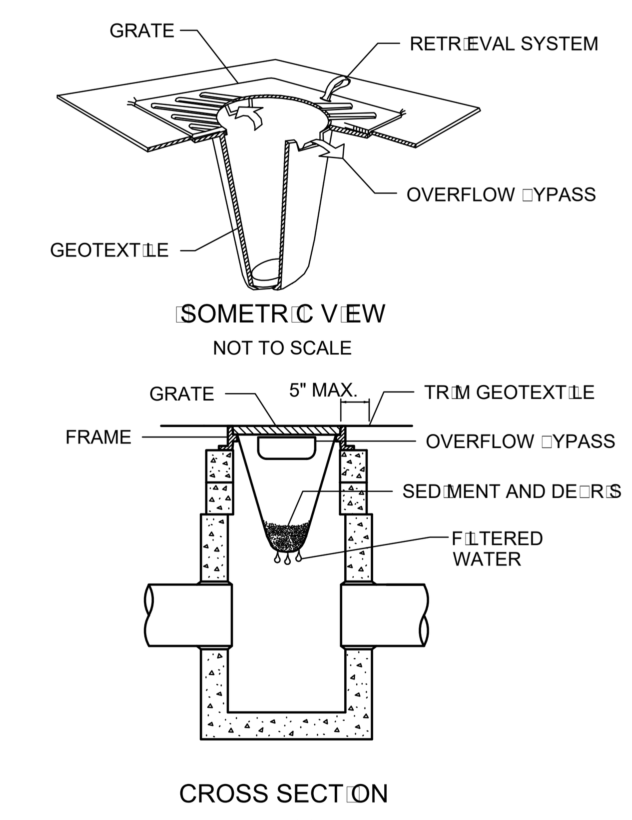


6 / C2 GRASS SWALE
NO SCALE



7 / C2 PIPE SLOPE DRAIN
NO SCALE

NOTES
1. PERFORM MAINTENANCE IN ACCORDANCE WITH STANDARD SPECIFICATION 8-01.3(15).
2. SIZE THE BELOW GRATE INLET DEVICE (BGID) FOR THE STORM WATER STRUCTURE IT WILL SERVICE.
3. THE BGID SHALL HAVE A BUILT-IN HIGH-FLOW RELIEF SYSTEM (OVERFLOW BYPASS).
4. THE RETRIEVAL SYSTEM MUST ALLOW REMOVAL OF THE BGID WITHOUT SPILLING THE COLLECTED MATERIAL.
5. INSTALL SOCK IN AND EXISTING OR PROPOSED CATCH BASINS WHICH MAY RECEIVE RUNOFF FROM THE PROJECT SITE



8 / C2 CATCH BASIN SOCK
NO SCALE

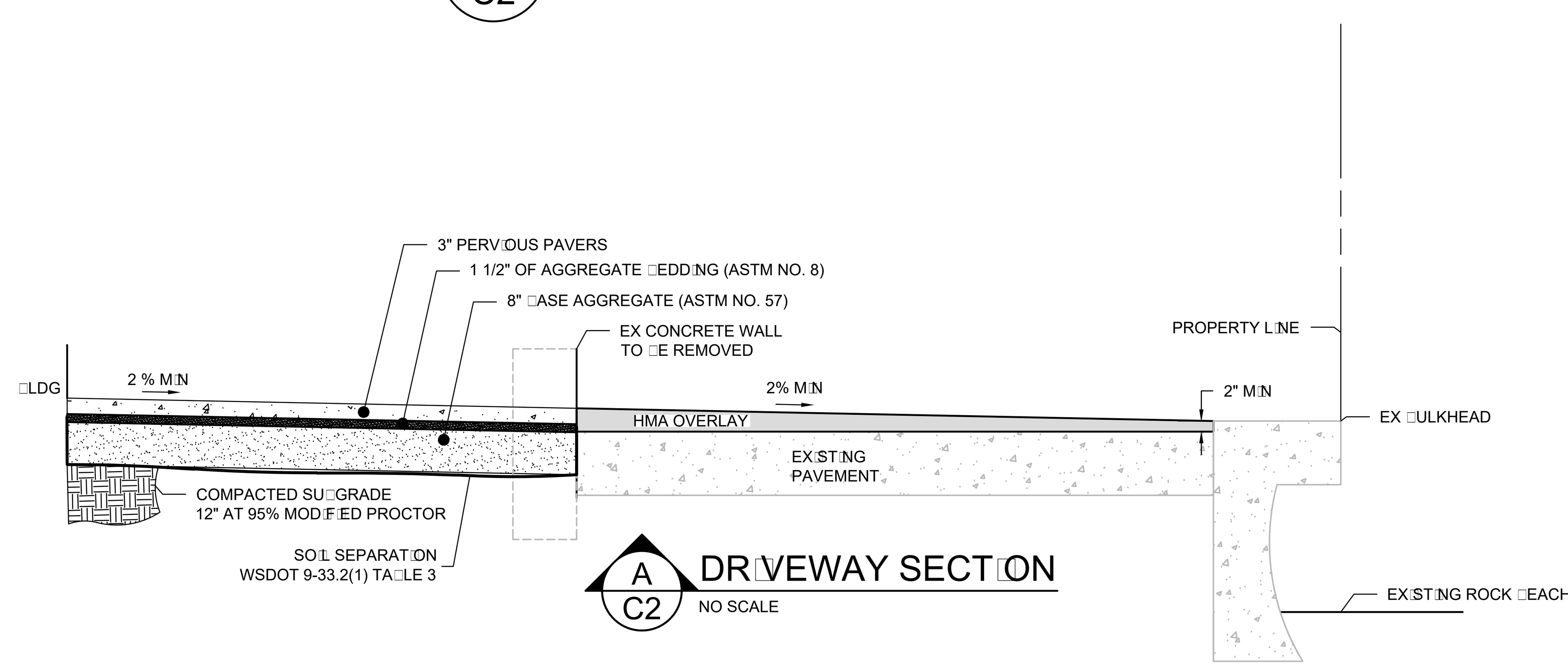
BASE AGGREGATE MIX+	
U.S. STD. SIEVE	% PASSING
1 1/2"	100
1"	95-100
1/2"	25-60
#4	0-10
#8	0-5

+ BASE AGGREGATE SHOULD BE 3/8" TO 3/4" UNIFORMLY GRADED STONE WITH A WASH LOSS OF NO MORE THEN 0.5% (AASHTO SIZE #57).

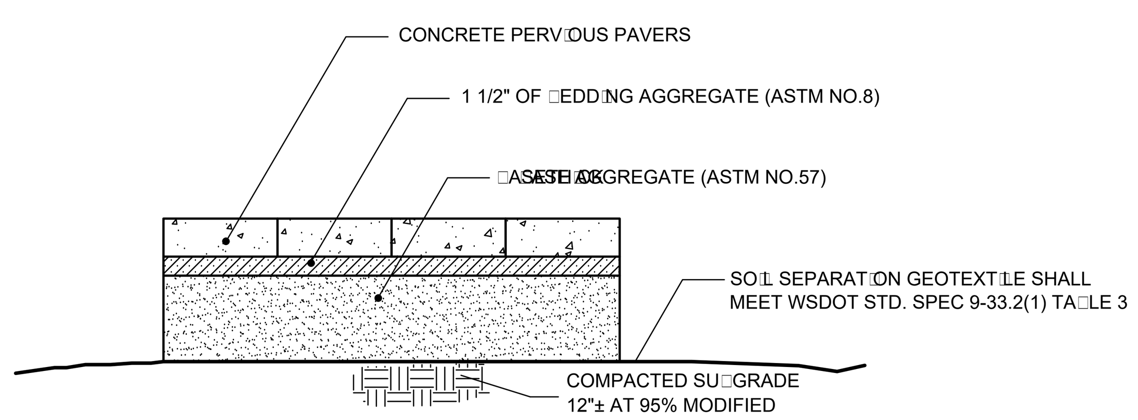
BEDDING AGGREGATE MIX*	
U.S. STD. SIEVE	% PASSING
1/2"	100
3/8"	85-100
#4	10-30
#8	0-10
#16	0-5

* BEDDING AGGREGATE SHOULD BE 3/8" UNIFORMLY GRADED STONE WITH A WASH LOSS OF NO MORE THEN 0.5% (AASHTO SIZE #8).

NOTES
1. INSTALL PAVERS IN ACCORDANCE WITH THE MANUFACTURER'S SPECIFICATION AND RECOMMENDATIONS.
2. REMOVE AND REPLACE UNSUITABLE MATERIAL AS DIRECTED BY GEOTECHNICAL ENGINEER.
3. CONTRACTOR SHALL PROTECT PERVIOUS PAVING AREA DURING CONSTRUCTION AND AFTER INSTALLATION FROM COMPACTION AND SOIL CONTAMINATION OF BOTH THE PERVIOUS PAVING AND THE UNDERLYING NATIVE SOIL. IF THE CONTRACTOR FAILS TO DO THIS THE COMPACTED MATERIAL OR CONTAMINATED SOIL SHALL BE REMOVED AND REPLACED WITH BEDDING AGGREGATE.



A / C2 DRIVEWAY SECTION
NO SCALE



9 / C2 PERMEABLE PAVERS
NO SCALE

PRELIMINARY
NOT FOR CONSTRUCTION

DETAILS
DUFRESNE RESIDENCE
BUILDING PERMIT APPLICATION

BROWNE • WHEELER ENGINEERS, INC
241 ERICKSEN AVENUE NE
BAINBRIDGE ISLAND, WA 98110
P 206.842.0605 INFO@BrowneWheeler.COM

MARGARET DUFRESNE
3912 NE STATE HIGHWAY 104
POULSBO WA 98370

DATE	2-15-2017
DESIGNED	AEW
DRAWN	NDW
CHECKED	AEW
PROJECT #	DU05-001

C2

LUCIA ENGINEERING INC.

7307 12th Avenue NE
Seattle, Washington 98115
Phone: 206.790.8039

Dufrense Residence
11143 Rolling Bay Walk NE
Bainbridge Island, Washington 98110

October 18, 2017

Attention: **Margaret Dufrense**

Reference: **New Residence**
Design of Permanent Soldier Pile Wall
Lateral & Vertical Member Design for the New Residence

Margaret,

Attached are the design calculations and plans for the Permanent Soldier Pile Wall and Lateral & Vertical Member Design for the New Residence.

Please contact me with any questions.

JOSEPH M. LUCIA
STATE OF WASHINGTON
23314
REGISTERED
PROFESSIONAL ENGINEER

October 18, 2017
Joseph M. Lucia, P.E.

Kitsap County Parcel Details

| Search | Details | Maps | Links | Tax Bill | Print | Download |

General

Parcel #: 4156-001-004-1006

11143 ROLLING BAY WALK NE
BAINBRIDGE ISLAND, WA 98110

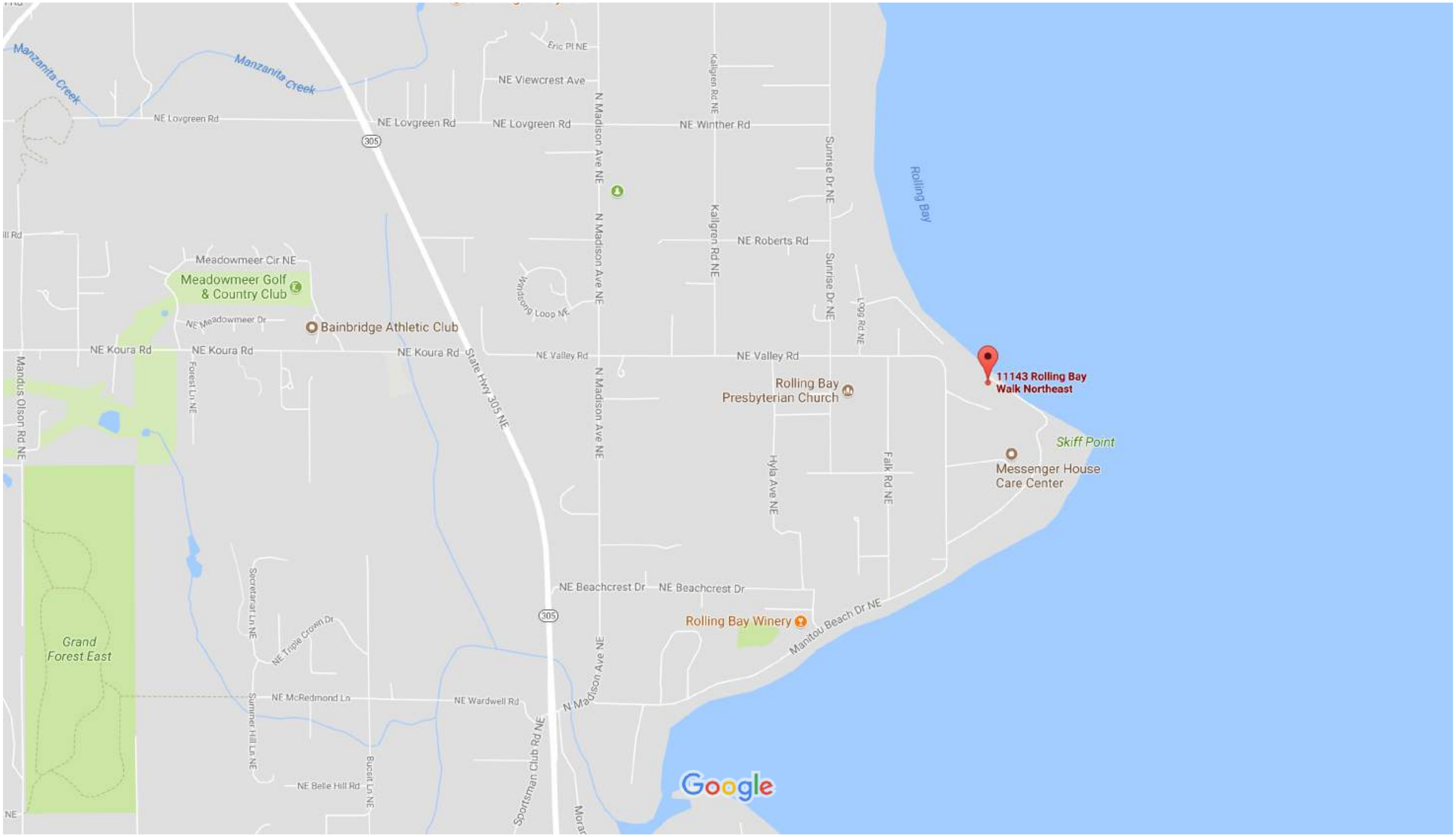
Taxpayer Name	DUFRESNE MARGARET A
Mailing Address	3912 NE STATE HIGHWAY 104 POULSBO, WA 98370 *(Change Mailing Address)*
Parcel No.	4156-001-004-1006
Account ID	2606333
Site Address	11143 ROLLING BAY WALK NE BAINBRIDGE ISLAND, WA 98110
Status	Active
Property Use	910- Undeveloped land

Parcel Map   Disclaimer   Privacy Policy   Comments/Email

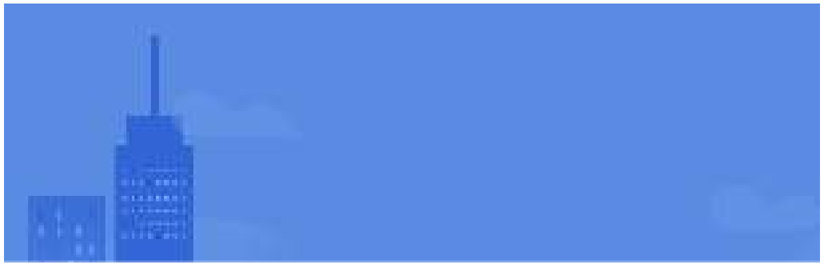
Google Maps

11143 Rolling Bay Walk NE

11143



Map data ©2017 Google United States 1000 ft



11143 Rolling Bay Walk NE

Bainbridge Island, WA 98110

Design Maps Summary Report

User–Specified Input

Report Title Dufrense Residence
Mon October 16, 2017 16:28:20 UTC

Building Code Reference Document 2012/2015 International Building Code
(which utilizes USGS hazard data available in 2008)

Site Coordinates 47.66408°N, 122.50307°W

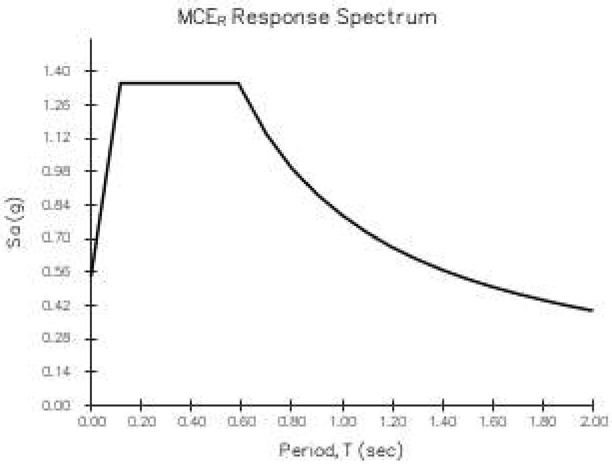
Site Soil Classification Site Class D – "Stiff Soil"

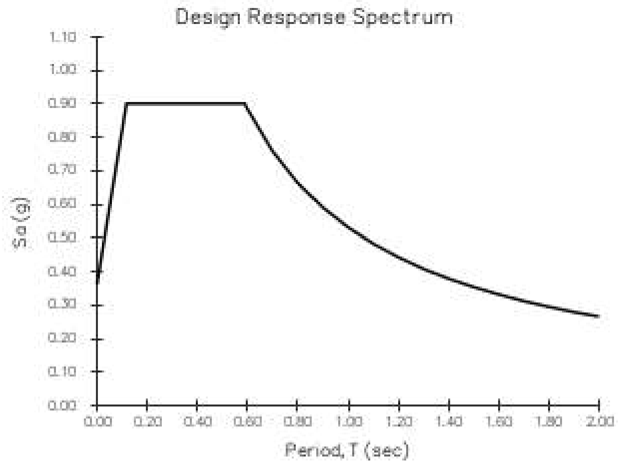
Risk Category I/II/III

USGS–Provided Output

S_S =	1.352 g	S_{MS} =	1.352 g	S_{DS} =	0.901 g
S_1 =	0.531 g	S_{M1} =	0.797 g	S_{D1} =	0.531 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.


MCE$_R$ Response Spectrum
1.40
1.26
1.12
0.98
0.84
0.70
0.56
0.42
0.28
0.14
0.00
Sa(g)
0.00 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 1.80 2.00
Period, T (sec)


Design Response Spectrum
1.10
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
Sa(g)
0.00 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 1.80 2.00
Period, T (sec)

Design Maps Detailed Report

2012/2015 International Building Code (47.66408°N, 122.50307°W)

Site Class D – “Stiff Soil”, Risk Category I/II/III

Section 1613.3.1 — Mapped acceleration parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2012/2015 International Building Code are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 1613.3.3.

From Figure 1613.3.1(1) [1]	S_S = 1.352 g
From Figure 1613.3.1(2) [2]	S_1 = 0.531 g

Section 1613.3.2 — Site class definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Section 1613.

2010 ASCE-7 Standard – Table 20.3-1
SITE CLASS DEFINITIONS

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		

F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 1613.3.3 — Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a

Site Class	Mapped Spectral Response Acceleration at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 1.352 g, F_a = 1.000

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_v

Site Class	Mapped Spectral Response Acceleration at 1–s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	$\boxed{1.5}$
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.531 g, F_v = 1.500

Equation (16-37): $S_{MS} = F_aS_S = 1.000 \times 1.352 = 1.352$ g

Equation (16-38): $S_{M1} = F_vS_1 = 1.500 \times 0.531 = 0.797$ g

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39): $S_{DS} = ⅔\, S_{MS} = ⅔ \times 1.352 = 0.901$ g

Equation (16-40): $S_{D1} = ⅔\, S_{M1} = ⅔ \times 0.797 = 0.531$ g

Section 1613.3.5 — Determination of seismic design category

TABLE 1613.3.5(1)
SEISMIC DESIGN CATEGORY BASED ON SHORT-PERIOD (0.2 second) RESPONSE ACCELERATION

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	**III**	**IV**
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.901 g, Seismic Design Category = D

TABLE 1613.3.5(2)
SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATION

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	**III**	**IV**
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.531 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 1613.3.5(1) or 1613.3.5(2)" = D

Note: See Section 1613.3.5.1 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 1613.3.1(1)*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(1).pdf

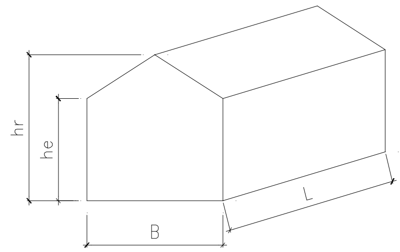
2. *Figure 1613.3.1(2)*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(2).pdf

Lucia Engineering, Inc.	PROJECT :	Dufrense Residence - Bainbridge Island, WA	PAGE :	
	CLIENT :	Margaret Dufrense	DESIGN BY :	JML
	JOB NO. :	DATE : 10/16/17	REVIEW BY :	

Wind Analysis for Low-rise Building, Based on ASCE 7-2010

INPUT DATA

Exposure category (B, C or D, ASCE 7-10 26.7.3)		D	
Importance factor (ASCE 7-10 Table 1.5-2)	I_w =	1.00	for all Category
Basic wind speed (ASCE 7-10 26.5.1 or 2015 IBC)	V =	110	mph
Topographic factor (ASCE 7-10 26.8 & Table 26.8-1)	K_{zt} =	1	Flat
Building height to eave	h_e =	28	ft
Building height to ridge	h_r =	29	ft
Building length	L =	60	ft
Building width	B =	20	ft
Effective area of components (or Solar Panel area)	A =	28	ft^2



DESIGN SUMMARY

Max horizontal force normal to building length, L, face	=	36.89 kips, SD level (LRFD level), Typ.
Max horizontal force normal to building length, B, face	=	13.67 kips
Max total horizontal torsional load	=	266.37 ft-kips
Max total upward force	=	26.83 kips

ANALYSIS

Velocity pressure

$q_h = 0.00256\, K_h\, K_{zt}\, K_d\, V^2$ = **30.23** psf

where: q_h = velocity pressure at mean roof height, h. (Eq. 28.3-1 page 298 & Eq. 30.3-1 page 316)

K_h = velocity pressure exposure coefficient evaluated at height, h, (Tab. 28.3-1, pg 299) = **1.15**

K_d = wind directionality factor. (Tab. 26.6-1, for building, page 250) = **0.85**

h = mean roof height = **28.50** ft

< 60 ft, [Satisfactory] (ASCE 7-10 26.2.1)

Design pressures for MWFRS

$p = q_h\, [(G\,C_{pf}) - (G\,C_{pi})]$

where: p = pressure in appropriate zone. (Eq. 28.4-1, page 298). p_{min} = **16** psf (ASCE 7-10 28.4.4)

$G\,C_{pf}$ = product of gust effect factor and external pressure coefficient, see table below. (Fig. 28.4-1, page 300 & 301)

$G\,C_{pi}$ = product of gust effect factor and internal pressure coefficient.(Tab. 26.11-1, Enclosed Building, page 258)

= **0.18** or **-0.18**

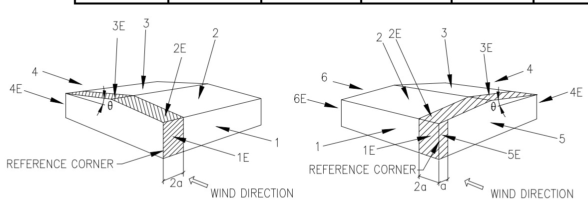
a = width of edge strips, Fig 28.4-1, note 9, page 301, MAX[MIN(0.1B, 0.1L, 0.4h), MIN(0.04B, 0.04L), 3] = **3.00 ft**

Net Pressures (psf), Basic Load Cases

Surface	Roof angle θ = 5.71			Roof angle θ = 0.00		
	$G\,C_{pf}$	Net Pressure with $(+GC_{pi})$	Net Pressure with $(-GC_{pi})$	$G\,C_{pf}$	Net Pressure with $(+GC_{pi})$	Net Pressure with $(-GC_{pi})$
1	0.41	6.84	17.72	-0.45	-19.04	-8.16
2	-0.69	-26.30	-15.42	-0.69	-26.30	-15.42
3	-0.38	-16.78	-5.90	-0.37	-16.62	-5.74
4	-0.30	-14.41	-3.53	-0.45	-19.04	-8.16
5				0.40	6.65	17.53
6				-0.29	-14.21	-3.32
1E	0.62	13.27	24.15	-0.48	-19.95	-9.07
2E	-1.07	-37.78	-26.90	-1.07	-37.78	-26.90
3E	-0.54	-21.69	-10.81	-0.53	-21.46	-10.58
4E	-0.44	-18.74	-7.86	-0.48	-19.95	-9.07
5E				0.61	13.00	23.88
6E				-0.43	-18.44	-7.56

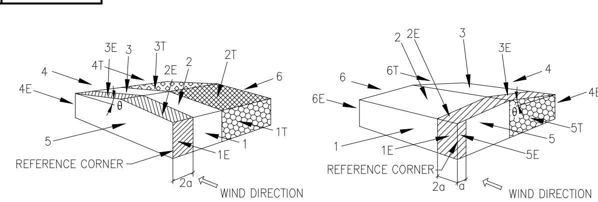
Net Pressures (psf), Torsional Load Cases

Surface	Roof angle θ = 5.71		
	$G\,C_{pf}$	Net Pressure with $(+GC_{pi})$	Net Pressure with $(-GC_{pi})$
1T	0.41	1.71	4.43
2T	-0.69	-6.57	-3.85
3T	-0.38	-4.20	-1.48
4T	-0.30	-3.60	-0.88
Surface	Roof angle θ = 0.00		
	$G\,C_{pf}$	Net Pressure with $(+GC_{pi})$	Net Pressure with $(-GC_{pi})$
5T	0.40	1.66	4.38
6T	-0.29	-3.55	-0.83



Load Case A (Transverse) Load Case B (Longitudinal)

Basic Load Cases



Load Case A (Transverse) Load Case B (Longitudinal)

Torsional Load Cases

Basic Load Case A (Transverse Direction)

Surface	Area (ft^2)	Pressure (k) with ($+GC_{pi}$)	Pressure (k) with ($-GC_{pi}$)
1	1512	10.34	26.79
2	543	-14.27	-8.37
3	543	-9.11	-3.20
4	1512	-21.78	-5.33
1E	168	2.23	4.06
2E	60	-2.28	-1.62
3E	60	-1.31	-0.65
4E	168	-3.15	-1.32
Σ	Horiz.	36.89	36.89
	Vert.	-26.83	-13.77
Min. wind	Horiz.	27.84	27.84
28.4.4	Vert.	-19.20	-19.20

Basic Load Case B (Longitudinal Direction)

Surface	Area (ft^2)	Pressure (k) with ($+GC_{pi}$)	Pressure (k) with ($-GC_{pi}$)
2	543	-14.27	-8.37
3	543	-9.02	-3.12
5	401	2.67	7.03
6	401	-5.70	-1.33
2E	60	-2.28	-1.62
3E	60	-1.29	-0.64
5E	169	2.20	4.03
6E	169	-3.11	-1.28
Σ	Horiz.	13.67	13.67
	Vert.	-24.09	-8.68
Min. wind	Horiz.	9.12	9.12
28.4.4	Vert.	-19.20	-19.20

Torsional Load Case A (Transverse Direction)

Surface	Area (ft^2)	Pressure (k) with ($+GC_{pi}$)	Pressure (k) with ($-GC_{pi}$)	Torsion (ft-k) ($+GC_{pi}$)	Torsion (ft-k) ($-GC_{pi}$)
1	672	4.59	11.91	62	161
2	241	-6.34	-3.72	-9	-5
3	241	-4.05	-1.42	5	2
4	672	-9.68	-2.37	131	32
1E	168	2.23	4.06	60	110
2E	60	-2.28	-1.62	-6	-4
3E	60	-1.31	-0.65	4	2
4E	168	-3.15	-1.32	85	36
1T	840	1.44	3.72	-22	-56
2T	301	-1.98	-1.16	3	2
3T	301	-1.26	-0.44	-2	-1
4T	840	-3.03	-0.74	-45	-11
Total Horiz. Torsional Load, M_T				266	266

Torsional Load Case B (Longitudinal Direction)

Surface	Area (ft^2)	Pressure (k) with ($+GC_{pi}$)	Pressure (k) with ($-GC_{pi}$)	Torsion (ft-k) ($+GC_{pi}$)	Torsion (ft-k) ($-GC_{pi}$)
2	543	-14.27	-8.37	-2	-1
3	543	-9.02	-3.12	1	0
5	116	0.77	2.04	3	7
6	116	-1.65	-0.39	6	1
2E	60	-2.28	-1.62	6	5
3E	60	-1.29	-0.64	-4	-2
5E	169	2.20	4.03	19	34
6E	169	-3.11	-1.28	26	11
5T	285	0.47	1.25	-2	-6
6T	285	-1.01	-0.24	-5	-1
Total Horiz. Torsional Load, M_T				48.2	48.2

Design pressures for components and cladding

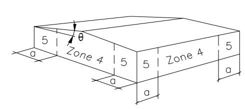
$p = q_h[(G C_p) - (G C_{pi})]$

where: p = pressure on component. (Eq. 30.4-1, pg 318)

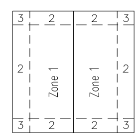
p_{min} = 16.00 psf (ASCE 7-10 30.2.2)

$G C_p$ = external pressure coefficient.

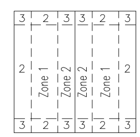
see table below. (ASCE 7-10 30.4.2)



Walls



Roof θ≤7°



Roof θ>7°

	Effective Area (ft^2)	Zone 1 GC_P	Zone 1 $-GC_P$	Zone 2 GC_P	Zone 2 $-GC_P$	Zone 3 GC_P	Zone 3 $-GC_P$	Zone 4 GC_P	Zone 4 $-GC_P$	Zone 5 GC_P	Zone 5 $-GC_P$
Comp.	28	0.26	-0.96	0.26	-1.49	0.26	-2.04	0.83	-0.92	0.83	-1.12

(Walls reduced 10 %, Fig. 6-11A note 5.)

Comp. & Cladding Pressure (psf)	Zone 1 Positive	Zone 1 Negative	Zone 2 Positive	Zone 2 Negative	Zone 3 Positive	Zone 3 Negative	Zone 4 Positive	Zone 4 Negative	Zone 5 Positive	Zone 5 Negative
	16.00	-34.32	16.00	-50.39	16.00	-67.10	30.50	-33.22	30.50	-39.23

Note: If the effective area is roof Solar Panel area, the only zone 1, 2 , or 3 apply.

LUCIA ENGINEERING, INC.
7307 12th Avenue NE
Seattle, WA 98115
(206) 790-8039

Lateral Calcualtions: 3 Story wood structure over 2 story concrete structure

Project Name: **Dufrense Residence**
Poject Location: 11143 Rolling Bay Walk NE
Bainbridge Island, WA 98110

Rev - 0 9/16/2017

Owner: **Margaret Dufrense**
Contact: **Margaret Dufrense**
Phone: (206) 954-5200

Architect: **Steale Krank Design**
Contact: **Jeff Knutson**
Phone: (360) 876-6242

Referenced Design Standards:
IBC 2015
SBC 2015 — **Wind:** Simplified Procedure ASCE 7-10
NDS 2015 — **Seismic:** Equivalent Lateral Force Procedure ASCE 7-10

Input Data:

Address:
11143 Rolling Bay Walk NE

Bainbridge Island, WA 98110	98110		
Building Length: "L" =	60.00	feet	
Building Width: "B" =	20.00	feet	
Height to Eave: "he" =	28.00	feet	
Height to Ridge: "hr" =	29.00	feet	
Topographic Factor: "K_{zt}" =	1.00	Used, Seattle Wind Map	
Basic Wind speed:	110.00	mph	V_{asd} = 85 mph (SCB 1609.3.1)
Exposure Category:	D		
Importance Factor: "I" =	1.00		
Site Classification:	D	Default Site Class (Soil properties are not know in sufficient detail to determine site class)	

Mapped Acceleration Parameters:

S_1 =	0.531	See the attached results from the USGS report
S_s =	1.352	See the attached results from the USGS report
S_{Ms} =	1.352	See the attached results from the USGS report
S_{M1} =	0.979	See the attached results from the USGS report
F_a =	1.00	See the attached results from the USGS report
F_v =	1.50	See the attached results from the USGS report
S_{DS} =	0.901	See the attached results from the USGS report
S_{D1} =	0.531	See the attached results from the USGS report
R_w =	6.5	Wood Framed Structure - ASCE 7-10, Table 12.2-1 (A-13)
R_c =	5.0	Concrete Structure - ASCE 7-10, Table 12.2-1 (A-1)
$C_{d,w}$ =	4.00	Wood Framed Structure - ASCE 7-10, Table 12.2-1 (A-13)
$C_{d,c}$ =	5.00	Concrete Structure - ASCE 7-10, Table 12.2-1 (A-1)
$\Omega_{o,w}$ =	3.00	Wood Framed Structure - ASCE 7-10, Table 12.2-1 (A-13)
$\Omega_{o,c}$ =	2.50	Concrete Structure - ASCE 7-10, Table 12.2-1 (A-1)
ρ =	1.00	ASCE 7-10, Section 12.3.4.2 Redundancy Factor for Seismic Design Categories D through F
C_t =	0.02	ASCE 7-10, Table 12.8-2 Values of Approximate Period Parameters C_t & x
C_{t1} =	0.016	ASCE 7-10, Table 12.8-2 Values of Approximate Period Parameters C_t & x b (Concrete)
x =	0.75	ASCE 7-10, Table 12.8-2 Values of Approximate Period Parameters C_t & x
x_1 =	0.90	ASCE 7-10, Table 12.8-2 Values of Approximate Period Parameters C_t & x (Concrete)

Roof	Roof Area:	
Roof Area:	1364.00	$feet^2$
Diaphragm Area:	1364.00	$feet^2$

3rd Floor	Floor Area:	
Floor Area	1200.00	$feet^2$
Diaphragm Area:	1200.00	$feet^2$

2nd Floor	Floor Area:	
Floor Area	1200.00	$feet^2$
Diaphragm Area:	1200.00	$feet^2$

1st Floor	Floor Area:	
Floor Area:	0.00	$feet^2$
Diaphragm Area:	0.00	$feet^2$

1/2 Level	Floor Area:	
Floor Area:	0.00	$feet^2$
Diaphragm Area:	0.00	$feet^2$

Garage-Street Level	Floor Area:	
Floor Area:	1080.00	$feet^2$
Diaphragm Area:	1080.00	$feet^2$

Roof Level:	13,367	lbs	Timber Framed
3rd Floor	12,000	lbs	Timber Framed
2nd Floor	12,000	lbs	Timber Framed
1st Floor	-	lbs	Timber Framed
1/2 Level	-	lbs	Elevated Concrete Slab
Garage-Street Level	-	lbs	Concrete
	37,367	lbs Total Floor Weight	

Roof Level:				
3rd Floor	Level Wall Height:	10.00	feet	Timber Framed
2nd Floor	Level Wall Height:	9.00	feet	Timber Framed
1st Floor	Level Wall Height:	0.00	feet	Timber Framed
1/2 Level	Level Wall Height:	0.00	feet	Concrete
Garage-Street Level	Level Wall Height:	9.08	feet	Concrete

Roof Level:			
3rd Floor	Level Wall Weight:	11,520	lbs (Assumes full parameter wall lengths)
2nd Floor	Level Wall Weight:	10,368	lbs (Assumes full parameter wall lengths)
1st Floor	Level Wall Weight:	-	lbs (Assumes full parameter wall lengths)
1/2 Level	Level Wall Weight:	-	lbs (Assumes full parameter wall lengths)
Garage-Street Level	Level Wall Weight:	176,079	lbs (Assumes full parameter wall lengths)
		197,967	lbs Total Wall Weight

235,335 lbs Total Building Weight, to Basement Slab

Building Weights:

Roof Dead Load:			
Roofing:		2.00	psf
30 lb Building Paper:		0.30	psf
1/2" Plywood Sheathing:		1.50	psf
Roof Framing: (Timber Framed)		2.50	psf
Batt Insulation:		0.50	psf
5/8 Gypsum Ceiling:		2.00	psf
HVAC Equipment & Ducting:		0.50	psf
Misc.:		0.50	psf
		9.8	psf

Floor Dead Load: (Timber Framed)			
Floor Covering: (Carpet/Linoleum)		1.50	psf
3/4" Plywood Sheeting:		1.50	psf
Floor Joists: (TJI's or 2x12's)		3.50	psf
Batt Insulation:		0.50	psf
5/8" Gypsum Ceiling:		2.00	psf
HVAC Equipment & Ducting:		0.50	psf
Misc.:		0.50	psf
		10.00	psf

Floor Dead Load: (Elevated Concrete Slab)			
Floor Covering: (Carpet/Linoleum)		0.00	psf
3/4" Plywood Sheeting:		0.00	psf
Concrete Slab:	(@ 150 pcf)	0.00	psf
Slab Depth =	0 inches		
Rigid Insulation:		0.00	psf
5/8" Gypsum Ceiling:		0.00	psf
HVAC Equipment & Ducting:		0.00	psf
Misc.:		0.00	psf
		0.00	psf

Wall Dead Load: (Timber Framed)			
1/2" Siding:		1.50	psf
1/2" Sheeting:		1.50	psf
2x6 Timber Framing:		1.00	psf
Batt Insulation:		0.50	psf
1/2" Gypsum Wallboard:		2.20	psf
Misc.:		0.50	psf
		7.2	psf

Wall Dead Load: (Concrete)			
Rigid Insulation:		0.50	psf
1/2" Gypsum Wallboard:		2.20	psf
Concrete Wall & 8" Thickness	(@ 150 pcf)	100.00	psf
Wall Thickness =	8 inches		
Brick Facing:		18.0	psf
Misc.:		0.50	psf
		121.2	psf

WIND & SEISMIC LOADING:

WIND LOADING:

I	=	1.00	Importance Factor	Roof Level:		Ave. Elev. =	19.00	q_z	=	30.23 psf *
V	=	110.00 MPH	Basic Wind Speed	4th Floor Level:	10.00 feet	Elev. =	9.00	q_z	=	30.23 psf
K_{ZT}	=	1.00	Per City of Seattle	3rd Floor Level:	9.00 feet	Elev. =	0.00	q_z	=	30.23 psf
h_e	=	28.00 FT	Height to Eave	2nd Floor Level:	0.00 feet	Elev. =	0.00	q_z	=	30.23 psf
h_r	=	29.00 FT	Height to Ridge	1st Floor level:	0.00 feet	Elev. =	0.00	q_z	=	30.23 psf
B	=	20.00 FT	Building Length	Basement Level:	9.08 feet	Elev. =	0.00	q_z	=	30.23 psf
L	=	60.00 FT	Building Width							

* See the attached Wind Loading Calculations

Wind Loading: Vx

		PSF		B		AREA		Wind Loading	
Roof	=	30.23	x	20.00	x	5.00	=	**3.02** KIPS	
3rd Floor	=	30.23	x	20.00	x	9.50	=	**5.74** KIPS	
2nd Floor	=	30.23	x	20.00	x	4.50	=	**2.72** KIPS	
1st Floor	=	30.23	x	20.00	x	-	=	**-** KIPS	Timber Framed
1/2 Level	=	30.23	x	20.00	x	-	=	**-** KIPS	Concrete
Garage-Street Level	=	30.23	x	20.00	x	9.04	=	**5.47** KIPS	

Wind Loading: Vy

		PSF		L		AREA		Wind Loading	
Roof	=	30.23	x	60.00	x	5.00	=	**9.07** KIPS	
3rd Floor	=	30.23	x	60.00	x	9.50	=	**17.23** KIPS	
2nd Floor	=	30.23	x	60.00	x	4.50	=	**8.16** KIPS	
1st Floor	=	30.23	x	60.00	x	-	=	**-** KIPS	Timber Framed
1/2 Level	=	30.23	x	60.00	x	-	=	**-** KIPS	Concrete
Garage-Street Level	=	30.23	x	60.00	x	9.04	=	**16.40** KIPS	

SEISMIC LOADING:

Mapped Acceleration Parameters:

S_1 =	0.531	See the attached results from the USGS report	R_c =	5.0	Concrete Structure - ASCE 7-10, Table 12.2-1 (A-1)
S_s =	1.352	See the attached results from the USGS report	$C_{d,w}$ =	4.00	Wood Framed Structure - ASCE 7-10, Table 12.2-1 (A-13)
S_{Ms} =	1.352	See the attached results from the USGS report	$C_{d,c}$ =	5.00	Concrete Structure - ASCE 7-10, Table 12.2-1 (A-1)
S_{M1} =	0.979	See the attached results from the USGS report	$\Omega_{o,w}$ =	3.00	Wood Framed Structure - ASCE 7-10, Table 12.2-1 (A-13)
F_a =	1.00	See the attached results from the USGS report	$\Omega_{o,c}$ =	2.50	Concrete Structure - ASCE 7-10, Table 12.2-1 (A-1)
F_v =	1.50	See the attached results from the USGS report	ρ =	1.00	ASCE 7-10, Section 12.3.4.2 Redundancy Factor for Seismic Design Categories D through F
S_{DS} =	0.863	See the attached results from the USGS report	C_t =	0.02	ASCE 7-10, Table 12.8-2 Values of Approximate Period Parameters C_t & x
S_{D1} =	0.531	See the attached results from the USGS report	C_{t1} =	0.016	ASCE 7-10, Table 12.8-2 Values of Approximate Period Parameters C_t & x b (Concrete)
R_w =	6.5	Wood Framed Structure - ASCE 7-10, Table 12.2-1 (A-13)	x =	0.75	ASCE 7-10, Table 12.8-2 Values of Approximate Period Parameters C_t & x
I =	1.00	Importance Factor	x_1 =	0.90	ASCE 7-10, Table 12.8-2 Values of Approximate Period Parameters C_t & x (Concrete)

Seismic Base Shear:

For Wood Framed Story Levels:

$V = C_sW$ ASCE 7-10, Equation (12.8-1)

$C_s = S_{ds} / (R/I) \le S_{D1} / T(R/I)$ or $S_{D1}T_L / T^2(R/I) \ge 0.01$ ASCE 7-10, Equations (12.8-2, 3 & 4)

Where: $T = T_a = C_th_n^x$ (Approximate Fundamental Period)

T_L = 6.00 ASCE 7-10, Section 11.4.5

Where:

C_t = 0.02 ASCE 7-10,Table 12.8-2

x = 0.75 ASCE 7-10,Table 12.8-2

T = 0.250 For h = 29.00 feet

Therefore k = 1.00 ASCE 7-10, 12.8.3

V = 0.133 W $\le$ 0.327 W or 7.846 W $\ge$ 1.00

For Concrete Story Levels:

$V = C_sW$ ASCE 7-10, Equation (12.8-1)

$C_s = S_{ds} / (R/I) \le S_{D1} / T(R/I)$ or $S_{D1}T_L / T^2(R/I) \ge 0.01$ ASCE 7-10, Equations (12.8-2, 3 & 4)

Where: $T = T_a = C_th_n^x$ (Approximate Fundamental Period)

T_L = 6.00 ASCE 7-10, Section 11.4.5

Where:

C_{t1} = 0.016 ASCE 7-10,Table 12.8-2 (For Concrete)

x_1 = 0.90 ASCE 7-10,Table 12.8-2 (For Concrete)

T = 0.331 For h = 29.00 feet

Therefore k = 1.05 ASCE 7-10, 12.8.3

V = 0.1726 W $\le$ 0.321 W or 5.804 W $\ge$ 1.00

Vertical Distribution of Shear forces

$F_x = C_{vx}V$ Where:

$C_{vx} = w_xh_x^k / \text{sum } w_ih_i^k$ ASCE 7-10, Equation (12.8-3)

V = 0.133 W = - Kips **For Timber Framed Story Levels**

0.1726 W = - Kips **For Concrete Story Level**

Level	Floor to Floor Height	Height	k*	hx^k	w_x (lbs)	$w_xh_x^k$	C_{vx}	V (Kips)	F_x	F_x/w	O.M. (k-ft)
Roof		28.08 feet	1.00	28.08	13,367	375,351	0.20	1.77	0.36	0.03	3.57
3rd Floor	10.00	18.08 feet	1.00	18.08	23,520	425,242	0.23	3.12	0.71	0.03	13.19
2nd Floor	9.00	9.08 feet	1.00	9.08	22,368	203,101	0.11	2.97	0.32	0.01	13.19
1st Floor	0.00	9.08 feet	1.00	9.08	-	-	-	-	0.00	-	13.19
1/2 Level	0.00	9.08 feet	1.05	10.14	-	-	-	-	0.00	-	25.83
Garage-Street Level	9.08	0.00 feet	1.05	4.90	176,079	862,218	0.46	23.38	10.80	0.06	30.05
Totals					235,335	1,865,912					

* Table is adjusted for variations in the types of construction. (Wood or Concrete)

Diaphragm Forces		
Sum F_i	Sum W_i	P_{px} (Kips)
0.36	13.37	0.36
0.71	13.39	1.25
0.32	13.39	0.54
0.00	13.40	-
0.00	13.41	-
10.80	13.42	142

Vx & Vy

		Seismic Load: (Kips)		
Roof	Level:	1.77	KIPS	
3rd Floor	Level:	3.12	KIPS	
2nd Floor	Level:	2.97	KIPS	
1st Floor	Level:	-	KIPS	Timber Framed
1/2 Level	Level:	-	KIPS	Concrete
Garage-Street Level	Level:	23.38	KIPS	

Comparison of Design Loads for Wind & Seismic

USE WORST CASE: WIND OR SEISMIC (Figures highlighted in **RED** indicate values used in the design.)

		Wind:	**Seismic:**	**Design Load:**	Deisng Load for Wood to Concrete Holdowns
Vx	Design Shear in the "X" Direction				
Roof	Level:	**3.02** KIPS	1.77 KIPS	**3.02** KIPS	
3rd Floor	Level:	**5.74** KIPS	3.12 KIPS	**8.77** KIPS	
2nd Floor	Level:	2.72 KIPS	**2.97** KIPS	**11.74** KIPS	Sum Seismic to 1st Floor Level
1st Floor	Level:	- KIPS	- KIPS	**11.74** KIPS	7.87 KIPS x 2.5 = **19.67** KIPS
1/2 Level	Level:	- KIPS	- KIPS	**11.74** KIPS	Overstrength Factor
Garage-Street Level	Level:	5.47 KIPS	**23.38** KIPS	**35.11** KIPS	
Vy	Design Shear in the "Y" Direction				
Roof	Level:	**9.07** KIPS	1.77 KIPS	**9.07** KIPS	
3rd Floor	Level:	**17.23** KIPS	3.12 KIPS	**26.30** KIPS	
2nd Floor	Level:	**8.16** KIPS	2.97 KIPS	**34.46** KIPS	Sum Seismic to 1st Floor Level
1st Floor	Level:	- KIPS	- KIPS	**34.46** KIPS	7.87 KIPS x 2.5 = **19.67** KIPS
1/2 Level	Level:	- KIPS	- KIPS	**34.46** KIPS	Overstrength Factor
Garage-Street Level	Level:	16.40 KIPS	**23.38** KIPS	**57.84** KIPS	

		Wind	**Seismic**
X-Direction Design Shear @ 2nd Level Elevated Composite Slab	=	**34.46** KIPS	**7.87** KIPS
Y-Direction Design Shear @ 2nd Level Elevated Composite Slab	=	**11.49** KIPS	**7.87** KIPS
X-Direction Design Shear @ 1st Level Elevated Composite Slab	=	**34.46** KIPS	**7.87** KIPS
Y-Direction Design Shear @ 1st Level Elevated Composite Slab	=	**11.74** KIPS	**7.87** KIPS

Base Shear = 35.11 KIPS

Seismic Load Effect: For individual member design

$E = E_h + E_v$

Where: $E_h = \rho Q_E$

$\rho = 1.0$

$Q_E = V_i$ Shear Load on member at given floor/roof level

$E_v = 0.2 S_{DS} D$

$S_{DS} = 0.863$

$D =$ Dead Load on member

Strength Design Load combinations:

5. $(1.2 + S_{DS})D + \rho Q_E + L + 0.2S$

Where:

$L = 0.5$ Section 12.4.2.3

$Q_E = V_i$ Shear Load on member at given floor/roof level

$S_{DS} = 0.863$

$\rho = 1.0$

7. $(0.9 - 0.2S_{DS})D = \rho Q_E + 1.6H$

Where:

$D =$ Dead Load on member

$Q_E = V_i$ Shear Load on member at given floor/roof level

$S_{DS} = 0.863$

$\rho = 1.0$

Allowable Stress Design Load Combinations:

5. $(1.0 + 0.14S_{DS})D + H + F + 0.7\rho Q_E$

Where:

$H = 0$

$S_{DS} = 0.863$

$D =$ Dead Load on member

$\rho = 1.0$

$F = F_i =$ Shear at Level i

$Q_E = V_i$ Shear Load on member at given floor/roof level

6. $(1.0 + 0.14S_{DS})D + H + F = 0.252\rho Q_E + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$

Where:

$H = 0$

$S_{DS} = 0.863$

$D =$ Dead Load on member

$\rho = 1.0$

$F = F_i =$ Shear at Level i

$Q_E = V_i$ Shear Load on member at given floor/roof level

$L = 0.5$

8. $(1.0 + 0.14S_{DS})D + 0.7\rho Q_E + H$

Where:

$H = 0$

$S_{DS} = 0.863$

$D =$ Dead Load on member

$\rho = 1.0$

$Q_E = V_i$ Shear Load on member at given floor/roof level

Diaphragm Shear:

	Design Load:		Width (Feet)	Loading (Kips)	Length (Feet)	Loading (Kips)	Design Loading (PLF)	Allowable Diaphragm Loading	
	Vy	Design Shear in the "Y" Direction							
Roof Lev Level:	**3.02**	KIPS	50.00	30.23	47.00	32.16	30	270	3/8 O.S.B. APA 24W 8d Nails @ 6" @ Edges
4th Floor Level:	**5.74**	KIPS	50.00	57.44	47.00	61.10	57	320	15/32 O.S.B. APA 24W 10d Nails @ 6" @ Edges
3rd Floor Level:	**2.72**	KIPS	50.00	27.21	47.00	28.94	27	320	15/32 O.S.B. APA 24W 10d Nails @ 6" @ Edges
2nd Floor Level:	**-**	KIPS	50.00	0.00	47.00	0.00	0		
1st Floor Level:	**-**	KIPS	50.00	0.00	62.00	0.00	0	> 3080	Concrete Slab On Grade
									Attachment method
	Vx	Design Shear in the "X" Direction							
Roof Level:	**9.07**	KIPS	50.00	90.69	47.00	96.48	91	270	15/12 O.S.B. APA 24W 8d Nails @ 6" @ Edges
4th Floor Level:	**17.23**	KIPS	50.00	172.31	47.00	183.31	172	270	15/12 O.S.B. APA 24W 8d Nails @ 6" @ Edges
3rd Floor Level:	**8.16**	KIPS	50.00	81.62	47.00	86.83	82	270	15/12 O.S.B. APA 24W 8d Nails @ 6" @ Edges
2nd Floor Level:	**-**	KIPS	50.00	0.00	47.00	0.00	0		
1st Floor Level:	**-**	KIPS	50.00	0.00	62.00	0.00	0	> 3080	Concrete Slab On Grade
									Attachment method

SHEAR WALL DESIGNATIONS, LENGTHS & SHEAR LOADING

3rd Floor Walls

SHEAR WALL DESIGNATION		LENGTH	Height	Aspect Ratio	Use of Table 2306.4.1	Tributary Area Supported by Wall
SWX	201	3.00 FT	10.00 FT	3.3	</= 3.5 Check - O.K.	171 SQFT
SWX	202	3.62 FT	10.00 FT	2.8	</= 3.5 Check - O.K.	171 SQFT
SWX	203	3.00 FT	10.00 FT	3.3	</= 3.5 Check - O.K.	85 SQFT
SWX	204	3.00 FT	10.00 FT	3.3	</= 3.5 Check - O.K.	85 SQFT
SWX	205	3.00 FT	10.00 FT	3.3	</= 3.5 Check - O.K.	170 SQFT
SWX	206	30.00 FT	8.00 FT	0.3	</= 3.5 Check - O.K.	682 SQFT
		45.62			Total Roof Area:	1,364 SQFT Calculated
						1,364.00 SQFT Actual

TOTAL SHEAR WALL LENGTH IN "X" DIRECTION: **45.62** FT
TOTAL SHEAR WALL TRIBUTARY AREA IN "X" DIRECTION: **1,364** SQFT

SHEAR WALL DESIGNATION		LENGTH	Height	Aspect Ratio	Use of Table 2306.4.1	Tributary Area Supported by Wall
SWY	201	12.00 FT	10.00 FT	0.8	</= 2.5 Check - O.K.	511 SQFT
SWY	202	15.00 FT	10.00 FT	0.7	</= 2.5 Check - O.K.	511 SQFT
SWY	203	15.00 FT	10.00 FT	0.7	</= 2.5 Check - O.K.	171 SQFT
SWY	204	8.25 FT	10.00 FT	1.2	</= 2.5 Check - O.K.	171 SQFT
		50.25			Total Roof Area:	1,364 SQFT Calculated
						1,364.00 SQFT Actual

Shear Wall Loading-Nailing Schedule			
Capacity		Mark	x 1.4 for Wind
240	PLF	P1-8-6	336
350	PLF	P1-8-4	490
450	PLF	P1-8-3	630
585	PLF	P1-8-2	819
700	PLF	P2-8-4	980
900	PLF	P2-8-3	1260
1170	PLF	P2-8-2	1638
310	PLF	P1-10-6	434
460	PLF	P1-10-4	644
600	PLF	P1-10-3	840
770	PLF	P1-10-2	1078

See Plans for Shear Wall Schedule Information
Per IBC Section 2306.4.1

TOTAL SHEAR WALL LENGTH IN "Y" DIRECTION: **50.25** FT
TOTAL SHEAR WALL TRIBUTARY AREA IN "Y" DIRECTION: **1,364** SQFT

FORCE IN "X" DIRECTION TO TOP OF WALL

F_{max} = **3.02** kips
FT_{max} = **0.002** kips (F_{max} / Total Tributary Area)

							Wall Loading	Table 2306.4.1 x (2w/h)	Wall Designation
SWX	201	V_{max}	=	F_{max} x FT_{max}	=	**0.38** kips / Wall Length =	126		**P1-8-6, P1-10-6**
SWX	202	V_{max}	=	F_{max} x FT_{max}	=	**0.38** kips / Wall Length =	105		**P1-8-6, P1-10-6**
SWX	203	V_{max}	=	F_{max} x FT_{max}	=	**0.19** kips / Wall Length =	63		**P1-8-6, P1-10-6**
SWX	204	V_{max}	=	F_{max} x FT_{max}	=	**0.19** kips / Wall Length =	63		**P1-8-6, P1-10-6**
SWX	205	V_{max}	=	F_{max} x FT_{max}	=	**0.38** kips / Wall Length =	126		**P1-8-6, P1-10-6**
SWX	206	V_{max}	=	F_{max} x FT_{max}	=	**1.51** kips / Wall Length =	50		**P1-8-6, P1-10-6**
						3.02 kips			

FORCE IN "Y" DIRECTION TO TOP OF WALL

F_{max} = 9.07 kips

FT_{max} = 0.007 kips (F_{max} / Total Tributary Area)

							Wall Loading	Table 2306.4.1 x (2w/h)	Wall Designation
SWY	201	V_{max}	=	F_{max} x FT_{max}	=	3.40 kips / Wall Length =	283		P1-8-6, P1-10-6
SWY	202	V_{max}	=	F_{max} x FT_{max}	=	3.40 kips / Wall Length =	227		P1-8-6, P1-10-6
SWY	203	V_{max}	=	F_{max} x FT_{max}	=	1.14 kips / Wall Length =	76		P1-8-6, P1-10-6
SWY	204	V_{max}	=	F_{max} x FT_{max}	=	1.14 kips / Wall Length =	138		P1-8-6, P1-10-6

9.07 **kips**

Roof Weight Acting on Shear Wall

SHEAR WALL DESIGNATION		LENGTH	Tributary Area Supported by Wall	Dead Load x 60%	100% Dead Load Per Foot Of Wall	100% Live Load Per Foot Of Wall
SWX	201	3.00 FT	171 FT	1,005	558.6	1,425
SWX	202	3.62 FT	171 FT	1,005	462.9	1,181
SWX	203	3.00 FT	85 FT	500	277.7	708
SWX	204	3.00 FT	85 FT	500	277.7	708
SWX	205	3.00 FT	170 FT	1,000	555.3	1,417
SWX	206	30.00 FT	682 FT	4,010	222.8	568

SHEAR WALL DESIGNATION		LENGTH	Floor Length Supported by Wall	Dead Load x 60%	100% Dead Load	100% Live Load
SWY	201	12.00 FT	511 FT	3,005	417.3	1,065
SWY	202	15.00 FT	511 FT	3,005	333.9	852
SWY	203	15.00 FT	171 FT	1,005	111.7	285
SWY	204	8.25 FT	171 FT	1,005	203.1	518

SHEAR WALL DESIGNATIONS, LENGTHS & SHEAR LOADING

2nd Floor Walls

SHEAR WALL DESIGNATION		LENGTH	Height	Aspect Ratio	Use of Table 2306.4.1	Tributary Area Supported by Wall
SWX	101	FT	FT			SQFT
SWX	102	2.58 FT	9.00 FT	3.5	</= 2.5 Check - O.K.	150 SQFT
SWX	103	3.00 FT	9.00 FT	3.0	</= 2.5 Check - O.K.	150 SQFT
SWX	104	3.00 FT	9.00 FT	3.0	</= 2.5 Check - O.K.	150 SQFT
SWX	105	3.00 FT	9.00 FT	3.0	</= 2.5 Check - O.K.	150 SQFT
SWX	106	30.00 FT	9.00 FT	0.3	</= 2.5 Check - O.K.	600 SQFT
		41.58			Total Floor Area:	1200 SQFT Calculated
						1,200.00 SQFT Actual

TOTAL SHEAR WALL LENGTH IN "X" DIRECTION: 41.58 FT

TOTAL SHEAR WALL TRIBUTARY AREA IN "X" DIRECTION: 1,200 SQFT

SHEAR WALL DESIGNATION		LENGTH	Height	Aspect Ratio	Use of Table 2306.4.1	Tributary Area Supported by Wall
SWY	101	12.00 FT	9.00 FT	0.8	</= 2.5 Check - O.K.	450 SQFT
SWY	102	15.00 FT	9.00 FT	0.6	</= 2.5 Check - O.K.	450 SQFT
SWY	103	15.00 FT	9.00 FT	0.6	</= 2.5 Check - O.K.	150 SQFT
SWY	104	8.25 FT	9.00 FT	1.1	</= 2.5 Check - O.K.	150 SQFT
		50.25			Total Floor Area:	1200 SQFT Calculated
						1,200.00 SQFT Actual

See Plans for Shear Wall Schedule Information
Per IBC Section 2306.4.1

Shear Wall Loading-Nailing Schedule

Capacity		Mark	x 1.3 for Wind
240	PLF	P1-8-6	336
350	PLF	P1-8-4	490
450	PLF	P1-8-3	630
585	PLF	P1-8-2	819
700	PLF	P2-8-4	980
900	PLF	P2-8-3	1260
1170	PLF	P2-8-2	1638
310	PLF	P1-10-6	434
460	PLF	P1-10-4	644
600	PLF	P1-10-3	840
770	PLF	P1-10-2	1078

TOTAL SHEAR WALL LENGTH IN "Y" DIRECTION: 50.25 FT

TOTAL SHEAR WALL TRIBUTARY AREA IN "Y" DIRECTION: 1,200 SQFT

FORCE IN "X" DIRECTION TO TOP OF WALL

F_{max} = 8.77 kips

FT_{max} = 0.007 kips (F_{max} / Total Tributary Area)

						Wall Loading	Table 2306.4.1 x (2w/h)	Wall Designation
SWX	101	V_{max}	=	$F_{max} \times FT_{max}$	=	0.00 kips / Wall Length =		
SWX	102	V_{max}	=	$F_{max} \times FT_{max}$	=	1.10 kips / Wall Length =	425	P1-8-4, P1-10-6
SWX	103	V_{max}	=	$F_{max} \times FT_{max}$	=	1.10 kips / Wall Length =	365	P1-8-4, P1-10-6
SWX	104	V_{max}	=	$F_{max} \times FT_{max}$	=	1.10 kips / Wall Length =	365	P1-8-4, P1-10-6
SWX	105	V_{max}	=	$F_{max} \times FT_{max}$	=	1.10 kips / Wall Length =	365	P1-8-4, P1-10-6
SWX	106	V_{max}	=	$F_{max} \times FT_{max}$	=	4.38 kips / Wall Length =	146	P1-8-6, P1-10-6
						8.77 kips		

FORCE IN "Y" DIRECTION TO TOP OF WALL

F_{max} = 26.30 kips

FT_{max} = 0.022 kips (F_{max} / Total Tributary Area)

									Wall Loading	Table 2306.4.1 x (2w/h)	Wall Designation
SWY	101	V_{max}	=	F_{max} x FT_{max}	=	9.86 kips	/ Wall Length	=	822		P2-8-4, P1-10-3
SWY	102	V_{max}	=	F_{max} x FT_{max}	=	9.86 kips	/ Wall Length	=	658		P2-8-4, P1-10-3
SWY	103	V_{max}	=	F_{max} x FT_{max}	=	3.29 kips	/ Wall Length	=	219		P1-8-6, P1-10-6
SWY	104	V_{max}	=	F_{max} x FT_{max}	=	3.29 kips	/ Wall Length	=	398		P1-8-4, P1-10-6

26.30 **kips**

Floor Weight Acting on Shear Wall

SHEAR WALL DESIGNATION		LENGTH	Tributary Area Supported by Wall	Dead Load x 60%	100% Dead Load Per Foot Of Wall	100% Live Load Per Foot Of Wall
SWX	101	0.00 FT	0 FT	-	#DIV/0!	#DIV/0!
SWX	102	2.58 FT	150 FT	900	581.4	1,453
SWX	103	3.00 FT	150 FT	900	500.0	1,250
SWX	104	3.00 FT	150 FT	900	500.0	1,250
SWX	105	3.00 FT	150 FT	900	500.0	1,250
SWX	106	30.00 FT	600 FT	3,600	200.0	500

SHEAR WALL DESIGNATION		LENGTH	Floor Length Supported by Wall	Dead Load x 60%	100% Dead Load	100% Live Load
SWY	101	12.00 FT	450 FT	2,700	375.0	938
SWY	102	15.00 FT	450 FT	2,700	300.0	750
SWY	103	15.00 FT	150 FT	900	100.0	250
SWY	104	8.25 FT	150 FT	900	181.8	455

SHEAR WALL DESIGNATIONS, LENGTHS & SHEAR LOADING

1st Floor Walls

SHEAR WALL DESIGNATION		LENGTH	Height	Aspect Ratio	Use of Table 2306.4.1	Tributary Area Supported by Wall
SWX	001	**2.37** FT	4.08 FT	1.7	</= 2.5 Check - O.K.	125 SQFT
SWX	002	**3.75** FT	4.08 FT	1.1	</= 2.5 Check - O.K.	150 SQFT
SWX	003	**3.37** FT	4.08 FT	1.2	</= 2.5 Check - O.K.	125 SQFT
SWX	004	**2.87** FT	4.08 FT	1.4	</= 2.5 Check - O.K.	100 SQFT
SWX	005	**2.87** FT	4.08 FT	1.4	</= 2.5 Check - O.K.	100 SQFT
SWX	006	**60.00** FT	9.00 FT	0.2	</= 2.5 Check - O.K.	600 SQFT
		75.23			Total Floor Area:	1200 SQFT Calculated
						1,200.00 SQFT Actual

TOTAL SHEAR WALL LENGTH IN "X" DIRECTION: **75.23** FT
TOTAL SHEAR WALL TRIBUTARY AREA IN "X" DIRECTION: **1,200** SQFT

SHEAR WALL DESIGNATION		LENGTH	Height	Aspect Ratio	Use of Table 2306.4.1	Tributary Area Supported by Wall
SWY	001	**6.00** FT	4.08 FT	0.7	</= 2.5 Check - O.K.	100 SQFT
SWY	002	**6.00** FT	4.08 FT	0.7	</= 2.5 Check - O.K.	50 SQFT
SWY	003	**5.81** FT	4.08 FT	0.7	</= 2.5 Check - O.K.	50 SQFT
SWY	004	**9.08** FT	4.08 FT	0.4	</= 2.5 Check - O.K.	300 SQFT
SWY	005	**15.00** FT	4.08 FT	0.3	</= 2.5 Check - O.K.	300 SQFT
SWY	006	**15.00** FT	4.08 FT	0.3	</= 2.5 Check - O.K.	200 SQFT
SWY	007	**5.81** FT	4.08 FT	0.7	</= 2.5 Check - O.K.	50 SQFT
SWY	008	**6.00** FT	4.08 FT	0.7	</= 2.5 Check - O.K.	50 SQFT
SWY	009	**6.00** FT	4.08 FT	0.7	</= 2.5 Check - O.K.	100 SQFT
		74.70			Total Floor Area:	1200 SQFT Calculated
						1,200.00 SQFT Actual

Shear Wall Loading-Nailing Schedule

Capacity		Mark	x 1.3 for Wind
240	PLF	P1-8-6	336
350	PLF	P1-8-4	490
450	PLF	P1-8-3	630
585	PLF	P1-8-2	819
700	PLF	P2-8-4	980
900	PLF	P2-8-3	1260
1170	PLF	P2-8-2	1638
310	PLF	P1-10-6	434
460	PLF	P1-10-4	644
600	PLF	P1-10-3	840
770	PLF	P1-10-2	1078

See Plans for Shear Wall Schedule Information
Per IBC Section 2306.4.1

TOTAL SHEAR WALL LENGTH IN "Y" DIRECTION: **74.70** FT
TOTAL SHEAR WALL TRIBUTARY AREA IN "Y" DIRECTION: **1,200** SQFT

FORCE IN "X" DIRECTION TO TOP OF WALL

F_{max} = **11.74** kips
FT_{max} = **0.010** kips (F_{max} / Total Tributary Area)

						Wall Loading	Table 2306.4.1 x (2w/h)	Wall Designation
SWX 001	V_{max}	=	F_{max} x FT_{max}	=	**1.22** kips / Wall Length =	516		**P1-8-3, P1-10-4**
SWX 002	V_{max}	=	F_{max} x FT_{max}	=	**1.47** kips / Wall Length =	391		**P1-8-4, P1-10-6**
SWX 003	V_{max}	=	F_{max} x FT_{max}	=	**1.22** kips / Wall Length =	363		**P1-8-4, P1-10-6**
SWX 004	V_{max}	=	F_{max} x FT_{max}	=	**0.98** kips / Wall Length =	341		**P1-8-4, P1-10-6**
SWX 005	V_{max}	=	F_{max} x FT_{max}	=	**0.98** kips / Wall Length =	341		**P1-8-4, P1-10-6**
SWX 006	V_{max}	=	F_{max} x FT_{max}	=	**5.87** kips / Wall Length =	98		**Concrete**
					11.74 kips			

FORCE IN "Y" DIRECTION TO TOP OF WALL

F_{max} = **34.46** kips

FT_{max} = **0.029** kips (**F_{max}** / Total Tributary Area)

							Wall Loading	Table 2306.4.1 x (2w/h)	Wall Designation
SWY 001	V_{max}	=	F_{max} x FT_{max}	=	**2.87** kips	/ Wall Length =	479		**P1-8-4, P1-10-6**
SWY 002	V_{max}	=	F_{max} x FT_{max}	=	**1.44** kips	/ Wall Length =	239		**P1-8-6, P1-10-6**
SWY 003	V_{max}	=	F_{max} x FT_{max}	=	**1.44** kips	/ Wall Length =	247		**P1-8-6, P1-10-6**
SWY 004	V_{max}	=	F_{max} x FT_{max}	=	**8.62** kips	/ Wall Length =	949		**P2-8-4, P1-10-2**
SWY 005	V_{max}	=	F_{max} x FT_{max}	=	**8.62** kips	/ Wall Length =	574		**P1-8-3, P1-10-4**
SWY 006	V_{max}	=	F_{max} x FT_{max}	=	**5.74** kips	/ Wall Length =	383		**P1-8-4, P1-10-6**
SWY 007	V_{max}	=	F_{max} x FT_{max}	=	**1.44** kips	/ Wall Length =	247		**P1-8-6, P1-10-6**
SWY 008	V_{max}	=	F_{max} x FT_{max}	=	**1.44** kips	/ Wall Length =	239		**P1-8-6, P1-10-6**
SWY 009	V_{max}	=	F_{max} x FT_{max}	=	**2.87** kips	/ Wall Length =	479		**P1-8-4, P1-10-6**

34.46 **kips**

Floor Weight Acting on Shear Wall

SHEAR WALL DESIGNATION		LENGTH	Tributary Area Supported by Wall	Dead Load x 60%	100% Dead Load Per Foot Of Wall	100% Live Load Per Foot Of Wall
SWX	001	**2.37** FT	**125** FT	750	527.4	1,319
SWX	002	**3.75** FT	**150** FT	900	400.0	1,000
SWX	003	**3.37** FT	**125** FT	750	370.9	927
SWX	004	**2.87** FT	**100** FT	600	348.4	871
SWX	005	**2.87** FT	**100** FT	600	348.4	871
SWX	006	**60.00** FT	**600** FT	3,600	100.0	250

SHEAR WALL DESIGNATION		LENGTH	Floor Length Supported by Wall	Dead Load x 60%	100% Dead Load	100% Live Load
SWY	001	**6.00** FT	**100** FT	600	166.7	417
SWY	002	**6.00** FT	**50** FT	300	83.3	208
SWY	003	**5.81** FT	**50** FT	300	86.1	215
SWY	004	**9.08** FT	**300** FT	1,800	330.4	826
SWY	005	**15.00** FT	**300** FT	1,800	200.0	500
SWY	006	**15.00** FT	**200** FT	1,200	133.3	333
SWY	007	**5.81** FT	**50** FT	300	86.1	215
SWY	008	**6.00** FT	**50** FT	300	83.3	208
SWY	009	**6.00** FT	**100** FT	600	166.7	417

OVERTURNING CALCULATIONS: X-DIRECTION

SW X 001

		Wall Loading		Wall Length		Wall Height		Overturning Moment
		psf		ft		ft		
M_o	=	516	x	2.37	x	4.08	=	4,988 ft-lbs

			60% Floor Weight		Floor Trib. Area		1/2 Wall Length		Resisting Moment
			psf		ft		ft		
M_{roof} =	M_{wall}	=	6.00	x	125	x	1.19	=	888.75 ft-lbs
									889 ft-lbs

M_U = 4,988 - 889 = 4,099 ft-lbs

Tiedown Load:

FT-LBS		FT						
4,099	/	2.08	=	**1,971** LBS	+	- LBS	=	**1,971** LBS
				Assume 2 - 2x6 end studs.		From Wall Above		
				holddown is at center of end studs.				

Use Simpson type: **HD9B** **Strap** **or equal**

SWL = **6,580** lbs

(Use 3-1/2" Stud)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 1,971 lbs

P_{allow} = 405 psi

1,971 / 405 = 4.87 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

SW X 002

		Wall Loading		Wall Length		Wall Height		Overturning Moment
M_o	=	391 psf	x	3.75 ft	x	4.08 ft	=	5,986 ft-lbs

				60% Floor Weight		Floor Trib. Area		1/2 Wall Length		Resisting Moment
M_{roof}	=	M_{wall}	=	6.00 psf	x	150 ft	x	1.88 ft	=	1687.5 ft-lbs
										1,688 ft-lbs

M_U = 5,986 - 1,688 = 4,298 ft-lbs

Tiedown Load:

4,298 FT-LBS / 3.46 FT = **1,242** LBS + **4,391** LBS = **5,634** LBS

Assume 2 - 2x6 end studs. From Wall Above

holddown is at center of end studs.

Use Simpson type: HD9B Strap or equal

SWL = **6,580** lbs

(Use 3-1/2" Stud)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 5,634 lbs

P_{allow} = 405 psi

5,634 / 405 = 13.91 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: X-DIRECTION

SW X 003

		Wall Loading		Wall Length		Wall Height		Overturning Moment
		psf		ft		ft		
M_o	=	363	x	3.37	x	4.08	=	4,988 ft-lbs

			60% Floor Weight		Floor Trib. Area		1/2 Wall Length		Resisting Moment
			psf		ft		ft		
M_{roof} =	M_{wall}	=	6.00	x	125	x	1.69	=	1263.75 ft-lbs
									1,264 ft-lbs

M_U = 4,988 - 1,264 = 3,724 ft-lbs

Tiedown Load:

FT-LBS		FT						
3,724	/	3.08	=	**1,209** LBS	+	**3,560** LBS	=	**4,769** LBS

Assume 2 - 2x6 end studs.
holddown is at center of end studs.

From Wall Above

Use Simpson type: **HD9B** **Strap** **or equal**

SWL = **6,580** lbs

(Use 3-1/2" Stud)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 4,769 lbs

P_{allow} = 405 psi

4,769 / 405 = 11.77 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: X-DIRECTION

SW X 004

		Wall Loading		Wall Length		Wall Height		Overturning Moment
		psf		ft		ft		
M_o	=	341	x	2.87	x	4.08	=	3,990 ft-lbs

				60% Floor Weight		Floor Trib. Area		1/2 Wall Length		Resisting Moment
				psf		ft		ft		
M_{roof}	=	M_{wall}	=	6.00	x	100	x	1.44	=	861 ft-lbs
										861 ft-lbs

M_U = 3,990 - 861 = 3,129 ft-lbs

Tiedown Load:

FT-LBS		FT						
3,129	/	2.58	=	**1,213** LBS	+	**3,560** LBS	=	**4,773** LBS
			Assume 2 - 2x6 end studs.			From Wall Above		
			holddown is at center of end studs.					

Use Simpson type: HD9B Strap or equal

SWL = **6,580** lbs

(Use 3-1/2" Stud)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate) (NDS 2005 Edition, Table 4A)

P_{max} = 4,773 lbs

P_{allow} = 405 psi

4,773 / 405 = 11.78 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: X-DIRECTION

SW X 005

		Wall Loading		Wall Length		Wall Height		Overturning Moment
		psf		ft		ft		
M_o	=	341	x	2.87	x	4.08	=	3,990 ft-lbs

				60% Floor Weight		Floor Trib. Area		1/2 Wall Length		Resisting Moment
				psf		ft		ft		
M_{roof}	=	M_{wall}	=	6.00	x	100	x	1.44	=	861 ft-lbs
										861 ft-lbs

M_U = 3,990 - 861 = 3,129 ft-lbs

Tiedown Load:

FT-LBS		FT						
3,129	/	2.58	=	**1,213** LBS	+	- LBS	=	**1,213** LBS

Assume 2 - 2x6 end studs.
holddown is at center of end studs.

From Wall Above

Use Simpson type: HD9B Strap or equal

SWL = **6,580** lbs

(Use 3-1/2" Stud)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 1,213 lbs

P_{allow} = 405 psi

1,213 / 405 = 2.99 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: X-DIRECTION

SW X 006

CONCRETE

OVERTURNING CALCULATIONS: Y-DIRECTION

SW Y 001

	Wall Loading		Wall Length		Wall Height		Overturning Moment
	psf		ft		ft		
M_o =	479	x	6.00	x	4.08	=	11,717 ft-lbs

		60% Floor Weight		Floor Trib. Area		1/2 Wall Length		Resisting Moment
		psf		ft		ft		
M_{roof} =	M_{wall} =	6.00	x	100	x	3.00	=	1800 ft-lbs
								1,800 ft-lbs

M_U = 11,717 - 1,800 = 9,917 ft-lbs

Tiedown Load:

FT-LBS		FT						
9,917	/	5.71	=	**1,737** LBS	+	**7,559** LBS	=	**9,295** LBS

Assume 2 - 2x6 end studs. From Wall Above

holddown is at center of end studs.

Use Simpson type: **HD12** **Strap** **or equal**

SWL = **10,765** lbs

(Use 4-1/2" Stud)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 9,295 lbs

P_{allow} = 405 psi

9,295 / 450 = 20.66 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: Y-DIRECTION

SW Y 002

	Wall Loading (psf)		Wall Length (ft)		Wall Height (ft)		Overturning Moment
M_o =	239	x	6.00	x	4.08	=	5,859 ft-lbs

		60% Floor Weight (psf)		Floor Trib. Area (ft)		1/2 Wall Length (ft)		Resisting Moment
M_{roof} =	M_{wall} =	6.00	x	50	x	3.00	=	900 ft-lbs
								900 ft-lbs

M_U = 5,859 - 900 = 4,959 ft-lbs

Tiedown Load:

FT-LBS		FT						
4,959	/	5.71	=	**868** LBS	+	5,435 LBS	=	**6,304** LBS

Assume 2 - 2x6 end studs.
holddown is at center of end studs.

From Wall Above

Use Simpson type: **HD12** **Strap** **or equal**

SWL = **10,765** lbs

(Use 4-1/2" Stud)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 6,304 lbs

P_{allow} = 405 psi

6,304 / 450 = 14.01 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: Y-DIRECTION

SW Y 003

		Wall Loading		Wall Length		Wall Height		Overturning Moment
M_o	=	247 psf	x	5.81 ft	x	4.08 ft	=	5,859 ft-lbs

			60% Floor Weight		Floor Trib. Area		1/2 Wall Length		Resisting Moment
M_{roof} =	M_{wall}	=	6.00 psf	x	50 ft	x	2.91 ft	=	871.5 ft-lbs
									872 ft-lbs

M_U = 5,859 - 872 = 4,987 ft-lbs

Tiedown Load:

FT-LBS		FT						
4,987	/	5.52	=	**903** LBS	+	**1,813** LBS	=	**2,716** LBS

Assume 2 - 2x6 end studs.

holddown is at center of end studs.

From Wall Above

Use Simpson type:	**HD12**	**Strap**	**or equal**

SWL = **10,765** lbs

(Use 4-1/2" Stud)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 2,716 lbs

P_{allow} = 405 psi

2,716 / 450 = 6.04 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: Y-DIRECTION

SW Y 004

		Wall Loading		Wall Length		Wall Height		Overturning Moment
		psf		ft		ft		
M_o	=	949	x	9.08	x	4.08	=	35,151 ft-lbs

				60% Floor Weight		Floor Trib. Area		1/2 Wall Length		Resisting Moment
				psf		ft		ft		
M_{roof}	=	M_{wall}	=	6.00	x	300	x	4.54	=	8172 ft-lbs
										8,172 ft-lbs

M_U = 35,151 - 8,172 = 26,979 ft-lbs

Tiedown Load:

FT-LBS		FT						
26,979	/	8.79	=	**3,069** LBS	+	**3,704** LBS	=	**6,774** LBS

Assume 2 - 2x6 end studs.
holddown is at center of end studs.

From Wall Above

Use Simpson type: **HD12** **Strap** **or equal**

SWL = **10,765** lbs

(Use 4-1/2" Stud)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 6,774 lbs

P_{allow} = 405 psi

6,774 / 450 = 15.05 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: Y-DIRECTION

SW Y 005

		Wall Loading		Wall Length		Wall Height		Overturning Moment
		psf		ft		ft		
M_o	=	574	x	15.00	x	4.08	=	35,151 ft-lbs

				60% Floor Weight		Floor Trib. Area		1/2 Wall Length		Resisting Moment
				psf		ft		ft		
M_{roof}	=	M_{wall}	=	6.00	x	300	x	7.50	=	13500 ft-lbs
										13,500 ft-lbs

M_U	=	35,151	-	13,500	=	21,651 ft-lbs

Tiedown Load:

FT-LBS		FT						
21,651	/	14.71	=	**1,472** LBS	+	**454** LBS	=	**1,926** LBS
				Assume 2 - 2x6 end studs.		From Wall Above		
				holddown is at center of end studs.				

Use Simpson type: HDB9 Strap or equal

SWL = **6,580** lbs

(Use 3-1/2" Stud)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 1,926 lbs

P_{allow} = 405 psi

1,926 / 450 = 4.28 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: Y-DIRECTION

SW Y 006

		Wall Loading		Wall Length		Wall Height		Overturning Moment
M_o	=	383 psf	x	15.00 ft	x	4.08 ft	=	23,434 ft-lbs

			60% Floor Weight		Floor Trib. Area		1/2 Wall Length		Resisting Moment
M_{roof} =	M_{wall}	=	6.00 psf	x	200 ft	x	7.50 ft	=	9000 ft-lbs
									9,000 ft-lbs

M_U = 23,434 - 9,000 = 14,434 ft-lbs

Tiedown Load:

14,434 FT-LBS / 14.71 FT = **981** LBS + - LBS = **981** LBS

Assume 2 - 2x6 end studs. From Wall Above

holddown is at center of end studs.

Use Simpson type: HD9B Strap or equal

SWL = **6,580** lbs

(Use 3-1/2" Stud)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate) (NDS 2005 Edition, Table 4A)

P_{max} = 981 lbs

P_{allow} = 405 psi

981 / 450 = 2.18 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: Y-DIRECTION

SW Y 007

	Wall Loading		Wall Length		Wall Height		Overturning Moment
	psf		ft		ft		
M_o =	247	x	5.81	x	4.08	=	5,859 ft-lbs

		60% Floor Weight		Floor Trib. Area		1/2 Wall Length		Resisting Moment
		psf		ft		ft		
M_{roof} =	M_{wall} =	6.00	x	50	x	2.91	=	871.5 ft-lbs
								872 ft-lbs

M_U = 5,859 - 872 = 4,987 ft-lbs

Tiedown Load:

FT-LBS		FT						
4,987	/	5.52	=	**903** LBS	+	- LBS	=	**903** LBS
				Assume 2 - 2x6 end studs.		From Wall Above		

holddown is at center of end studs.

Use Simpson type: HD9B Strap or equal

SWL = **6,580** lbs

(Use 3-1/2" Stud)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 903 lbs

P_{allow} = 405 psi

903 / 450 = 2.01 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: Y-DIRECTION

SW Y 008

		Wall Loading		Wall Length		Wall Height		Overturning Moment
		psf		ft		ft		
M_o	=	239	x	6.00	x	4.08	=	5,859 ft-lbs

				60% Floor Weight		Floor Trib. Area		1/2 Wall Length		Resisting Moment
				psf		ft		ft		
M_{roof}	=	M_{wall}	=	6.00	x	50	x	3.00	=	900 ft-lbs
										900 ft-lbs

M_U = 5,859 - 900 = 4,959 ft-lbs

Tiedown Load:

FT-LBS		FT						
4,959	/	5.71	=	**868** LBS	+	- LBS	=	**868** LBS

Assume 2 - 2x6 end studs. From Wall Above

holddown is at center of end studs.

Use Simpson type: HD12 Strap or equal

SWL = **10,765** lbs

(Use 4-1/2" Stud)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 868 lbs

P_{allow} = 405 psi

868 / 450 = 1.93 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

CONCRETE

OVERTURNING CALCULATIONS: **X-DIRECTION**

SW X 101

NOT USED

OVERTURNING CALCULATIONS: X-DIRECTION

SW X 102

	Wall Loading		Wall Length		Wall Height		Overturning Moment
M_o =	425 psf	x	2.58 ft	x	9.00 ft	=	9,863 ft-lbs

		60% Floor Weight		Floor Trib. Area		1/2 Wall Length		Resisting Moment
M_{roof} =	M_{wall} =	6.00 psf	x	150 ft	x	1.29 ft	=	1161 ft-lbs
								1,161 ft-lbs

M_U = 9,863 - 1,161 = 8,702 ft-lbs

Tiedown Load:

8,702 FT-LBS / 2.29 FT = **3,800** LBS + 592 LBS = **4,391** LBS

Assume 2 - 2x6 end studs.
holddown is at center of end studs.

Load From Wall Above

Use Simpson type: **CMSTC16** **Strap** **or equal**

SWL = **4,585** lbs

(Use 58-16d nails)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 4,391 lbs

P_{allow} = 405 psi

4,391 / 405 = 10.84 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: X-DIRECTION

SW X 103

		Wall Loading		Wall Length		Wall Height		Overturning Moment
		psf		ft		ft		
M_o	=	365	x	3.00	x	9.00	=	9,863 ft-lbs

			60% Floor Weight		Floor Trib. Area		1/2 Wall Length		Resisting Moment
			psf		ft		ft		
M_{roof}	=	M_{wall} =	6.00	x	150	x	1.50	=	1350 ft-lbs
									1,350 ft-lbs

M_U = 9,863 - 1,350 = 8,513 ft-lbs

Tiedown Load:

FT-LBS		FT						
8,513	/	2.71	=	**3,141** LBS	+	418 LBS	=	**3,560** LBS

Assume 2 - 2x6 end studs.
holddown is at center of end studs.

Load From Wall Above

Use Simpson type: **CMSTC16** **Strap** **or equal**

SWL = **4,585** lbs

(Use 58-16d nails)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 3,560 lbs

P_{allow} = 405 psi

3,560 / 405 = 8.79 sq-inches   Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: X-DIRECTION

SW X 104

		Wall Loading		**Wall Length**		**Wall Height**		**Overturning Moment**
		psf		ft		ft		
M_o	=	365	x	3.00	x	9.00	=	9,863 ft-lbs

			60% Floor Weight		**Floor Trib. Area**		**1/2 Wall Length**		**Resisting Moment**
			psf		ft		ft		
M_{roof} =	M_{wall}	=	6.00	x	150	x	1.50	=	1350 ft-lbs
									1,350 ft-lbs

M_U = 9,863 - 1,350 = 8,513 ft-lbs

Tiedown Load:

FT-LBS		FT						
8,513	/	2.71	=	**3,141** LBS	+	**418** LBS	=	**3,560** LBS

Assume 2 - 2x6 end studs. Load From Wall Above

holddown is at center of end studs.

Use Simpson type: CMSTC16 Strap or equal

SWL = **4,585** lbs

(Use 58-16d nails)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 3,560 lbs

P_{allow} = 405 psi

3,560 / 405 = 8.79 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: X-DIRECTION

SW X 105

	Wall Loading		Wall Length		Wall Height		Overturning Moment
	psf		ft		ft		
M_o =	365	x	3.00	x	9.00	=	9,863 ft-lbs

		60% Floor Weight		Floor Trib. Area		1/2 Wall Length		Resisting Moment
		psf		ft		ft		
M_{roof} =	M_{wall} =	6.00	x	150	x	1.50	=	1350 ft-lbs
								1,350 ft-lbs

M_U = 9,863 - 1,350 = 8,513 ft-lbs

Tiedown Load:

FT-LBS		FT						
8,513	/	2.71	=	**3,141** LBS	+	837 LBS	=	**3,978** LBS

Assume 2 - 2x6 end studs.
holddown is at center of end studs.
Load From Wall Above

Use Simpson type: **CMSTC16** **Strap** **or equal**

SWL = **4,585** lbs
(Use 58-16d nails)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate) (NDS 2005 Edition, Table 4A)

P_{max} = 3,978 lbs

P_{allow} = 405 psi

3,978 / 405 = 9.82 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: X-DIRECTION

SW X 106

		Wall Loading		Wall Length		Wall Height		Overturning Moment
		psf		ft		ft		
M_o	=	146	x	30.00	x	9.00	=	39,450 ft-lbs

			60% Floor Weight		Floor Trib. Area		1/2 Wall Length		Resisting Moment
			psf		ft		ft		
M_{roof} =	M_{wall}	=	6.00	x	600	x	15.00	=	54000 ft-lbs
									54,000 ft-lbs

M_U = 39,450 - 54,000 = (14,550) ft-lbs

Tiedown Load:

FT-LBS		FT						
(14,550)	/	29.71	=	**(490)** LBS	+	**(1,618)** LBS	=	**(2,107)** LBS

Assume 2 - 2x6 end studs. Load From Wall Above

holddown is at center of end studs.

Use Simpson type: **Non Required** **Strap** **or equal**

SWL = **N/A** lbs

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate) (NDS 2005 Edition, Table 4A)

P_{max} = (2,107) lbs

P_{allow} = 405 psi

(2,107) / 405 = -5.20 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: Y-DIRECTION

SW Y 101

	Wall Loading		Wall Length		Wall Height		Overturning Moment
	psf		ft		ft		
M_o =	822	x	12.00	x	9.00	=	88,763 ft-lbs

		60% Floor Weight		Floor Trib. Area		1/2 Wall Length		Resisting Moment
		psf		ft		ft		
M_{roof} =	M_{wall} =	6.00	x	450	x	6.00	=	16200 ft-lbs
								16,200 ft-lbs

M_U = 88,763 - 16,200 = 72,563 ft-lbs

Tiedown Load:

FT-LBS		FT						
72,563	/	11.71	=	**6,197** LBS	+	**1,362** LBS	=	**7,559** LBS
		Assume 2 - 2x6 end studs.				Load From Wall Above		

holddown is at center of end studs.

Use Simpson type: **CMST12** **Strap** **or equal** **HD9B HOLDOWN**

SWL = **9,215** lbs (Use 84-16d nails) **7,740** lbs

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate) (NDS 2005 Edition, Table 4A)

P_{max} = 7,559 lbs

P_{allow} = 405 psi

7,559 / 450 = 16.80 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.50 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: **X-DIRECTION**

SW Y 102

	Wall Loading		Wall Length		Wall Height		Overturning Moment
	psf		ft		ft		
M_o =	658	x	15.00	x	9.00	=	88,763 ft-lbs

		60% Floor Weight		Floor Trib. Area		1/2 Wall Length		Resisting Moment
		psf		ft		ft		
M_{roof} =	M_{wall} =	6.00	x	450	x	7.50	=	20250 ft-lbs
								20,250 ft-lbs

M_U = 88,763 - 20,250 = 68,513 ft-lbs

Tiedown Load:

FT-LBS		FT						
68,513	/	14.71	=	**4,658** LBS	+	778 LBS	=	**5,435** LBS

Assume 2 - 2x6 end studs.
holddown is at center of end studs.

Load From Wall Above

Use Simpson type: **CMST14** **Strap** **or equal**

SWL = **6,490** lbs

(Use 66-16d nails)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 5,435 lbs

P_{allow} = 405 psi

5,435 / 450 = 12.08 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: X-DIRECTION

SW Y 103

	Wall Loading (psf)		Wall Length (ft)		Wall Height (ft)		Overturning Moment
M_o =	219	x	15.00	x	9.00	=	29,588 ft-lbs

	60% Floor Weight (psf)		Floor Trib. Area (ft)		1/2 Wall Length (ft)		Resisting Moment
M_{roof} = M_{wall} =	6.00	x	150	x	7.50	=	6750 ft-lbs
							6,750 ft-lbs

M_U = 29,588 - 6,750 = 22,838 ft-lbs

Tiedown Load:

FT-LBS		FT						
22,838	/	14.71	=	**1,553** LBS	+	**260** LBS	=	**1,813** LBS

Assume 2 - 2x6 end studs.
holddown is at center of end studs.

Load From Wall Above

Use Simpson type: **CS14** **Strap** **or equal**

SWL = **2,158** lbs

(Use 26-10d nails)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 1,813 lbs

P_{allow} = 405 psi

1,813 / 450 = 4.03 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: X-DIRECTION

SW Y 104

	Wall Loading		Wall Length		Wall Height		Overturning Moment
	psf		ft		ft		
M_o =	398	x	8.25	x	9.00	=	29,588 ft-lbs

		60% Floor Weight		Floor Trib. Area		1/2 Wall Length		Resisting Moment
		psf		ft		ft		
M_{roof} =	M_{wall} =	6.00	x	150	x	4.13	=	3712.5 ft-lbs
								3,713 ft-lbs

M_U = 29,588 - 3,713 = 25,875 ft-lbs

Tiedown Load:

FT-LBS		FT						
25,875	/	7.96	=	**3,251** LBS	+	**454** LBS	=	**3,704** LBS

Assume 2 - 2x6 end studs.
holddown is at center of end studs.

Load From Wall Above

Use Simpson type: **CMSTC16** **Strap** **or equal**

SWL = **4,585** lbs

(Use 58-16d nails)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 3,704 lbs

P_{allow} = 405 psi

3,704 / 450 = 8.23 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: X-DIRECTION

SW X 201

	Wall Loading		Wall Length		Wall Height		Overturning Moment
	psf		ft		ft		
M_o =	126	x	3.00	x	10.00	=	3,790 ft-lbs

	60% Roof Weight		Roof Trib. Area		1/2 Wall Length		Resisting Moment
	psf		ft		ft		
M_{roof} = M_{wall} =	5.88	x	171	x	1.50	=	1508.22 ft-lbs
							1,508 ft-lbs

M_U = 3,790 - 1,508 = 2,282 ft-lbs

Tiedown Load:

2,282 FT-LBS / 2.71 FT = **842** LBS

Assume 2 - 2x6 end studs.
holddown is at center of end studs.

Use Simpson type: CS22 Strap or equal

SWL = **845** lbs

(Use 12-10d nails)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 842 lbs

P_{allow} = 405 psi

842 / 405 = 2.08 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: X-DIRECTION

SW X 202

	Wall Loading		Wall Length		Wall Height		Overturning Moment
	psf		ft		ft		
M_o =	105	x	3.62	x	10.00	=	3,790 ft-lbs

	60% Roof Weight		Roof Trib. Area		1/2 Wall Length		Resisting Moment
	psf		ft		ft		
M_{roof} = M_{wall} =	5.88	x	171	x	1.81	=	1819.9188 ft-lbs
							1,820 ft-lbs

M_U = 3,790 - 1,820 = 1,970 ft-lbs

Tiedown Load:

FT-LBS		FT		
1,970	/	3.33	=	**592** LBS

Assume 2 - 2x6 end studs.

holddown is at center of end studs.

Use Simpson type: CS22 Strap or equal

SWL = **845** lbs

(Use 12-10d nails)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate) (NDS 2005 Edition, Table 4A)

P_{max} = 592 lbs

P_{allow} = 405 psi

592 / 405 = 1.46 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: X-DIRECTION

SW X 203

	Wall Loading		Wall Length		Wall Height		Overturning Moment
	psf		ft		ft		
M_o =	63	x	3.00	x	10.00	=	1,884 ft-lbs

		60% Roof Weight		Roof Trib. Area		1/2 Wall Length		Resisting Moment
		psf		ft		ft		
M_{roof} =	M_{wall} =	5.88	x	85	x	1.50	=	749.7 ft-lbs
								750 ft-lbs

M_U = 1,884 - 750 = 1,134 ft-lbs

Tiedown Load:

FT-LBS		FT		
1,134	/	2.71	=	**418** LBS

Assume 2 - 2x6 end studs.
holddown is at center of end studs.

Use Simpson type: **CS22** **Strap** **or equal**

SWL = **845** lbs
(Use 12-10d nails)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 418 lbs

P_{allow} = 405 psi

418 / 405 = 1.03 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: X-DIRECTION

SW X 204

		Wall Loading		Wall Length		Wall Height		Overturning Moment
		psf		ft		ft		
M_o	=	63	x	3.00	x	10.00	=	1,884 ft-lbs

			60% Roof Weight		Roof Trib. Area		1/2 Wall Length		Resisting Moment
			psf		ft		ft		
M_{roof}	=	M_{wall} =	5.88	x	85	x	1.50	=	749.7 ft-lbs
									750 ft-lbs

M_U = 1,884 - 750 = 1,134 ft-lbs

Tiedown Load:

FT-LBS		FT		
1,134	/	2.71	=	**418** LBS

Assume 2 - 2x6 end studs.
holddown is at center of end studs.

Use Simpson type: CS22 Strap or equal

SWL = **845** lbs

(Use 12-10d nails)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 418 lbs

P_{allow} = 405 psi

418 / 405 = 1.03 sq-inches   Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: X-DIRECTION

SW X 205

		Wall Loading		Wall Length		Wall Height		Overturning Moment
M_o	=	126 psf	x	3.00 ft	x	10.00 ft	=	3,768 ft-lbs

			60% Roof Weight		Roof Trib. Area		1/2 Wall Length		Resisting Moment
M_{roof}	=	M_{wall} =	5.88 psf	x	170 ft	x	1.50 ft	=	1499.4 ft-lbs
									1,499 ft-lbs

M_U = 3,768 - 1,499 = 2,268 ft-lbs

Tiedown Load:

2,268 FT-LBS / 2.71 FT = **837** LBS

Assume 2 - 2x6 end studs.

holddown is at center of end studs.

Use Simpson type: CS22 Strap or equal

SWL = **845** lbs

(Use 12-10d nails)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate) (NDS 2005 Edition, Table 4A)

P_{max} = 837 lbs

P_{allow} = 405 psi

837 / 405 = 2.07 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: X-DIRECTION

SW X 206

		Wall Loading		Wall Length		Wall Height		Overturning Moment
M_o	=	50 psf	x	30.00 ft	x	8.00 ft	=	12,092 ft-lbs

			60% Roof Weight		Roof Trib. Area		1/2 Wall Length		Resisting Moment
M_{roof} =	M_{wall}	=	5.88 psf	x	682 ft	x	15.00 ft	=	60152.4 ft-lbs
									60,152 ft-lbs

M_U = 12,092 - 60,152 = (48,060) ft-lbs

Tiedown Load:

(48,060) FT-LBS / 29.71 FT = **(1,618)** LBS

Assume 2 - 2x6 end studs.

holddown is at center of end studs.

Use Simpson type: **Non Required** **Strap** **or equal**

SWL = **N/A** lbs

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate) (NDS 2005 Edition, Table 4A)

P_{max} = (1,618) lbs

P_{allow} = 405 psi

(1,618) / 405 = -3.99 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: Y-DIRECTION

SW Y 201

	Wall Loading		Wall Length		Wall Height		Overturning Moment
	psf		ft		ft		
M_o =	283	x	12.00	x	10.00	=	33,976 ft-lbs

	60% Roof Weight		Roof Trib. Area		1/2 Wall Length		Resisting Moment
	psf		ft		ft		
M_{roof} = M_{wall} =	5.88	x	511	x	6.00	=	18028.08 ft-lbs
							18,028 ft-lbs

M_U = 33,976 - 18,028 = 15,947 ft-lbs

Tiedown Load:

FT-LBS		FT		
15,947	/	11.71	=	**1,362** LBS

Assume 2 - 2x6 end studs.
holddown is at center of end studs.

Use Simpson type: CS16 Strap or equal

SWL = **1,705** lbs

(Use 22-10d nails)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate) (NDS 2005 Edition, Table 4A)

P_{max} = 1,362 lbs

P_{allow} = 405 psi

1,362 / 450 = 3.03 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: X-DIRECTION

SW Y 202

		Wall Loading		Wall Length		Wall Height		Overturning Moment
		psf		ft		ft		
M_o	=	227	x	15.00	x	10.00	=	33,976 ft-lbs

			60% Roof Weight		Roof Trib. Area		1/2 Wall Length		Resisting Moment
			psf		ft		ft		
M_{roof} =	M_{wall}	=	5.88	x	511	x	7.50	=	22535.1 ft-lbs
									22,535 ft-lbs

M_U = 33,976 - 22,535 = 11,440 ft-lbs

Tiedown Load:

FT-LBS		FT		
11,440	/	14.71	=	**778** LBS

Assume 2 - 2x6 end studs.
holddown is at center of end studs.

Use Simpson type: CS22 Strap or equal

SWL = **845** lbs
(Use 12-10d nails)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 778 lbs

P_{allow} = 405 psi

778 / 450 = 1.73 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: X-DIRECTION

SW Y 203

		Wall Loading (psf)		Wall Length (ft)		Wall Height (ft)		Overturning Moment
M_o	=	76	x	15.00	x	10.00	=	11,369 ft-lbs

			60% Roof Weight (psf)		Roof Trib. Area (ft)		1/2 Wall Length (ft)		Resisting Moment
M_{roof}	=	M_{wall} =	5.88	x	171	x	7.50	=	7541.1 ft-lbs
									7,541 ft-lbs

M_U = 11,369 - 7,541 = 3,828 ft-lbs

Tiedown Load:

3,828 FT-LBS / 14.71 FT = **260** LBS

Assume 2 - 2x6 end studs.
holddown is at center of end studs.

Use Simpson type: **CS22** **Strap** **or equal**

SWL = **845** lbs

(Use 12-10d nails)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 260 lbs

P_{allow} = 405 psi

260 / 450 = 0.58 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

OVERTURNING CALCULATIONS: X-DIRECTION

SW Y 204

		Wall Loading		Wall Length		Wall Height		Overturning Moment
		psf		ft		ft		
M_o	=	138	x	8.25	x	10.00	=	11,369 ft-lbs

			60% Roof Weight		Roof Trib. Area		1/2 Wall Length		Resisting Moment
			psf		ft		ft		
M_{roof}	=	M_{wall} =	5.88	x	171	x	4.13	=	4147.605 ft-lbs
									4,148 ft-lbs

M_U = 11,369 - 4,148 = 7,222 ft-lbs

Tiedown Load:

FT-LBS		FT		
7,222	/	7.96	=	**907** LBS

Assume 2 - 2x6 end studs.
holddown is at center of end studs.

Use Simpson type: CS18 Strap or equal

SWL = **1,370** lbs

(Use 14-10d nails)

Check End Stud Bearing:

Assume: Hem Fir No. 2 or bettyer Framing Lumber

Alowable Compression Perpendicular to Grain = **405** PSI (End Stud to Bottom Plate)
(NDS 2005 Edition, Table 4A)

P_{max} = 907 lbs

P_{allow} = 405 psi

907 / 450 = 2.02 sq-inches Assume 2 - 2x6 end studs.

Bearing Area = 16.5 sq-inches

Therefore use 2 - 2x6 End Studs

ROOF SHEATHING DESIGN:

Assumptions:

1. Plywood shall be: **1/2 inch** APA Rated Sheathing Ext.(3)
2. 2 x _ Trusses spaced at: **24** inches o.c.. maximum
   Therefore: L_1 = 24
   L_2 = 22.5
   L_3 = 22.75
3. Plywood spans over three or more supports.
4. Roof Design Live Load: **LL** = **16** (See UBC Table 16-C)
5. Roof Design Snow Load: **SL** = **25**

1/2 inch Plywood Properties:

APA Design Values

t_s	=	**0.5**	inches
A	=	**2.884**	inches^2
I	=	**0.075**	inches^3
KS	=	**0.267**	inches^4
lb/Q	=	**4.891**	inches^2
F_b	=	**1930**	PSI
F_s	=	**72**	PSI
E_e	=	**1,500,000**	PSI
E	=	**1,650,000**	PSI

Check Plywood Stresses:

A) Check Bending:

$KS_{Req'd}$ = $W(L_1)^2 / 120(F_b)$ = **0.126** inches^4

Where **W** =D.L +Roof L.L. + Roof Snow I 50.8 PSF

$KS_{Furnished}$ = **0.267** inches^4

Check - O.K.

B) Check Rolling Shear:

$Fs_{Req'd}$ = $W(L_2) / 20(lb/Q)$ = **8.00** PSI

$Fs_{Furnished}$ = $L_2/360$ = **72** PSI

Check - O.K.

C) Check Shear Deflection:

$D_{maximum}$ = $[WC(t_s)^2(L_2)^2] / (1270\ E_e\ I)$ = **0.0018** inches

$D_{Allowable}$ = $L_2/360$ = **0.0625** inches

Check - O.K.

D) Check Bending Deflection:

$D_{maximum}$ = $[W(L_3)^4] / 1743\ E\ I$ = **0.0036** inches

$D_{Allowable}$ = $L_3/360$ = **0.0632** inches

Check - O.K.

FIRST, SECOND & THIRD FLOOR SHEATHING DESIGN:

Assumptions:

1. Plywood shall be: **3/4 inch** APA Rated Strd-I-Floor Ext.1 or 2[(3)]
2. 2 x _ floor joists spaced at: **16** inches o.c.. maximum
   Therefore: L_1 = 16
   L_2 = 12.5
   L_3 = 12.75
3. Plywood spans over three or more supports.
4. Floor Design Live Load: **LL** = **40**
   (See UBC Table 16-A)

3/4 inch Plywood Properties:

		APA Design Values
t_s	=	**0.568** inches
A	=	**2.884** inches^2
I	=	**0.199** inches^3
KS	=	**0.455** inches^4
lb/Q	=	**7.187** inches^2
F_b	=	**1930** PSI
F_s	=	**72** PSI
E_e	=	**1,500,000** PSI
E	=	**1,650,000** PSI

Check Plywood Stresses:

A) Check Bending:

$KS_{Req'd}$ = $W(L_1)^2 / 120(F_b)$ = **0.055** inches^4

$KS_{Furnished}$ = **0.455** inches^4

Check - O.K.

B) Check Rolling Shear:

$Fs_{Req'd}$ = $W(L_2) / 20(lb/Q)$ = **4.35** PSI

$Fs_{Furnished}$ = $L_2/360$ = **72** PSI

Check - O.K.

C) Check Shear Deflection:

$D_{maximum}$ = $[WC(t_s)^2(L_2)^2] / (1270\ E_e\ I)$ = **0.0004**

$D_{Allowable}$ = $L_2/360$ = **0.0347**

Check - O.K.

D) Check Bending Deflection:

$D_{maximum}$ = $[W(L_3)^4] / 1743\ E\ I$ = **0.0023** inches

$D_{Allowable}$ = $L_3/360$ = **0.0354** inches

Check - O.K.

BEAM DESIGN:

Assumptions:

1. 5.125 x 9.0 GLULAM Beam Western Species
2. Span = 20.00 feet
   240 inches o.c. max.
3. Roof Span suppported: 8 feet
   Roof Span suppported: 12 feet
4. Assume bearing area = (3.5" x 5.125") 17.94 $inches^2$

5.125 x 9 GluLam Girder Properties:

Reference: APA Glued Laminated Beam Desing Tables
Western Species GluLam Beam

A	=	**20.10**	$inches^2$
E	=	**1,600,000**	PSI
S	=	**206**	inches3
I	=	**1,547**	inches4
$Fv_{Allowable}$	=	**13,200**	LBS
$Fp_{Allowable}$	=	**405**	PSI
Fb	=	**1,000**	PSI
$Mm_{Allowable}$	=	**41,250**	FT-LBS

Reactions:

W = Roof Load x Span Supported 550.00 PLF

Where **W** =

Floor D.L.	=	15	PSF
Floor L.L.	=	40	PSF
		55	PSF

Reactions:

V_{max}	=	$0.5WL$	=	**5,500**	lbs
P_{max}	=	$0.5WL$	=	**5,500**	lbs
M_{max}	=	$(WL^2)/8$	=	**27,500**	ft-lbs
D_{max}	=	$0.013WL^4 / EI$	=	**0.07**	inches

Check Shear:

Fv	=	$V_{max}(1.5) / A$	=	**5,500**	lbs
$Fv_{Allowable}$	=			**13,200**	lbs

Check - O.K.

Check Bearing:

Fp	=	$\mathbf{P_{max} / A}$	=	**306.62** PSI
$Fp_{Allowable}$	=			**405** PSI
				Check - O.K.

Check Bending:

$\mathbf{M_{max}}$	=		=	**27,500** FT-LBS
$\mathbf{Mm}_{Allowable}$	=			**41,250** FT-LBS
				Check - O.K.

Check Deflection:

$D_{max.}$	=			**0.0666** inches
$D_{Allowable}$	=	**L**/360	=	**0.6667** inches
				Check - O.K.

BEAM DESIGN:

Assumptions:

1. 5.125 x 9.0 GLULAM Beam Western Species			
2. Span =		15.50	feet
		186	inches o.c. max.
3. Roof Span supperted:		**15.5**	feet
Roof Span supperted:		**0**	feet
4. Assume bearing area =	(3.5" x 5.125")	17.94	$inches^2$

5.125 x 9 GluLam Girder Properties:

Reference: APA Glued Laminated Beam Desing Tables
Western Species GluLam Beam

A	=	**20.10**	$inches^2$
E	=	**1,600,000**	PSI
S	=	**206**	inches3
I	=	**1,547**	inches4
$\mathbf{Fv}_{Allowable}$	=	**13,200**	LBS
$\mathbf{Fp}_{Allowable}$	=	**405**	PSI
Fb	=	**1,000**	PSI
$\mathbf{Mm}_{Allowable}$	=	**41,250**	FT-LBS

Reactions:

W	=	Roof Load x Span Supported	426.25	PLF
Where **W** =				
	Floor D.L.	=	15	PSF
	Floor L.L.	=	40	PSF
			55	PSF

Reactions:

$\mathbf{V}_{max}$	=	$0.5\mathbf{WL}$	=	**3,303**	lbs
$\mathbf{P}_{max}$	=	$0.5\mathbf{WL}$	=	**3,303**	lbs
$\mathbf{M}_{max}$	=	$(\mathbf{WL}^2)/8$	=	**12,801**	ft-lbs
$\mathbf{D}_{max}$	=	$0.013\mathbf{WL}^4 / \mathbf{EI}$	=	**0.02**	inches

Check Shear:

Fv	=	$\mathbf{V}_{max}(1.5) / \mathbf{A}$	=	**3,303**	lbs
$Fv_{Allowable}$	=			**13,200**	lbs

Check - O.K.

Check Bearing:

F_p	=	$\mathbf{P_{max} / A}$	=	**184.16** PSI
$F_{p_{Allowable}}$	=			**405** PSI
				Check - O.K.

Check Bending:

$\mathbf{M_{max}}$	=		=	**12,801** FT-LBS
$\mathbf{Mm}_{Allowable}$	=			**41,250** FT-LBS
				Check - O.K.

Check Deflection:

$D_{max.}$	=			**0.0186** inches
$D_{Allowable}$	=	**L**/360	=	**0.5167** inches
				Check - O.K.

TJI - FLOOR JOIST DESIGN:

Main Level

Assumptions:

1. 9-1/2" TJI PRO 360 Foor Joists
2. Supports spaced @: **20** feet
3. Floor Joist Spacing: **1.33** feet O.C.
4. Minimum Bearing Area: 2.31" x 3.5" = **8.09** $inches^2$

9-1/2" PRI 40 TJI Properties:

Weight	=	**2.37**	LBS
EI	=	**215,000,000**	PSI
$M_{Allowable}$	=	**3,760**	$inches^4$
$V_{Allowable}$	=	**1,330**	$inches^3$
$Pe_{Allowable}$	=	**2000**	PSI
$Ps_{Allowable}$	=	**2000**	PSI

Reactions: From Uniform Loading:

P_{max}	=	**W** x Joist Spacing	=	**73**	PLF
Where **W**	=	Floor D.L.	=	15	PSF
		Floor L.L	=	40	PSF
				55	PSF

V_{max}	=	$P_{max}/2$	=	**732**	lbs
P_{max}	=	V_{max}	=	**732**	lbs
M_{max}	=	$(WL^2)/8$	=	**3,658**	ft-lbs
D_{max}	=	$5WL^4/384EI$	=	**0.25**	inches

Check Shear:

V_{max}	=	=	**732**	PSI
$V_{Allowable}$	=	=	**2,000**	PSI

Check - O.K.

Check Bearing:

P_{max}	=	=	**732**	PSI
$Ps_{Allowable}$	=		**2,000**	PSI

Check - O.K.

Check Bending:

$\mathbf{M_{max}}$	=		=	**3,658** inches^3
$\mathbf{M}_{Allowable}$	=			**3,760** inches^3
				Check - O.K.

Check Deflection:

$D_{max.}$	=			**0.25** inches
$D_{Allowable}$	=	**L**/350	=	**0.69** inches
				Check - O.K.

TJI - FLOOR JOIST DESIGN:

Main Level

Assumptions:

1. 11-7/8" TJI PRI 40 Roof Rafters
2. Supports spaced @: **20** feet
3. Floor Joist Spacing: **2.00** feet O.C.
4. Minimum Bearing Area: 2.31" x 3.5" = **8.09** $inches^2$

11-7/8" PRI 40 TJI Properties:

Weight	=	**2.69**	LBS
EI	=	**1,336,000,000**	PSI
$M_{Allowable}$	=	**4,855**	$inches^4$
$V_{Allowable}$	=	**1,550**	$inches^3$
$Pe_{Allowable}$	=	**2000**	PSI
$Ps_{Allowable}$	=	**2000**	PSI

Reactions: From Uniform Loading:

P_{max}	=	**W** x Rafter Spacing	=	**80**	PLF
Where **W**	=	Roof D.L.	=	15	PSF
		Roof L.L	=	25	PSF
				40	PSF

V_{max}	=	P_{max} /2	=	**800**	lbs
P_{max}	=	V_{max}	=	**800**	lbs
M_{max}	=	$(WL^2)/8$	=	**4,000**	ft-lbs
D_{max}	=	$5WL^4/384EI$	=	**0.04**	inches

Check Shear:

V_{max} = = **800** PSI

$V_{Allowable}$ = = **2,000** PSI

Check - O.K.

Check Bearing:

P_{max} = = **800** PSI

$Ps_{Allowable}$ = **2,000** PSI

Check - O.K.

Check Bending:

$\mathbf{M_{max}}$	=		=	**4,000** inches^3
$\mathbf{M}_{Allowable}$	=			**4,855** inches^3
				Check - O.K.

Check Deflection:

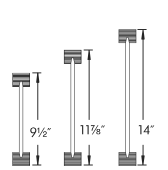
$D_{max.}$	=			**0.04** inches
$D_{Allowable}$	=	**L**/350	=	**0.69** inches
				Check - O.K.

RFPI®-Joist Design Properties

I-JOIST DIMENSIONS

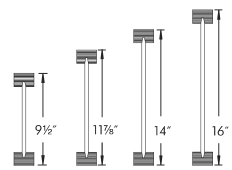
LVL FLANGE

RFPI® 20



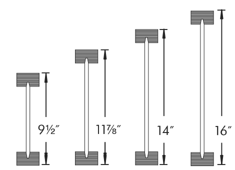
1-3/4" wide x 1-3/8" LVL Flange
3/8" OSB Web

RFPI® 400



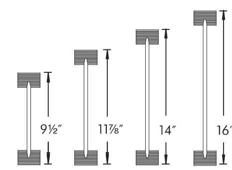
2-1/16" wide x 1-3/8" LVL Flange
3/8" OSB Web

RFPI® 40



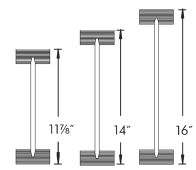
2-5/16" wide x 1-3/8" LVL Flange
3/8" OSB Web

RFPI® 70



2-5/16" wide x 1-1/2" LVL Flange
3/8" OSB Web

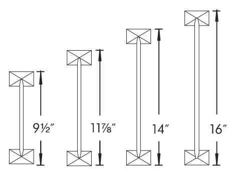
RFPI® 90



3-1/2" wide x 1-1/2" LVL Flange
7/16" OSB Web

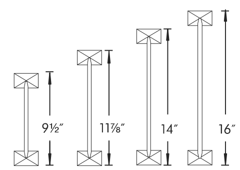
SOLID SAWN FLANGE

RFPI® 40S



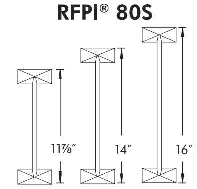
2-1/2" wide x 1-1/2" Solid Sawn Flange
3/8" OSB Web

RFPI® 60S



2-1/2" wide x 1-1/2" Solid Sawn Flange
3/8" OSB Web

RFPI® 80S



3-1/2" wide x 1-1/2" Solid Sawn Flange
3/8" OSB Web

DESIGN PROPERTIES FOR RFPI-JOISTS(1)

Roseburg Designation	APA Designation	EI(2) x10⁶ lb-in.²	M(3) lb-ft	V(4) lbs	VLC(5) lbs/ft.	K(6) x10⁶ lb	Weight plf
9½" RFPI 20 (7)	9½" PRI 20	165	2,820	1,220	2,000	4.94	1.99
9½" RFPI 40S (7)	9½" PRI 40	193	2,735	1,120	2,000	4.94	2.56
9½" RFPI 400	Not Applicable	193	3,345	1,220	2,000	4.94	2.29
9½" RFPI 40 (7)	9½" PRI 40	215	3,760	1,330	2,000	4.94	2.37
9½" RFPI 60S (7)	9½" PRI 60	231	3,780	1,120	2,000	4.94	2.56
9½" RFPI 70	Not Applicable	266	5,130	1,330	2,000	4.94	2.57
11⅞" RFPI 20 (7)	11⅞" PRI 20	283	3,640	1,420	2,000	6.18	2.30
11⅞" RFPI 40S (7)	11⅞" PRI 40	330	3,545	1,420	2,000	6.18	2.83
11⅞" RFPI 400	Not Applicable	330	4,315	1,480	2,000	6.18	2.60
11⅞" RFPI 40 (7)	11⅞" PRI 40	366	4,855	1,550	2,000	6.18	2.69
11⅞" RFPI 60S (7)	11⅞" PRI 60	396	4,900	1,420	2,000	6.18	2.83
11⅞" RFPI 70 (7)	11⅞" PRI 70	455	6,645	1,550	2,000	6.18	2.91
11⅞" RFPI 80S (7)	11⅞" PRI 80	547	6,970	1,590	2,000	6.18	3.79
11⅞" RFPI 90 (7)	11⅞" PRI 90	676	10,145	2,050	2,000	6.18	3.84
14" RFPI 20	Not Applicable	420	4,330	1,610	2,000	7.28	2.51
14" RFPI 40S (7)	14" PRI 40	482	4,270	1,710	2,000	7.28	3.07
14" RFPI 400	Not Applicable	486	5,140	1,710	2,000	7.28	2.79
14" RFPI 40 (7)	14" PRI 40	540	5,785	1,770	2,000	7.28	2.95
14" RFPI 60S (7)	14" PRI 60	584	5,895	1,710	2,000	7.28	3.07
14" RFPI 70 (7)	14" PRI 70	672	7,925	1,770	2,000	7.28	3.13
14" RFPI 80S (7)	14" PRI 80	802	8,390	1,835	2,000	7.28	4.03
14" RFPI 90 (7)	14" PRI 90	992	12,100	2,195	2,000	7.28	4.19
16" RFPI 40S (7)	16" PRI 40	657	4,950	1,970	2,000	8.32	3.31
16" RFPI 400	Not Applicable	665	5,880	1,970	2,000	8.32	3.01
16" RFPI 40 (7)	16" PRI 40	737	6,615	1,970	2,000	8.32	3.14
16" RFPI 60S (7)	16" PRI 60	799	6,835	1,970	2,000	8.32	3.31
16" RFPI 70 (7)	16" PRI 70	918	9,080	1,970	2,000	8.32	3.35
16" RFPI 80S (7)	16" PRI 80	1,092	9,730	2,070	2,000	8.32	4.26
16" RFPI 90 (7)	16" PRI 90	1,350	13,865	2,330	2,000	8.32	4.42

(1) The tabulated values are design values for 100% duration of load. All values except for EI and K are permitted to be adjusted for other load durations as permitted by code, with the further exception that VLC shall not be increased for shorter durations of load. Design values listed are applicable for Allowable Stress Design (ASD).

(2) Bending stiffness (EI) of the I-joist.

(3) Moment capacity (M) of a single I-joist. **Moment capacity of the I-Joist shall not be increased by any repetitive member use factor.**

(4) Shear capacity (V) with a minimum bearing length of 4 inches.

(5) Vertical Load Capacity when continuously supported.

(6) Coefficient of shear deflection (K), used to calculate deflections for I-joist applications. Equations 1 and 2 below are provided for uniform load and center point load conditions for simple spans.

Uniform Load:

$$[1] \quad \delta = \frac{5\omega\ell^4}{384EI} + \frac{\omega\ell^2}{K}$$

Center-Point Load:

$$[2] \quad \delta = \frac{P\ell^3}{48EI} + \frac{2P\ell}{K}$$

where:
δ = calculated deflection (in.)
ω = uniform load (lb/in.)
ℓ = design span (in.)
P = concentrated load (lb)
EI = bending stiffness of the I-joist (lb-in²)
K = coefficient of shear deflection (lb)

(7) Design properties meet or exceed the requirements of the PRI-400 Performance Standard for APA EWS I-Joists for the corresponding I-joist series and depth.

TABLE 1

ALLOWABLE SHEAR (POUNDS PER FOOT) FOR APA PANEL SHEAR WALLS WITH FRAMING OF DOUGLAS-FIR, LARCH, OR SOUTHERN PINE[a] FOR WIND OR SEISMIC LOADING[b,h,i,j,k] (See also IBC Table 2306.4.1)

			Panels Applied Direct to Framing					Panels Applied Over 1/2" or 5/8" Gypsum Sheathing				
Panel Grade	Minimum Nominal Panel Thickness (in.)	Minimum Nail Penetration in Framing (in.)	Nail Size (common or galvanized box)[k]	Nail Spacing at Panel Edges (in.) 6	4	3	2[e]	Nail Size (common or galvanized box)	Nail Spacing at Panel Edges (in.) 6	4	3	2[e]
APA STRUCTURAL I grades	5/16	1-1/4	6d (0.113" dia.)	200	300	390	510	8d (0.131" dia.)	200	300	390	510
	3/8	1-3/8	8d (0.131" dia.)	230[d]	360[d]	460[d]	610[d]	10d (0.148" dia.)	280	430	550[f]	730
	7/16			255[d]	395[d]	505[d]	670[d]					
	15/32			280	430	550	730					
	15/32	1-1/2	10d (0.148" dia.)	340	510	665[f]	870		—	—	—	—
APA RATED SHEATHING; APA RATED SIDING[g] and other APA grades except Species Group 5	5/16 or 1/4[c]	1-1/4	6d (0.113" dia.)	180	270	350	450	8d (0.131 dia.)	180	270	350	450
	3/8			200	300	390	510		200	300	390	510
	3/8	1-3/8	8d (0.131" dia.)	220[d]	320[d]	410[d]	530[d]	10d (0.148" dia.)	260	380	490[f]	640
	7/16			240[d]	350[d]	450[d]	585[d]					
	15/32			260	380	490	640					
	15/32	1-1/2	10d (0.148" dia.)	310	460	600[f]	770	—	—	—	—	—
	19/32			340	510	665[f]	870	—	—	—	—	—
			Nail Size (galvanized casing)					Nail Size (galvanized casing)				
APA RATED SIDING[g] and other APA grades except Species Group 5	5/16[c]	1-1/4	6d (0.113" dia.)	140	210	275	360	8d (0.131" dia.)	140	210	275	360
	3/8	1-3/8	8d (0.131" dia.)	160	240	310	410	10d (0.148" dia.)	160	240	310[f]	410

(a) For framing of other species: Find specific gravity for species of lumber in the AF&PA National Design Specification (NDS). Find shear value from table above for nail size for actual grade and multiply value by the following adjustment factor: Specific Gravity Adjustment Factor = [1 – (0.5 – SG)], where SG = Specific Gravity of the framing lumber. This adjustment shall not be greater than 1.

(b) Panel edges backed with 2 inch nominal or wider framing. Install panels either horizontally or vertically. Space fasteners maximum 6 inches on center along intermediate framing members for 3/8 inch and 7/16 inch panels installed on studs spaced 24 inches on center. For other conditions and panel thicknesses, space nails maximum 12 inches on center on intermediate supports.

(c) 3/8 inch panel thickness or siding with a span rating of 16 inches on center is the minimum recommended where applied direct to framing as exterior siding.

(d) Allowable shear values are permitted to be increased to values shown for 15/32 inch sheathing with same nailing provided (1) studs are spaced a maximum of 16 inch on center, or (2) panels are applied with long dimension across studs.

(e) Framing at adjoining panel edges shall be 3 inch nominal or wider, and nails shall be staggered where nails are spaced 2 inch on center.

(f) Framing at adjoining panel edges shall be 3 inch nominal or wider, and nails shall be staggered where both the following conditions are met: (1) 10d (3 inch x 0.148 inch) nails having penetration into framing of more than 1-1/2 inch and (2) nails are spaced 3 inch on center.

(g) Values apply to all-veneer plywood. Thickness at point of fastening on panel edges governs shear values.

(h) Where panels applied on both faces of a wall and nail spacing is less than 6 inches o.c. on either side, panel joints shall be offset to fall on different framing members, or framing shall be 3 inch nominal or thicker at adjoining panel edges and nails on each side shall be staggered.

(i) In Seismic Design Category D, E or F, where shear design values exceed 350 pounds per lineal foot, all framing members receiving edge nailing from abutting panels shall not be less than a single 3 inch nominal member, or two 2 inch nominal members fastened together in accordance with IBC Section 2306.1 to transfer the design shear value between framing members. Wood structural panel joint and sill plate nailing shall be staggered in all cases. See IBC Section 2305.3.11 for sill plate size and anchorage requirements.

(j) Galvanized nails shall be hot dipped or tumbled.

(k) For shear loads of normal or permanent load duration as defined by the AF&PA NDS, the values in the table above shall be multiplied by 0.63 or 0.56, respectively.

Typical Layout for Shear Walls

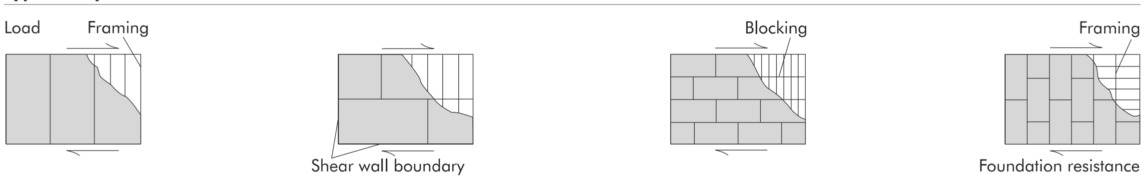


TABLE B-1

ALLOWABLE SHEAR (POUNDS PER FOOT) FOR WOOD STRUCTURAL PANEL BLOCKED DIAPHRAGMS UTILIZING MULTIPLE ROWS OF FASTENERS (HIGH-LOAD DIAPHRAGMS) WITH FRAMING OF DOUGLAS-FIR-LARCH OR SOUTHERN PINE[a] FOR WIND OR SEISMIC LOADING[b,f,g,h]

Panel Grade[c]	Common Nail Size or Staple Gage	Minimum Fastener Penetration in Framing (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Framing Member at Adjoining Panel Edges and Boundaries[e]	Lines of Fasteners	Blocked Diaphragms — Cases 1 and 2[d] — Fastener Spacing Per Line at Boundaries (in.): 4		2-1/2		2	
						Fastener Spacing Per Line at Other Panel Edges (in.): 6	4	4	3	3	2
APA STRUCTURAL I grades	10d common nails (0.148" dia.)	1-1/2	15/32	3	2	605	815	875	1,150	–	–
				4	2	700	915	1,005	1,290	–	–
				4	3	875	1,220	1,285	1,395	–	–
			19/32	3	2	670	880	965	1,255	–	–
				4	2	780	990	1,110	1,440	–	–
				4	3	965	1,320	1,405	1,790	–	–
			23/32	3	2	730	955	1,050	1,365	–	–
				4	2	855	1,070	1,210	1,565	–	–
				4	3	1,050	1,430	1,525	1,800	–	–
	14 gage staples	2	15/32	3	2	600	600	860	960	1,060	1,200
				4	3	860	900	1,160	1,295	1,295	1,400
			19/32	3	2	600	600	875	960	1,075	1,200
				4	3	875	900	1,175	1,440	1,475	1,795
APA RATED SHEATHING, APA RATED STURD-I-FLOOR and other APA grades except Species Group 5	10d common nails (0.148" dia.)	1-1/2	15/32	3	2	525	725	765	1,010	–	–
				4	2	605	815	875	1,105	–	–
				4	3	765	1,085	1,130	1,195	–	–
			19/32	3	2	650	860	935	1,225	–	–
				4	2	755	965	1,080	1,370	–	–
				4	3	935	1,290	1,365	1,485	–	–
			23/32	3	2	710	935	1,020	1,335	–	–
				4	2	825	1,050	1,175	1,445	–	–
				4	3	1,020	1,400	1,480	1,565	–	–
	14 gage staples[f]	2	15/32	3	2	540	540	735	865	915	1,080
				4	3	735	810	1,005	1,105	1,105	1,195
			19/32	3	2	600	600	865	960	1,065	1,200
				4	3	865	900	1,130	1,430	1,370	1,485
			23/32	4	3	865	900	1,130	1,490	1,430	1,545

For **SI**: 1 inch = 25.4 mm, 1 plf = 14.6 N/m.

(a) For framing of the other species: (1) Find specific gravity for species of framing lumber in AF&PA NDS, (2) For staples find shear value from table above for Structural I panels (regardless of actual grade) and multiply value by 0.82 for species with specific gravity of 0.42 or greater, or 0.65 for all other species. (3) For nails, find shear value from table above for nail size of actual grade and multiply value by the following adjustment factor: = [1 – (0.5 – SG)], where SG = Specific Gravity of the framing lumber. This adjustment factor shall not be greater than 1.

(b) Fastening along intermediate framing members: Space fasteners a maximum of 12 inches on center, except 6 inches on center for spans greater than 32 inches.

(c) Panels conforming to PS 1 or PS 2.

(d) This table gives shear values for Cases 1 and 2, as shown in IBC Table 2306.3.1. The values shown are applicable to Cases 3, 4, 5 and 6 as shown in IBC Table 2306.3.1, providing fasteners at all continuous panel edges are spaced in accordance with the boundary fastener spacing.

(e) The minimum nominal depth of framing members shall be 3 inches nominal. The minimum nominal width of framing members not located at boundaries or adjoining panel edges shall be 2 inches.

(f) Staples shall have a minimum crown width of 7/16 inch, and shall be installed with their crowns parallel to the long dimension of the framing members.

(g) High load diaphragms shall be subject to special inspection in accordance with IBC Section 1704.6.1.

(h) For shear loads of normal or permanent load duration as defined by the AF&PA NDS, the values in the table above shall be multiplied by 0.63 or 0.56, respectively.

TABLE 2

ALLOWABLE SHEAR (POUNDS PER FOOT) FOR HORIZONTAL APA PANEL DIAPHRAGMS WITH FRAMING OF DOUGLAS-FIR, LARCH OR SOUTHERN PINE[(a)] FOR WIND OR SEISMIC LOADING[(g)] (See also IBC Table 2306.3.1)

					Blocked Diaphragms				Unblocked Diaphragms	
					Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6)[(b)]				Nails Spaced 6" max. at Supported Edges[(b)]	
					6	4	2-1/2[(c)]	2[(c)]		
					Nail Spacing (in.) at other panel edges (Cases 1, 2, 3 & 4)[(b)]					
Panel Grade	Common Nail Size[(f)]	Minimum Nail Penetration in Framing (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Framing Member at Adjoining Panel Edges and Boundaries (in.)	6	6	4	3	Case 1 (No unblocked edges or continuous joints parallel to load)	All other configurations (Cases 2, 3, 4, 5 & 6)
APA STRUCTURAL I grades	6d[(e)] (0.113" dia.)	1-1/4	5/16	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d (0.131" dia.)	1-3/8	3/8	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d[(d)] (0.148" dia.)	1-1/2	15/32	2	320	425	640	730	285	215
				3	360	480	720	820	320	240
APA RATED SHEATHING; APA RATED STURD-I-FLOOR and other APA grades except Species Group 5	6d[(e)] (0.113" dia.)	1-1/4	5/16	2	170	225	335	380	150	110
				3	190	250	380	430	170	125
			3/8	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d (0.131" dia.)	1-3/8	3/8	2	240	320	480	545	215	160
				3	270	360	540	610	240	180
			7/16	2	255	340	505	575	230	170
				3	285	380	570	645	255	190
			15/32	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d[(d)] (0.148" dia.)	1-1/2	15/32	2	290	385	575	655	255	190
				3	325	430	650	735	290	215
			19/32	2	320	425	640	730	285	215
				3	360	480	720	820	320	240

(a) For framing of other species: Find specific gravity for species of lumber in the AF&PA NDS. Find shear value from table above for nail size for actual grade and multiply value by the following adjustment factor: Specific Gravity Adjustment Factor = [1 – (0.5 – SG)], where SG = Specific Gravity of the framing lumber. This adjustment shall not be greater than 1.

(b) Space fasteners maximum 12 inches o.c. along intermediate framing members (6 inches o.c. when supports are spaced 48 inches o.c. or greater).

(c) Framing at adjoining panel edges shall be 3 inch nominal or wider, and nails shall be staggered where nails are spaced 2 inches o.c. or 2-1/2 inches o.c.

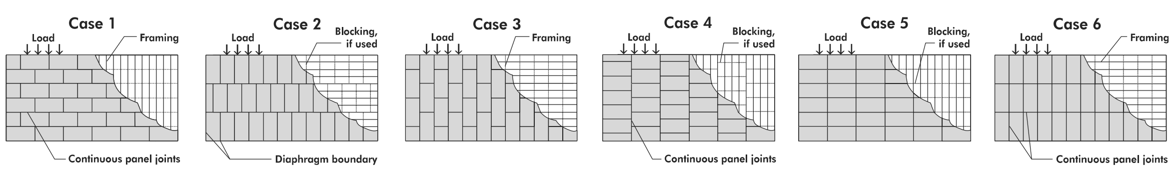
(d) Framing at adjoining panel edges shall be 3 inch nominal or wider, and nails shall be staggered where both of the following conditions are met: (1) 10d nails having penetration into framing of more than 1-1/2 inches and (2) nails are spaced 3 inches o.c. or less.

(e) 8d is recommended minimum for roofs due to negative pressures of high winds.

(f) The minimum nominal width of frraming members not located at boundaries or adjoining panel edges shall be 2 inches

(g) For shear loads of normal or permanent load duration as defined by AF&PA NDS, the values in the table above shall be multiplied by 0.63 and 0.56, respectively.

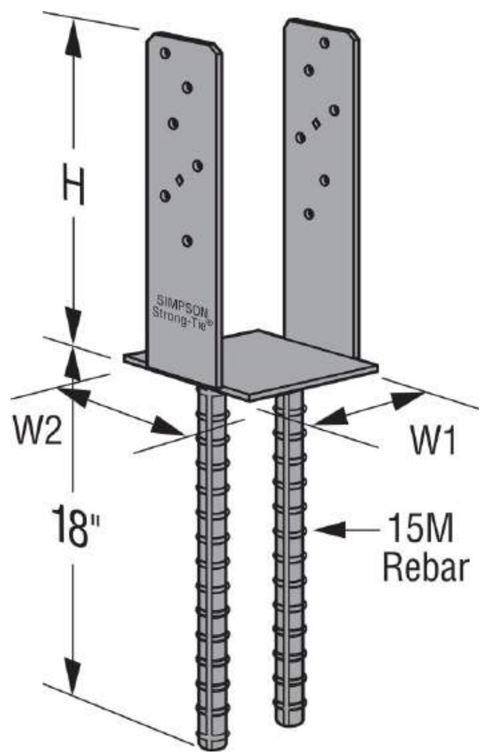
Note: Design for diaphragm stresses depends on direction of continuous panel joints with reference to load, not on direction of long dimension or strength axis of sheet. Continuous framing may be in either direction for blocked diaphragms.



CBQGT Column Base

(https://www.strongtie.com/)

Search strongtie.com



This product is only available in Canada

The CBQGT uses Simpson Strong-Tie® Strong-Drive® SDS Heavy-Duty Connector screws, which allows for fast installation, reduced reveal, high capacity and provides a greater net section area of the column compared to bolts.

Material

- See table

Finish

- Simpson Strong-Tie® gray paint, available in HDG

Installation

- Use all specified fasteners. See General Notes.
- Install 1⁄4"x2" Strong-Drive SDS Heavy-Duty Connector screws, which are provided with the column base. (Lag screws will not achieve the same load.)
- Minimum 3" side cover on concrete is required.
- Post bases do not provide adequate resistance to prevent members from rotating about the base and therefore are not recommended for non top-supported installations (such as fences or unbraced carports).

Options

Other sizes available. Check with Simpson Strong-Tie for details.

Related Links

- Wood Construction Connectors Technical and Installation Notes (/products/connectors/wood-construction-connectors/technical-notes)
- General Notes (/products/connectors/wood-construction-connectors/technical-notes/general-notes)
- Contact Simpson Strong-Tie® (http://www2.strongtie.com/contact_us.asp)

Related Literature

(/resources/literature/wood-construction-connectors-catalog-cn)

Wood Construction Connectors Canadian Limit States Design Catalogue

C-C-CAN2015

Complete product and application information as well as Limit States Design load tables and installation information for over 3,000 wood-to wood, wood-to concrete and wood-to-masonry connectors.

CATALOG

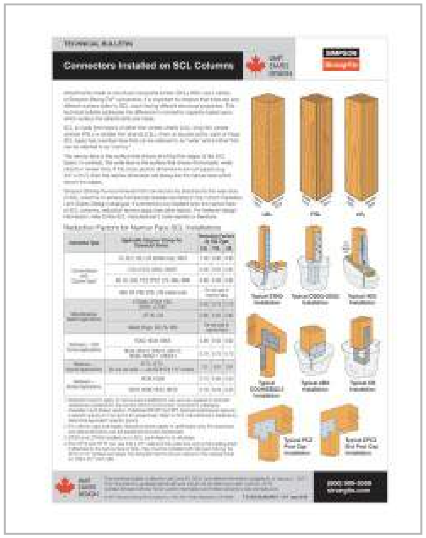
Connecteurs Pour Constructions En Bois Conception Aux États Limites Canadiens

C-C-CAN2015CF

Information complète sur les produits et applications et des tableaux de charge à la conception aux états limites et sur l'installation de 3 000 connecteurs bois à bois, à béton et à maçonnerie.

CATALOG

(https://embed.widencdn.net/download/ssttoolbox/naahpsjpql/C-C-CAN2015CF.pdf?u=cjmyin)


TECHNICAL BULLETIN

Connectors Installed on SCL Columns

Connectors Installed on SCL Columns

T-C-SCLCLMCAN17

A one-page Canadian Limit States version of the technical bulletin discussing the structural properties of two different surface sides of columns made from structural composite lumber (SCL).

TECHNICAL BULLETIN

(https://embed.widencdn.net/download/ssttoolbox/xhbxioydjs/T-C-SCLCLMCAN17.pdf?u=cjmyin)

Load Tables

Model No.	Nominal Post Size	Material		Dimensions (in)			Number of Simpson Strong-Tie 1/4"x2" SDS Screws	Factored Uplift Resistance (K_D = 1.15)	
		Base (Ga)	Strap (Ga x W)	W_1	W_2	H		D.Fir-L	S-P-F
								lbs	lbs
								kN	kN
CBQGT44-SDS2	4x4	7	7 Ga x 2	3 9/16	3 9/16	8 11/16	12	4350	4350
								19.35	19.35
CBQGT46-SDS2	4x6	7	7 Ga x 2	3 9/16	5 1/2	8 11/16	12	4350	4350
								19.35	19.35
CBQGT66-SDS2	6x6	7	7 Ga x 3	5 1/2	5 1/2	8 11/16	12	4350	4350
								19.35	19.35

These products are available with additional corrosion protection (/products/categories/zmax.html). Additional products on this page may also be available with this option, check with Simpson Strong-Tie (/contact_us.asp) for details.

1. Factored uplift resistances have been increased 15% for earthquake or wind loading, with no further increase allowed; reduce where other loads govern.

2. Structural composite lumber columns have sides that show either the wide face or the edges of the lumber strands/ veneers. Values in the tables reflect installation into the wide face.

3. Designer is responsible for concrete design.

4. Factored resistances shown assume dry service condition (K_{SF} = 1.00). Multiply table values by 0.67 under wet service conditions.

5. Minimum f'_c shall be 20 MPa.

Related Products

TABLE 1

DOUGLAS-FIR GLUED LAMINATED BEAM SECTION PROPERTIES AND CAPACITIES
F_b = 2,400 PSI, E = 1,800,000 PSI, F_v = 240 PSI

3-1/8-INCH WIDTH

Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)[1]	4.6	5.7	6.8	8.0	9.1	10.3	11.4	12.5	13.7	14.8	16.0	17.1	18.2	19.4	20.5
A (in.^{2})	18.8	23.4	28.1	32.8	37.5	42.2	46.9	51.6	56.3	60.9	65.6	70.3	75.0	79.7	84.4
S (in.^{3})	19	29	42	57	75	95	117	142	169	198	230	264	300	339	380
I (in.^{4})	56	110	190	301	450	641	879	1170	1519	1931	2412	2966	3600	4318	5126
EI (106 lb-in.^{2})	101	198	342	543	810	1153	1582	2106	2734	3476	4341	5339	6480	7773	9226
Moment Capacity (lb-ft)[2][3]	3750	5859	8438	11484	15000	18984	23438	28359	33750	39609	45938	52734	60000	67734	75938
Shear Capacity (lb)[3]	3000	3750	4500	5250	6000	6750	7500	8250	9000	9750	10500	11250	12000	12750	13500

3-1/2-INCH WIDTH

Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)[1]	5.1	6.4	7.7	8.9	10.2	11.5	12.8	14.0	15.3	16.6	17.9	19.1	20.4	21.7	23.0
A (in.^{2})	21.0	26.3	31.5	36.8	42.0	47.3	52.5	57.8	63.0	68.3	73.5	78.8	84.0	89.3	94.5
S (in.^{3})	21	33	47	64	84	106	131	159	189	222	257	295	336	379	425
I (in.^{4})	63	123	213	338	504	718	984	1310	1701	2163	2701	3322	4032	4836	5741
EI (106 lb-in.^{2})	113	221	383	608	907	1292	1772	2358	3062	3893	4862	5980	7258	8705	10334
Moment Capacity (lb-ft)[2][3]	4200	6563	9450	12863	16800	21263	26250	31763	37800	44363	51450	59063	67200	75863	85050
Shear Capacity (lb)[3]	3360	4200	5040	5880	6720	7560	8400	9240	10080	10920	11760	12600	13440	14280	15120

5-1/8-INCH WIDTH

Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)[1]	14.9	16.8	18.7	20.6	22.4	24.3	26.2	28.0	29.9	31.8	33.6	35.5	37.4	39.2	41.1
A (in.^{2})	61.5	69.2	76.9	84.6	92.3	99.9	107.6	115.3	123.0	130.7	138.4	146.1	153.8	161.4	169.1
S (in.^{3})	123	156	192	233	277	325	377	432	492	555	623	694	769	848	930
I (in.^{4})	738	1051	1441	1919	2491	3167	3955	4865	5904	7082	8406	9887	11531	13349	15348
EI (106 lb-in.^{2})	1328	1891	2595	3453	4483	5700	7119	8757	10627	12747	15131	17796	20756	24028	27627
Moment Capacity (lb-ft)[2][3]	24600	31134	38438	46509	55350	64959	75338	86484	98400	111084	124538	138759	153750	169509	186038
Shear Capacity (lb)[3]	9840	11070	12300	13530	14760	15990	17220	18450	19680	20910	22140	23370	24600	25830	27060

5-1/2-INCH WIDTH

Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)[1]	16.0	18.0	20.1	22.1	24.1	26.1	28.1	30.1	32.1	34.1	36.1	38.1	40.1	42.1	44.1
A (in.^{2})	66.0	74.3	82.5	90.8	99.0	107.3	115.5	123.8	132.0	140.3	148.5	156.8	165.0	173.3	181.5
S (in.^{3})	132	167	206	250	297	349	404	464	528	596	668	745	825	910	998
I (in.^{4})	792	1128	1547	2059	2673	3398	4245	5221	6336	7600	9021	10610	12375	14326	16471
EI (106 lb-in.^{2})	1426	2030	2784	3706	4811	6117	7640	9397	11405	13680	16238	19098	22275	25786	29648
Moment Capacity (lb-ft)[2][3]	26400	33413	41250	49913	59400	69713	80850	92813	105600	119213	133650	148913	165000	181913	199650
Shear Capacity (lb)[3]	10560	11880	13200	14520	15840	17160	18480	19800	21120	22440	23760	25080	26400	27720	29040

6-3/4-INCH WIDTH

Depth (in.)	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39
Beam Weight (lb/ft)[1]	29.5	32.0	34.5	36.9	39.4	41.8	44.3	46.8	49.2	51.7	54.1	56.6	59.1	61.5	64.0
A (in.^{2})	121.5	131.6	141.8	151.9	162.0	172.1	182.3	192.4	202.5	212.6	222.8	232.9	243.0	253.1	263.3
S (in.^{3})	365	428	496	570	648	732	820	914	1013	1116	1225	1339	1458	1582	1711
I (in.^{4})	3281	4171	5209	6407	7776	9327	11072	13021	15188	17581	20215	23098	26244	29663	33367
EI (106 lb-in.^{2})	5905	7508	9377	11533	13997	16789	19929	23438	27338	31647	36386	41577	47239	53394	60060
Moment Capacity (lb-ft)[2][3]	72900	85556	99225	113906	129600	146306	164025	182756	202500	223256	245025	267806	291600	316406	342225
Shear Capacity (lb)[3]	19440	21060	22680	24300	25920	27540	29160	30780	32400	34020	35640	37260	38880	40500	42120

8-3/4-INCH WIDTH

Depth (in.)	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39	40-1/2	42	43-1/2	45
Beam Weight (lb/ft)[1]	51.0	54.2	57.4	60.6	63.8	67.0	70.2	73.4	76.6	79.8	82.9	86.1	89.3	92.5	95.7
A (in.^{2})	210.0	223.1	236.3	249.4	262.5	275.6	288.8	301.9	315.0	328.1	341.3	354.4	367.5	380.6	393.8
S (in.^{3})	840	948	1063	1185	1313	1447	1588	1736	1890	2051	2218	2392	2573	2760	2953
I (in.^{4})	10080	12091	14352	16880	19688	22791	26204	29942	34020	38452	43253	48439	54023	60020	66445
EI (106 lb-in.^{2})	18144	21763	25834	30383	35438	41023	47167	53896	61236	69214	77856	87190	97241	108036	119602
Moment Capacity (lb-ft)[2][3]	168000	189656	212625	236906	262500	289406	317625	347156	378000	410156	443625	478406	514500	551906	590625
Shear Capacity (lb)[3]	33600	35700	37800	39900	42000	44100	46200	48300	50400	52500	54600	56700	58800	60900	63000

Notes:
(1) Beam weight is based on density of 35 pcf.
(2) Moment capacity must be adjusted for volume effect. The volume factor for various glulam sizes and simple spans, as well as the complete formula, is given in Appendix A.
(3) Moment and shear capacities are based on a normal (10 years) duration of load and should be adjusted for the design duration of load per the applicable building code.

Load Table: See code report listings below

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These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

These models are approved for installation with the Strong-Drive SD Structural-Connector screw.

Model No.	Total L	Ga	DF/SP		SPF/HF		Allowable Tension Loads (160)
			Fasteners	End Length	Fasteners	End Length	
			160	160	160	160	
CMST12	40'	12	74 - 16d	33"	84 - 16d	38"	9215
			86 - 10d	39"	98 - 10d	44"	9215
CMST14	52½'	14	56 - 16d	26"	66 - 16d	30"	6490
			66 - 10d	30"	76 - 10d	34"	6490
CMSTC16	54'	16	50 - 16d sinker	20"	58 - 16d sinker	25"	4585
CS14	100'	14	26 - 10d	15"	30 - 10d	16"	2490
			30 - 8d	16"	36 - 8d	19"	2490
CS16	150'	16	20-10d	11"	22 - 10d	12"	1705
			22 - 8d	13"	26 - 8d	14"	1705
CS18	200'	18	16 - 10d	9"	18 - 10d	10"	1370
			18 - 8d	11"	22 - 8d	12"	1370
CS20	250'	20	12 - 10d	6"	14 - 10d	8"	1030
			14 - 8d	9"	16 - 8d	9"	1030
CS22	300'	22	10 - 10d	7"	12 - 10d	7"	845
			12 - 8d	6"	14 - 8d	8"	845

1. Loads include a 60% load duration increase on the fasteners for wind or seismic loading.
2. Use half of the required nails in each member being connected to achieve the listed loads.
3. Calculate the connector value for a reduced number of nails as follows:

   $$\text{Allowable Load} = \frac{\text{No. of Nails Used}}{\text{No. of Nails in Table}} \times \text{Table Load}$$

   Example: CMSTC16 in DF/SP with 40 nails total.
   (Half of the nails in each member being connected)

   $$\text{Allowable Load} = \frac{\text{40 Nails (Used)}}{\text{50 Nails (Table)}} \times 4585 \text{ lbs} = 3668 \text{ lbs}$$

4. Tension loads apply for uplift when installed vertically.
5. NAILS: 16d = 0.162" dia. x 3 1/2" long, 16d Sinker = 0.148" dia. x 3 1/4" long, 10d = 0.148" dia. x 3 " long. See other nail sizes and information.

Code Reports (PDFs):

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	IAPMO UES ER	ICC-ES ESR	CITY OF LOS ANGELES	STATE OF FLORIDA	LEGACY REPORTS		
					ICC-ES NER	ICC-ES ER	ICC-ES ES
CMST	See specific model numbers for code listings.						
CMST12	ER-124	ESR-2105 / ESR-2523 *	RR25713 / RR25489	FL10852 / FL13872			
CMST14	ER-124	ESR-2105 / ESR-2523 *	RR25713 / RR25489	FL10852 / FL13872			
CMSTC	No code listing. Please contact us for test data.						
CMSTC16	ER-124	ESR-2105 / ESR-2523 *	RR25713 / RR25489	FL10852 / FL13872			
CS	See specific model numbers for code listings.						
CS14	ER-124	ESR-2105 / ESR-2523 *	RR25713 / RR25489	FL10852 / FL13872			
CS14-R	ER-124			FL10852			
CS16	ER-124	ESR-2105 / ESR-2523 *	RR25713 / RR25489	FL10852 / FL13872			
CS16-R	ER-124	ESR-2105 / ESR-2523 *	RR25489	FL10852			
CS16Z	ER-124	ESR-2105 / ESR-2523 *	RR25713 / RR25489	FL10852			
CS18	ER-124	ESR-2105 / ESR-2523 *	RR25713 / RR25489	FL10852 / FL13872			
CS18-R	ER-124	ESR-2105 / ESR-2523 *	RR25489	FL10852			
CS18S	ER-124	ESR-2105 / ESR-2523 *		FL10852			
CS20	ER-124	ESR-2105 / ESR-2523 *	RR25713 / RR25489	FL10852 / FL13872			
CS20-R	ER-124	ESR-2105 / ESR-2523 *	RR25489	FL10852			

(/resources/literature/high-wind-product-guide)

High Wind-Resistant Construction Application Guide

F-C-HWRCAG16

The newly updated High Wind-Resistant Construction Application Guide offers new applications, updated loads, as well as more fastening information that is used to resist high-wind.

PRODUCT GUIDE

Show 7 More

Load Tables

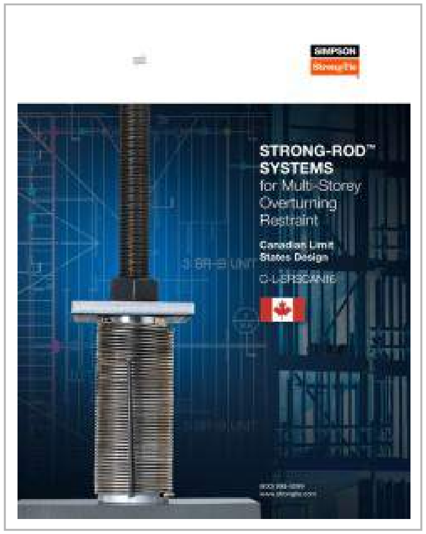
These products are available with additional corrosion protection (/products/connectors/wood-construction-connectors/technical-notes/corrosion-info/corrosion-resistant-wcc). Additional products on this page may also be available with this option, check with Simpson Strong-Tie (/contact_us.asp) for details.

Model No.	Material		Dimensions (in.)							Fasteners		Minimum Wood Member Thickness (in.)	Allowable Tension Loads (160)		Deflection at Highest Allowable Load
	Base (in.)	Body (ga.)	HB	SB	W	H	B	℄	SO	Anchor Dia.	Stud Bolts		DF/SP	SPF/HF	
HD3B	—	12	4¾	2½	2½	8⅝	2¼	1⁵⁄₁₆	⅜	⅝	(2) ⅝	1½	1,895	1,610	0.156
												2½	2,525	2,145	0.169
												3	3,130	3,050	0.120
												3½	3,130	3,050	0.120
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1½	2,405	2,070	0.153
												2½	3,750	3,190	0.129

HD5B	3⁄16	10	5¼	3	2½	9⅜	2½	1¼	2	⅝	(2) ¾	3	4,505	3,785	0.156
												3½	4,935	4,195	0.150
HD7B	3⁄16	10	5¼	3	2½	12⅜	2½	1¼	2	⅞	(3) ¾	3	6,645	5,650	0.142
												3½	7,310	6,215	0.154
												4½	7,345	6,245	0.155
HD9B	⅜	7	6⅛	3½	2⅞	14	2½	1¼	2⅜	⅞	(3) ⅞	3½	7,740	6,580	0.159
												4½	9,920	8,435	0.178
												5½	9,920	8,430	0.178
												7¼	10,035	8,530	0.179
HD12	⅜	3	7	4	3½	20 5⁄16	4¼	2⅛	3⅝	1	(4) 1	3½	11,350	9,215	0.171
												4½	12,665	10,765	0.171
												5½ x 5½	14,220	12,085	0.162
										1⅛	(4) 1	3½	11,775	9,215	0.171
												4½	13,335	11,055	0.177
												7¼	15,435	13,120	0.194
												5½ x 5½	15,510	12,690	0.162
HD19	⅜	3	7	4	3½	24½	4¼	2⅛	3⅝	1⅛	(5) 1	7¼	16,735	14,225	0.191
												5½ x 5½	16,775	12,690	0.200
										1¼	(5) 1	7¼	19,360	15,270	0.180
												5½ x 5½	19,070	16,210	0.137

1. To achieve published loads, machine bolts shall be installed with the nut on the opposite side of the holdown. If reversed, the Designer shall reduce the allowable loads shown per NDS requirements when bolt threads are in the shear plane.
2. Lag bolts will not develop the listed loads.
3. HD19 with 1 1/4" anchor rod requires No. 1 or better post to achieve published loads.

Code Reports



Strong-Rod™ Systems for Multi-Storey Overturning Restraint

C-L-SRSCAN16

A Canadian Limit States Design Catalogue for Strong-Rod™ Anchor Tiedown System. Includes product information, design methods, and pre-engineered runs using shrinkage compensation and other components.

CATALOG

(https://embed.widencdn.net/download/ssttoolbox/cdgpj94jlk/C-L-SRSCAN16.pdf?u=cjmyin)

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Load Tables

These products are available with additional corrosion protection (/products/connectors/wood-construction-connectors/technical-notes/corrosion-info/corrosion-resistant-wcc). Additional products on this page may also be available with this option, check with Simpson Strong-Tie (https://www2.strongtie.com/contact_us.asp) for details.

These models are approved for installation with the Strong-Drive® SD Connector screw (/strongdrive_exteriorwoodscrews/sd_screw/p/strong-drive-sd-connector-screw). See the load values below.

Model No.	Ga.	Dimensions (in.)		Fasteners (Total)	Allowable Tension Loads (DF/SP)	Allowable Tension Loads (SPF/HF)
		W	L		(160)	(160)
LSTA9		1¼	9	(8) 10d	740	635
LSTA12		1¼	12	(10) 10d	925	795
LSTA15		1¼	15	(12) 10d	1,110	950
LSTA18		1¼	18	(14) 10d	1,235	1,110
LSTA21		1¼	21	(16) 10d	1,235	1,235

	LSTA24	20	1¼	24	(18) 10d	1,235	1,235
	ST292		2 1/16	9 5/16	(12) 16d	1,265	1,120
	ST2122		2 1/16	12 13/16	(16) 16d	1,530	1,505
	ST2115		¾	16 5/16	(10) 16d	660	660
	ST2215		2 1/16	16 5/16	(20) 16d	1,875	1,875
	LSTA30	18	1¼	30	(22) 10d	1,640	1,640
	LSTA36		1¼	36	(24) 10d	1,640	1,640
	LSTI49		3¾	49	(32) 10d x 1½"	2,975	2,555
	LSTI73		3¾	73	(48) 10d x 1½"	4,205	3,830
	MSTA9		1¼	9	(8) 10d	750	645
SS	MSTA12		1¼	12	(10) 10d	940	810
	MSTA15		1¼	15	(12) 10d	1,130	970
SS	MSTA18		1¼	18	(14) 10d	1,315	1,130
	MSTA21		1¼	21	(16) 10d	1,505	1,290
SS	MSTA24		1¼	24	(18) 10d	1,640	1,455
	MSTA30	16	1¼	30	(22) 10d	2,050	1,820
SS	MSTA36		1¼	36	(26) 10d	2,050	2,050
	MSTA49		1¼	49	(26) 10d	2,020	2,020
	ST6215		2 1/16	16 5/16	(20) 16d	2,095	1,900
	ST6224		2 1/16	23 5/16	(28) 16d	2,540	2,540
	ST9		1¼	9	(8) 16d	885	760
	ST12		1¼	11 5/8	(10) 16d	1,105	950
	ST18		1¼	17¾	(14) 16d	1,420	1,330
	ST22		1¼	21 5/8	(18) 16d	1,420	1,420
	MSTC28		3	28¼	(36) 16d sinkers	3,455	2,980
	MSTC40		3	40¼	(52) 16d sinkers	4,745	4,305
	MSTC52		3	52¼	(62) 16d sinkers	4,745	4,745
	HTP37Z		3	7	(20) 10d x 1½"	1,850	1,600

MSTC66	14	3	65 3/4	(76) 16d sinkers	5,860	5,860
MSTC78		3	77 3/4	(76) 16d sinkers	5,860	5,860
ST6236		2 1/16	33 13/16	(40) 16d	3,845	3,845
HRS6	12	1 3/8	6	(6) 10d	605	525
HRS8		1 3/8	8	(10) 10d	1,010	880
HRS12		1 3/8	12	(14) 10d	1,415	1,230
MSTI26		2 1/16	26	(26) 10d x 1½"	2,745	2,325
MSTI36		2 1/16	36	(36) 10d x 1½"	3,800	3,220
MSTI48		2 1/16	48	(48) 10d x 1½"	5,065	4,290
MSTI60		2 1/16	60	(60) 10d x 1½"	5,080	5,080
MSTI72		2 1/16	72	(72) 10d x 1½"	5,080	5,080
HRS416Z		3 1/4	16	(16) ¼" x 1½" SDS	2,835	2,305

1. Allowable loads have been increased for wind or seismic loading with no further increase allowed; reduce where other loads govern.
2. See Fastener Designer Information (/products/connectors/wood-construction-connectors/technical-notes/fastener-types-and-sizes) for allowable nail substitutions and load reductions. When nailing strap over wood structural panels, use 2 1/2" long fastener, minimum.
3. Use half of the nails in each member being connected to achieve the listed loads.
4. Tension loads apply for uplift when installed vertically.
5. **NAILS**: 16d = 0.162" dia. x 3 1/2" long, 16d Sinker = 0.148" dia. x 3 1/4" long, 10d = 0.148" dia. x 3" long; 10d x 1 1/2 = 0.148" dia. x 1 1/2" long. See other nail sizes and information (/products/connectors/wood-construction-connectors/technical-notes/fastener-types-and-sizes).

Floor-to-Floor Clear Span Table

These products are available with additional corrosion protection (/products/connectors/wood-construction-connectors/technical-notes/corrosion-info/corrosion-resistant-wcc). Additional products on this page may also be available with this option, check with Simpson Strong-Tie (https://www2.strongtie.com/contact_us.asp) for details.

These models are approved for installation with the Strong-Drive® SD Connector screw (/strongdrive_exteriorwoodscrews/sd_screw/p/strong-driv -screw). See the load values below.

(https://www.strongtie.com/)

Search strongtie.com

Model No.	Clear Span (in.)	Fasteners (Total)	Allowable Tension Loads (DF/SP)	Allowable Tension Loads (SPF/HF)
			(160)	(160)
MSTA49	18	(26) 10d	2,020	2,020
	16	(26) 10d	2,020	2,020
MSTC28	18	(12) 16d sinkers	1,155	995
	16	(16) 16d sinkers	1,540	1,325
MSTC40	24	(20) 16d sinkers	2,310	1,985
	18	(28) 16d sinkers	2,695	2,320
	16	(32) 16d sinkers	3,080	2,650
MSTC52	24	(36) 16d sinkers	3,465	2,980
	18	(44) 16d sinkers	4,235	3,645
	16	(48) 16d sinkers	4,620	3,975
MSTC66	30	(48) 16d sinkers	4,780	4,120
	24	(54) 16d sinkers	5,380	4,640
	18	(64) 16d sinkers	5,860	5,495
	16	(68) 16d sinkers	5,860	5,840
MSTC78	30	(64) 16d sinkers	5,860	5,495
	24	(72) 16d sinkers	5,860	5,860
	18	(76) 16d sinkers	5,860	5,860
MST37	24	(14) 16d	1,725	1,495
	18	(20) 16d	2,465	2,135
	16	(22) 16d	2,710	2,345
MST48	24	(26) 16d	3,215	2,780
	18	(32) 16d	3,960	3,425
	16	(34) 16d	4,205	3,640
MST60	30	(34) 16d	4,605	3,995
	24	(40) 16d	5,240	4,700
	18	(46) 16d	6,235	5,405
MST72	30	(48) 16d	6,505	5,640
	24	(54) 16d	6,730	6,345
	18	(62) 16d	6,730	6,475

See footnotes below.

Model No.	Ga.	Dimensions (in.)		Fasteners (Total)			Allowable Tension Loads (DF/SP)		Allowable Tension Loads (SPF/HF)	
		W	L	Nails	Bolts		Nails	Bolts	Nails	Bolts
					Qty.	Dia.	(160)	(160)	(160)	(160)
MST27	12	2 1/16	27	(30) 16d	4	½	3,700	2,165	3,200	2,000
MST37		2 1/16	37½	(42) 16d	6	½	5,080	3,025	4,480	2,805
MST48		2 1/16	48	(50) 16d	8	½	5,310	3,675	5,190	3,410
MST60	10	2 1/16	60	(68) 16d	10	½	6,730	4,485	6,475	4,175
MST72		2 1/16	72	(68) 16d	10	½	6,730	4,485	6,475	4,175
HST2	7	2½	21¼	—	6	⅝	—	5,220	—	4,835
HST5		5	21¼	—	12	⅝	—	10,650	—	9,870
HST3	3	3	25½	—	6	¾	—	7,680	—	6,660
HST6		6	25½	—	12	¾	—	15,470	—	13,320

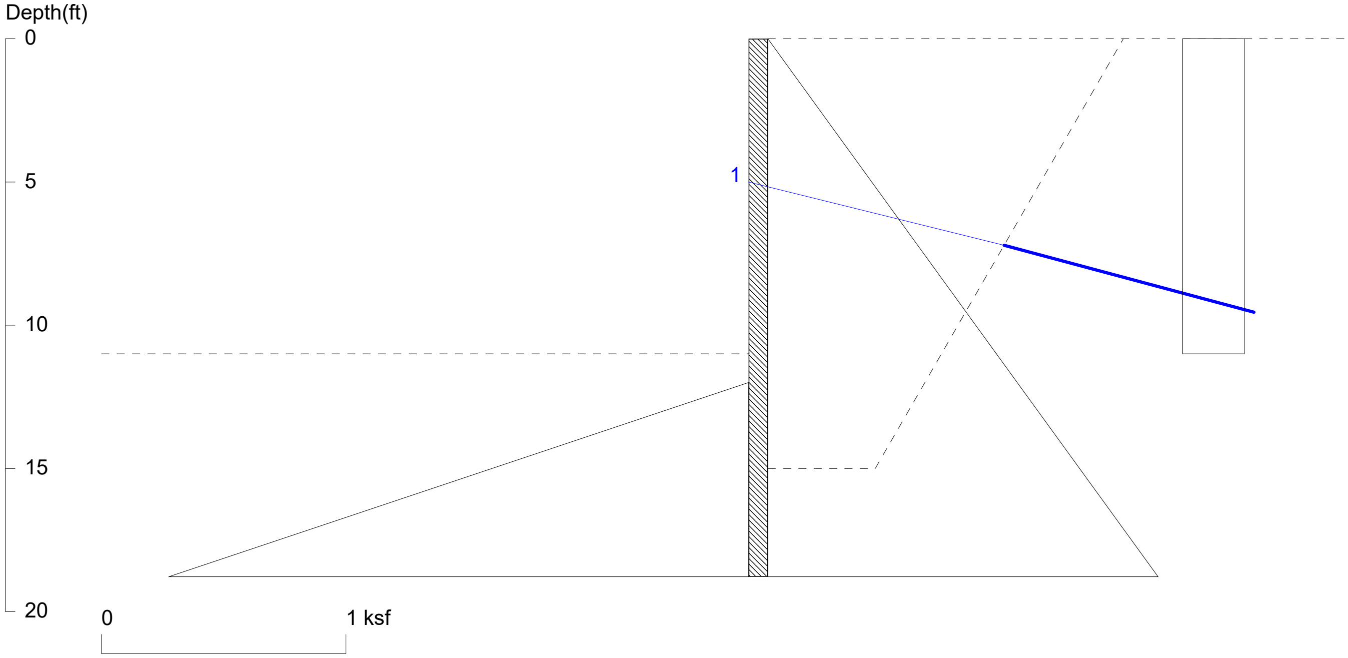
1. Allowable loads have been increased for wind or seismic loading with no further increase allowed; reduce where other loads govern.
2. Install bolts or nails as specified by Designer. Bolt and nail values may not be combined.
3. Allowable bolt loads are based on parallel-to-grain loading and these minimum member thicknesses: MST - 2 1/2"; HST2 and HST5 - 4"; HST3 and HST6 - 4 1/2.
4. Splitting may be a problem with installations on lumber smaller than 3 1/2"; either fill every nail hole with 10d x 1 1/2" nails or fill every-other hole with 16d common nails. Reduce the allowable load based upon the size and quantity of fasteners used.
5. Use half of the required nails in each member being connected to achieve the listed loads.
6. When installing strap over wood structural panel sheathing, use 2 1/2" long nail minimum.
7. Tension loads apply for uplift as well when installed vertically.
8. **NAILS**: 16d = 0.162" dia. x 3 1/2" long, 16d Sinker = 0.148" dia. x 3 1/4" long, 10dx1 1/2 = 0.148" dia. x 1 1/2" long. See other nail sizes and information (/products/connectors/wood-construction-connectors/technical-notes/fastener-types-and-sizes).

Load Values with Strong-Drive® SD Connector Screws

Model No.	Fasteners (Total)	Allowable Tension Loads (DF/SP)	Allowable Tension Loads (SPF/HF)
		(160)	(160)
LSTA9	8-SD9112	1095	715
LSTA12	10-SD9112	1235	895
LSTA15	12-SD9112	1235	1075
LSTA18	14-SD9112	1235	1235
LSTA21	14-SD9112	1235	1215

Dufrense Residence

11' - Permanent Soldier Pile - Siesmic Loading



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File: C:\Lucia Engineering\Lucia Engineering 2017\Misc. Small Projects\Margaret Dufresne\11' - Rear Wall - With Siesmic L

Wall Height=11.0 Pile Diameter=2.5 Pile Spacing=8.0 Wall Type: 2. Soldier Pile, Drilled

PILE LENGTH: Min. Embedment=7.78 (8~10ft is recommended!!!) Min. Pile Length=18.78 (in graphics and analysis)
MOMENT IN PILE: Max. Moment=111.17 per Pile Spacing=8.0 at Depth=11.09

PILE SELECTION:
Request Min. Section Modulus = 40.4 in3/pile=662.48 cm3/pile, Fy= 50 ksi = 345 MPa, Fb/Fy=0.66
-> Piles meet Min. Section Requirements: Top Deflection is shown in (in)
W8X48 (-0.53) HP10X42 (-0.47) W10X39 (-0.47) HP12X53 (-0.25) W12X35 (-0.34)
HP13X60 (-0.19) HP14X73 (-0.13) W14X30 (-0.34) W16X31 (-0.26) HP16X88 (-0.09)
W16X89 (-0.08) HP16X101 (-0.08) W16X100 (-0.07) HP16X121 (-0.06)

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

No. & Type	Depth	Angle	Space	Total F.	Horiz. F.	Vert. F.	L_free	Fixed Length
1. Tieback	5.0	15.0	6.0	49.8	48.1	12.9	8.5	21.1

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	0	11	0.935	0.085000
11	0.935	50	4.25	0.085000
*	Siesm	23H		
0	0.253	11	0.253	0.000000

PASSIVE PRESSURES:

Z1	P1	Z2	P2	Slope
12	0	19	2.45	0.3500

ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	8.00
2	15.00	2.50

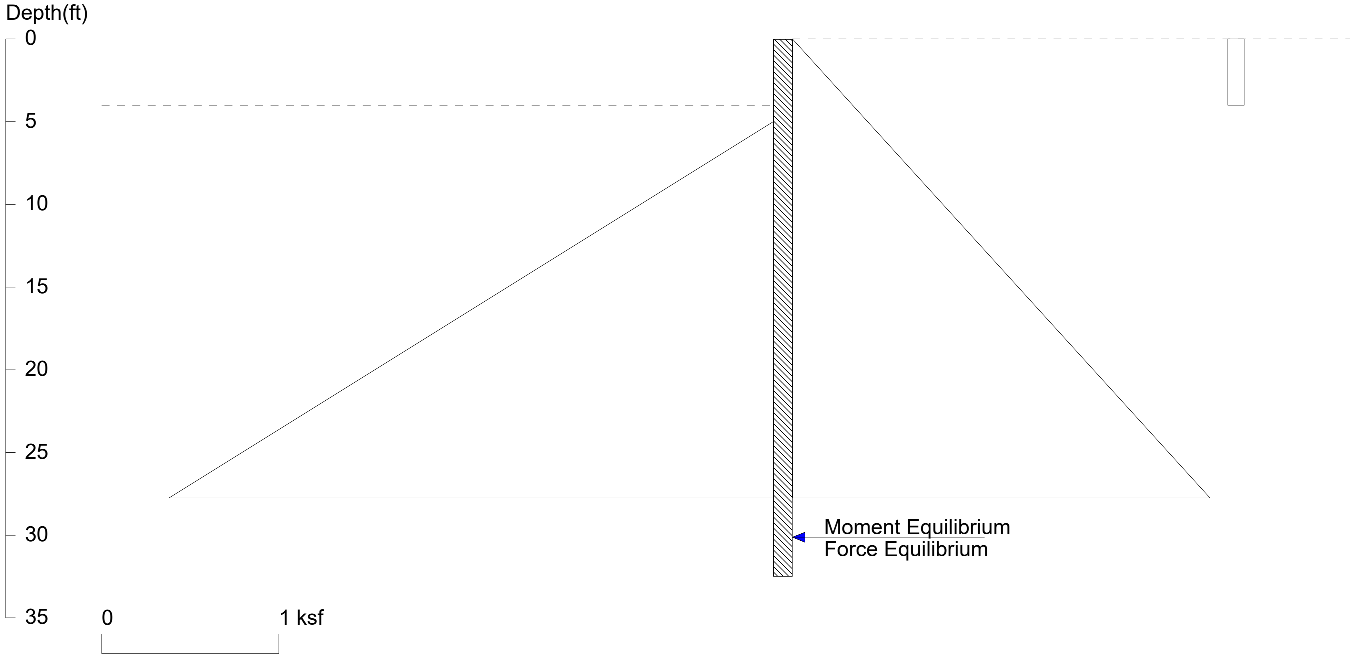
PASSIVE SPACING:

No.	Z depth	Spacing
1	12.00	6.00

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

Dufrense Residence

4' - Permanent Soldier Pile - Siesmic Loading



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File: C:\Lucia Engineering\Lucia Engineering 2017\Misc. Small Projects\Margaret Dufresne\4' - Rear Wall - With Siesmic Lo

Wall Height=4.0 Pile Diameter=2.5 Pile Spacing=8.0 Wall Type: 2. Soldier Pile, Drilled

PILE LENGTH: Min. Embedment=28.50 Min. Pile Length=32.50 (in graphics and analysis)
MOMENT IN PILE: Max. Moment=356.57 per Pile Spacing=8.0 at Depth=19.64

PILE SELECTION:
Request Min. Section Modulus = 129.7 in3/pile=2124.80 cm3/pile, Fy= 50 ksi = 345 MPa, Fb/Fy=0.66
-> Piles meet Min. Section Requirements: Top Deflection is shown in (in)
W12X96 (0.66) HP13X100 (0.62) HP14X89 (0.60) W14X90 (0.55) W16X77 (0.49)
HP16X88 (0.49) W16X89 (0.42) HP16X101 (0.42) W16X100 (0.37) HP16X121 (0.35)
W18X76 (0.41) HP18X135 (0.25) W18X130 (0.22) HP18X157 (0.21)

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

Z1	P1	Z2	P2	Slope
0	0	4	0.34	0.085000
4	0.34	50	4.25	0.085000
*	Siesm	23H		
0	0.092	4	0.092	0.000000

PASSIVE PRESSURES:

Z1	P1	Z2	P2	Slope
5	0	50	6.75	0.1500

ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	8.00
2	15.00	2.50

PASSIVE SPACING:

No.	Z depth	Spacing
1	5.00	6.00

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

report.out

```
******************************************************************************

                      SHORING WALL CALCULATION SUMMARY
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                   Software Copyright by CivilTech Software
                              www.civiltech.com

******************************************************************************
ShoringSuite Software is developed by CivilTech Software, Bellevue, WA, USA.
The calculation method is based on the following references:
    1. FHWA 98-011, FHWA-RD-97-130, FHWA SA 96-069, FHWA-IF-99-015
    2. STEEL SHEET PILING DESIGN MANUAL by Pile Buck Inc., 1987
    3. DESIGN MANUAL DM-7 (NAVFAC), Department of the Navy, May 1982
    4. TRENCHING AND SHORING MANUAL Revision 12, California Department of
       Transportation, January 2000
    6. EARTH SUPPORT SYSTEM & RETAINING STRUCTURES, Pile Buck Inc. 2002
    5. DESIGN OF SHEET PILE WALLS, EM 1110-2-2504, U.S. Army Corps of Engineers, 31
March 1994
    7. EARTH RETENTION SYSTEMS HANDBOOK, Alan Macnab, McGraw-Hill. 2002
    8. AASHTO HB-17, American Association of State and Highway Transportation
Officials, 2 September 2002

UNITS:   Width/Spacing/Diameter/Length/Depth - ft, Force - kip, Moment - kip-ft,
Friction/Bearing/Pressure - ksf, Pres. Slope - kip/ft3, Deflection - in
-----------------------------------------------------------------------------
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Projects\Margaret Dufresne\4' - Rear Wall - With Siesmic Loading.sh8

Title: Dufrense Residence
Subtitle: 4' - Permanent Soldier Pile - Siesmic Loading

**********************************INPUT DATA**********************************
Wall Type: 2. Soldier Pile, Drilled
        Wall Height: 4.00
        Pile Diameter: 2.50
        Pile Spacing: 8.00
        Factor of Safety (F.S.): 1.00
Lateral Support Type (Braces): 1. No
        Top Brace Increase (Multi-Bracing): Add 15%*
Embedment Option: 1. Yes
        Friction at Pile Tip: No
Pile Properties:
        Steel Strength, Fy:  50 ksi = 345 MPa
        Allowable Fb/Fy: 0.66
        Elastic Module, E: 29000.00
        Moment of Inertia, I: 833
        User Input Pile: W18X65
```

report.out

```
* DRIVING PRESSURE (ACTIVE, WATER, & SURCHARGE) *
 No.    Z1 top          Top Pres.       Z2 bottom       Bottom Pres.    Slope
-----------------------------------------------------------------------------
 1      0               0               4               0.34            0.085000
 2      4               0.34            50              4.25            0.085000
 3      *               Siesm           23H
 4      0               0.092           4               0.092           0.000000
-----------------------------------------------------------------------------

* PASSIVE PRESSURE *
 No.    Z1 top          Top Pres.       Z2 bottom       Bottom Pres.    Slope
-----------------------------------------------------------------------------
 1      5               0               50              6.75            0.1500
-----------------------------------------------------------------------------

* ACTIVE SPACE *
 No.    Z depth         Spacing
-----------------------------------------------------------------------------
 1      0.00            8.00
 2      15.00           2.50
-----------------------------------------------------------------------------

* PASSIVE SPACE *
 No.    Z depth         Spacing
-----------------------------------------------------------------------------
 1      5.00            6.00
-----------------------------------------------------------------------------
*For Tieback:  Input1 = Diameter;  Input2 = Bond Strength
*For Plate:  Input1 = Diameter;  Input2 = Allowable Pressure
*For Deadman:  Input1 = Horz. Width;  Input2 = Passive Pressure;
*For Sheet Pile Anchor:  Input1 = Horz. Width;  Input2 = Passive Slope;


********************************CALCULATION**********************************

   The calculated moment and shear are per pile spacing.  Sheet piles are per one
foot or meter; Soldier piles are per pile.

   Top Pressures start at depth =  0.00


        |      D1=0.00
        |
        |
        |
      ==|==    D2=4.00
        |
```

```
       |
       |      D3=32.50

       D1 - TOP DEPTH
       D2 - EXCAVATION BASE
       D3 - PILE TIP (20% increased, see EMBEDMENT Notes below)

   MOMENT BALANCE: M=0.00   AT DEPTH=27.75  WITH EMBEDMENT OF 23.75
   FORCE BALANCE: F=0.00   AT DEPTH=32.50  WITH EMBEDMENT OF 28.50
```

The program calculates an embedment for moment equilibrium, then increase the embedment by 20% to reach force equilibrium.
A Balance Force=95.70 is developed from depth=27.75 to depth=32.50
Total Passive Pressure = Total Active Pressure,   OK!

*********************************RESULTS***

* EMBEDMENT Notes *
Based on USS Design Manual, first calculate embedment for moment equilibrium, then increased by 20 to 40 % to get the total design depth.
The embedment for moment equilibrium is 23.75
* The 20% increased the total design depth is 28.50 (Used by Program)
The 30% increased the total design depth is 30.87
The 40% increased the total design depth is 33.25

Based on AASHTO 2002 Standard Specifications, first calculate embedment for moment equilibrium, then add safety factor of 30% for temporary shoring; add safety factor of 50% for permanent shoring.
The embedment for moment equilibrium is 23.75
Add 30% embedment for temporary shoring is 30.87
Add 50% embedment for permanent shoring is 35.62

* BASED ON USS DESIGN MANUAL (20% increased), PROGRAM CALCULATED MINIMUM EMBEDMENT = 28.50
TOTAL MINIMUM PILE LENGTH = 32.50

* MOMENT IN PILE (per pile spacing)*
Pile Spacing: sheet piles are one foot or one meter; soldier piles are one pile.
Overall Maximum Moment = 356.57 at 19.64
Maximum Shear = 95.16
Moment and Shear are per pile spacing: 8.0 foot or meter

* VERTICAL LOADING *
Vertical Loading from Braces = 0.00
Vertical Loading from External Load = 0.00
Total Vertical Loading = 0.00

```
***************************SOLDIER PILE SELECTION***************************

   Request Min. Section Modulus = 129.66 in3/pile = 2124.80 cm3/pile, Fy= 50 ksi =
345 MPa, Fb/Fy=0.66
   The pile selection is based on the magnitude of the moment only. Axial force is
neglected.


   W12X96
   (English Units):
   Area= 28.2 in.   Depth= 12.7 in.  Width= 12.2 in.  Height= 12 in.
   Flange thickness= 0.9 in.   Web thickness= 0.55 in.
   Ix= 833 in4/pile   Sx= 131 in3/pile   Iy= 270 in4/pile   Sy= 44.4 in3/pile
   (Metric Units):
   Ix= 346.69 x100cm4/pile   Sx= 2146.70 cm3/pile   Iy= 112.37 x100cm4/pile   Sy=
727.58 cm3/pile
       Top deflection = 0.656(in)
   HP13X100
   (English Units):
   Area= 29.4 in.   Depth= 13.15 in.  Width= 13.205 in.  Height= 13 in.
   Flange thickness= 0.765 in.   Web thickness= 0.765 in.
   Ix= 886 in4/pile   Sx= 135 in3/pile   Iy= 294 in4/pile   Sy= 44.5 in3/pile
   (Metric Units):
   Ix= 368.75 x100cm4/pile   Sx= 2212.25 cm3/pile   Iy= 122.36 x100cm4/pile   Sy=
729.22 cm3/pile
       Top deflection = 0.617(in)
   HP14X89
   (English Units):
   Area= 26.1 in.   Depth= 13.8 in.  Width= 14.7 in.  Height= 14 in.
   Flange thickness= 0.615 in.   Web thickness= 0.615 in.
   Ix= 904 in4/pile   Sx= 131 in3/pile   Iy= 326 in4/pile   Sy= 44.3 in3/pile
   (Metric Units):
   Ix= 376.24 x100cm4/pile   Sx= 2146.70 cm3/pile   Iy= 135.68 x100cm4/pile   Sy=
725.94 cm3/pile
       Top deflection = 0.605(in)
   W14X90
   (English Units):
   Area= 26.5 in.   Depth= 14 in.  Width= 14.5 in.  Height= 14 in.
   Flange thickness= 0.71 in.   Web thickness= 0.44 in.
   Ix= 999 in4/pile   Sx= 143 in3/pile   Iy= 362 in4/pile   Sy= 49.9 in3/pile
   (Metric Units):
   Ix= 415.78 x100cm4/pile   Sx= 2343.34 cm3/pile   Iy= 150.66 x100cm4/pile   Sy=
817.71 cm3/pile
       Top deflection = 0.547(in)
   W16X77
   (English Units):
   Area= 22.6 in.   Depth= 16.5 in.  Width= 10.3 in.  Height= 16 in.
```

```
   Flange thickness= 0.76 in.   Web thickness= 0.455 in.
   Ix= 1110 in4/pile   Sx= 134 in3/pile   Iy= 138 in4/pile   Sy= 26.9 in3/pile
   (Metric Units):
   Ix= 461.98 x100cm4/pile   Sx= 2195.86 cm3/pile   Iy= 57.44 x100cm4/pile   Sy=
440.81 cm3/pile
       Top deflection = 0.492(in)
   HP16X88
   (English Units):
   Area= 25.8 in.   Depth= 15.33 in.  Width= 15.665 in.  Height= 16 in.
   Flange thickness= 0.54 in.   Web thickness= 0.54 in.
   Ix= 1112 in4/pile   Sx= 145 in3/pile   Iy= 347 in4/pile   Sy= 44 in3/pile
   (Metric Units):
   Ix= 462.81 x100cm4/pile   Sx= 2376.12 cm3/pile   Iy= 144.42 x100cm4/pile   Sy=
721.03 cm3/pile
       Top deflection = 0.492(in)
   W16X89
   (English Units):
   Area= 26.2 in.   Depth= 16.8 in.  Width= 10.4 in.  Height= 16 in.
   Flange thickness= 0.875 in.   Web thickness= 0.525 in.
   Ix= 1300 in4/pile   Sx= 155 in3/pile   Iy= 163 in4/pile   Sy= 31.4 in3/pile
   (Metric Units):
   Ix= 541.06 x100cm4/pile   Sx= 2539.99 cm3/pile   Iy= 67.84 x100cm4/pile   Sy=
514.55 cm3/pile
       Top deflection = 0.420(in)
   HP16X101
   (English Units):
   Area= 29.8 in.   Depth= 15.5 in.  Width= 15.75 in.  Height= 16 in.
   Flange thickness= 0.625 in.   Web thickness= 0.625 in.
   Ix= 1297 in4/pile   Sx= 167 in3/pile   Iy= 408 in4/pile   Sy= 52.1 in3/pile
   (Metric Units):
   Ix= 539.81 x100cm4/pile   Sx= 2736.63 cm3/pile   Iy= 169.81 x100cm4/pile   Sy=
853.76 cm3/pile
       Top deflection = 0.421(in)
   W16X100
   (English Units):
   Area= 29.5 in.   Depth= 17 in.  Width= 10.4 in.  Height= 16 in.
   Flange thickness= 0.985 in.   Web thickness= 0.585 in.
   Ix= 1490 in4/pile   Sx= 175 in3/pile   Iy= 186 in4/pile   Sy= 35.7 in3/pile
   (Metric Units):
   Ix= 620.14 x100cm4/pile   Sx= 2867.73 cm3/pile   Iy= 77.41 x100cm4/pile   Sy=
585.02 cm3/pile
       Top deflection = 0.367(in)
   HP16X121
   (English Units):
   Area= 35.7 in.   Depth= 15.75 in.  Width= 15.875 in.  Height= 16 in.
   Flange thickness= 0.75 in.   Web thickness= 0.75 in.
   Ix= 1578 in4/pile   Sx= 200 in3/pile   Iy= 501 in4/pile   Sy= 63.1 in3/pile
   (Metric Units):
```

```
   Ix= 656.76 x100cm4/pile   Sx= 3277.40 cm3/pile   Iy= 208.52 x100cm4/pile   Sy=
1034.02 cm3/pile
       Top deflection = 0.346(in)
   W18X76
   (English Units):
   Area= 22.3 in.   Depth= 18.2 in.  Width= 11 in.  Height= 18 in.
   Flange thickness= 0.68 in.   Web thickness= 0.425 in.
   Ix= 1330 in4/pile   Sx= 146 in3/pile   Iy= 152 in4/pile   Sy= 27.6 in3/pile
   (Metric Units):
   Ix= 553.55 x100cm4/pile   Sx= 2392.50 cm3/pile   Iy= 63.26 x100cm4/pile   Sy=
452.28 cm3/pile
       Top deflection = 0.411(in)
   HP18X135
   (English Units):
   Area= 39.8 in.   Depth= 17.5 in.  Width= 17.75 in.  Height= 18 in.
   Flange thickness= 0.75 in.   Web thickness= 0.75 in.
   Ix= 2196 in4/pile   Sx= 251 in3/pile   Iy= 700 in4/pile   Sy= 78.8 in3/pile
   (Metric Units):
   Ix= 913.98 x100cm4/pile   Sx= 4113.14 cm3/pile   Iy= 291.34 x100cm4/pile   Sy=
1291.30 cm3/pile
       Top deflection = 0.249(in)
   W18X130
   (English Units):
   Area= 38.2 in.   Depth= 19.3 in.  Width= 11.2 in.  Height= 18 in.
   Flange thickness= 1.2 in.   Web thickness= 0.67 in.
   Ix= 2460 in4/pile   Sx= 256 in3/pile   Iy= 278 in4/pile   Sy= 49.9 in3/pile
   (Metric Units):
   Ix= 1023.85 x100cm4/pile   Sx= 4195.07 cm3/pile   Iy= 115.70 x100cm4/pile   Sy=
817.71 cm3/pile
       Top deflection = 0.222(in)
   HP18X157
   (English Units):
   Area= 46.2 in.   Depth= 17.74 in.  Width= 17.87 in.  Height= 18 in.
   Flange thickness= 0.87 in.   Web thickness= 0.87 in.
   Ix= 2583 in4/pile   Sx= 291 in3/pile   Iy= 829 in4/pile   Sy= 93 in3/pile
   (Metric Units):
   Ix= 1075.04 x100cm4/pile   Sx= 4768.62 cm3/pile   Iy= 345.03 x100cm4/pile   Sy=
1523.99 cm3/pile
       Top deflection = 0.212(in)

********************** LAGGING SIZE ESTIMATION **********************
Max. Pressure above base = 0.43
Piles are more rigid than timber lagging, due to arching, only portion of pressures
are acting to lagging, 30-50% loading is suggested.
     If 50% loading is used for lagging design, Design Pressure = 0.22
     Pile Spacing =8.0, Max. Moment in lagging = 1.72
     For 4"x12" Timber, Section Modules S=23.47 in3.  The request allowable bending
strength, fb=M/S=0.88
```

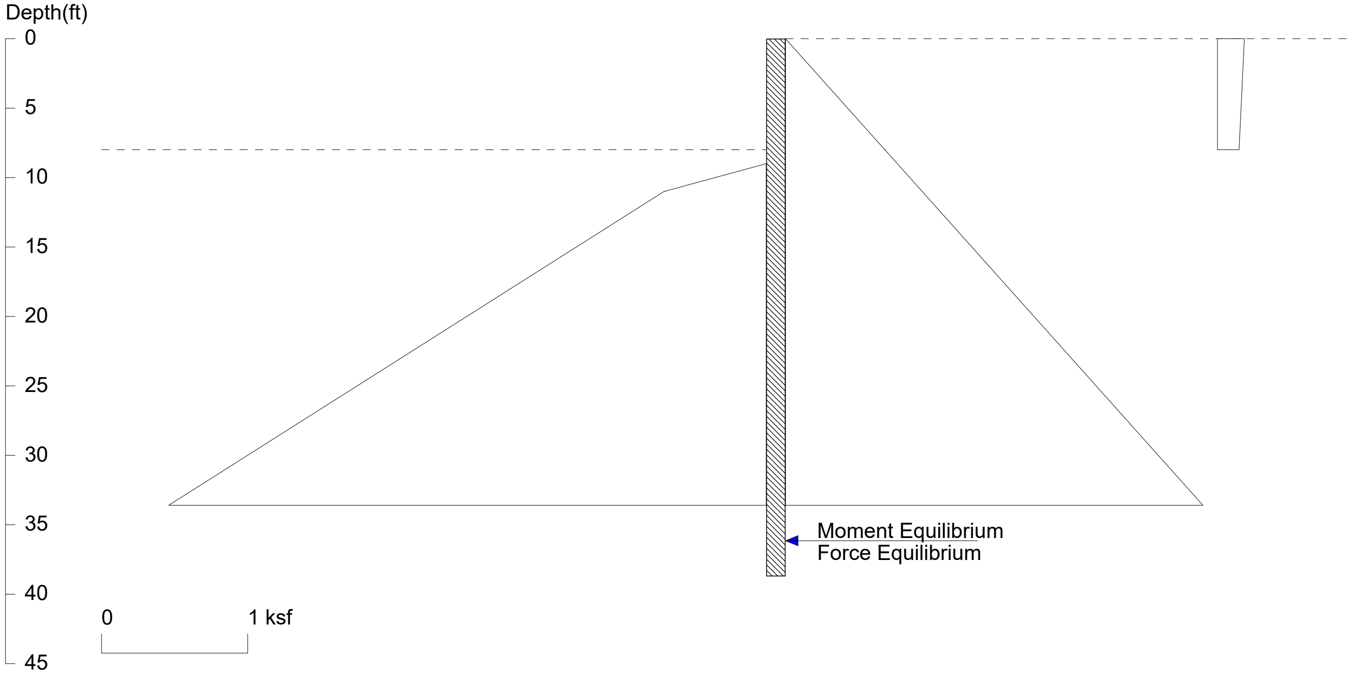
For 6"x12" Timber, Section Modules S=57.98 in3. The request allowable bending strength, fb=M/S=0.36

If 30% loading is used for lagging design, Design Pressure = 0.13
Pile Spacing =8.0, Max. Moment in lagging = 1.03
For 4"x12" Timber, Section Modules S=23.47 in3. The request allowable bending strength, fb=M/S=0.53
For 6"x12" Timber, Section Modules S=57.98 in3. The request allowable bending strength, fb=M/S=0.21

Unit: Pressure: ksf, Spacing: ft, Moment: kip-ft, Bending Strength, fb: ksi

Dufrense Residence

8' - Permanent Soldier Pile - Siesmic Loading



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File: C:\Lucia Engineering\Lucia Engineering 2017\Misc. Small Projects\Margaret Dufresne\8' - Rear Wall - With Siesmic Lo

Wall Height=8.0 Pile Diameter=2.5 Pile Spacing=8.0 Wall Type: 2. Soldier Pile, Drilled

PILE LENGTH: Min. Embedment=30.71 Min. Pile Length=38.71 (in graphics and analysis)
MOMENT IN PILE: Max. Moment=703.22 per Pile Spacing=8.0 at Depth=22.98

PILE SELECTION:
Request Min. Section Modulus = 255.7 in3/pile=4190.45 cm3/pile, Fy= 50 ksi = 345 MPa, Fb/Fy=0.66
W18X130 has Section Modulus = 256.0 in3/pile=4195.07 cm3/pile. It is greater than Min. Requirements!
Top Deflection = 1.13(in) based on E (ksi)=29000.00 and I (in4)/pile=2460.0

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

Z1	P1	Z2	P2	Slope
0	0	8	0.68	0.085000
8	0.68	50	4.25	0.085000
*	Siesm	23H		
0	0.184	8	0.148	-0.00450

PASSIVE PRESSURES:

Z1	P1	Z2	P2	Slope
9	0	11	0.70	0.3500
11	0.70	50	6.55	0.1500

ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	8.00
2	15.00	2.50

PASSIVE SPACING:

No.	Z depth	Spacing
1	9.00	6.00

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

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****

SHORING WALL CALCULATION SUMMARY
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****
ShoringSuite Software is developed by CivilTech Software, Bellevue, WA, USA.
The calculation method is based on the following references:

1. FHWA 98-011, FHWA-RD-97-130, FHWA SA 96-069, FHWA-IF-99-015
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3. DESIGN MANUAL DM-7 (NAVFAC), Department of the Navy, May 1982
4. TRENCHING AND SHORING MANUAL Revision 12, California Department of Transportation, January 2000
6. EARTH SUPPORT SYSTEM & RETAINING STRUCTURES, Pile Buck Inc. 2002
5. DESIGN OF SHEET PILE WALLS, EM 1110-2-2504, U.S. Army Corps of Engineers, 31 March 1994
7. EARTH RETENTION SYSTEMS HANDBOOK, Alan Macnab, McGraw-Hill. 2002
8. AASHTO HB-17, American Association of State and Highway Transportation Officials, 2 September 2002

UNITS: Width/Spacing/Diameter/Length/Depth - ft, Force - kip, Moment - kip-ft, Friction/Bearing/Pressure - ksf, Pres. Slope - kip/ft3, Deflection - in
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Title: Dufrense Residence
Subtitle: 8' - Permanent Soldier Pile - Siesmic Loading

*********************************INPUT DATA**********************************
Wall Type: 2. Soldier Pile, Drilled
        Wall Height: 8.00
        Pile Diameter: 2.50
        Pile Spacing: 8.00
        Factor of Safety (F.S.): 1.00
Lateral Support Type (Braces): 1. No
        Top Brace Increase (Multi-Bracing): Add 15%*
Embedment Option: 1. Yes
        Friction at Pile Tip: No
Pile Properties:
        Steel Strength, Fy: 50 ksi = 345 MPa
        Allowable Fb/Fy: 0.66
        Elastic Module, E: 29000.00
        Moment of Inertia, I: 1890
        User Input Pile: W18X130

* DRIVING PRESSURE (ACTIVE, WATER, & SURCHARGE) *

No.	Z1 top	Top Pres.	Z2 bottom	Bottom Pres.	Slope
1	0	0	8	0.68	0.085000
2	8	0.68	50	4.25	0.085000
3	*	Siesm	23H		
4	0	0.184	8	0.148	-0.00450

* PASSIVE PRESSURE *

No.	Z1 top	Top Pres.	Z2 bottom	Bottom Pres.	Slope
1	9	0	11	0.70	0.3500
2	11	0.70	50	6.55	0.1500

* ACTIVE SPACE *

No.	Z depth	Spacing
1	0.00	8.00
2	15.00	2.50

* PASSIVE SPACE *

No.	Z depth	Spacing
1	9.00	6.00

*For Tieback: Input1 = Diameter; Input2 = Bond Strength
*For Plate: Input1 = Diameter; Input2 = Allowable Pressure
*For Deadman: Input1 = Horz. Width; Input2 = Passive Pressure;
*For Sheet Pile Anchor: Input1 = Horz. Width; Input2 = Passive Slope;

********************************CALCULATION**********************************

The calculated moment and shear are per pile spacing. Sheet piles are per one foot or meter; Soldier piles are per pile.

Top Pressures start at depth = 0.00

```
        |      D1=0.00
        |
        |
        |
      ==|==    D2=8.00
```

```
       |
       |
       |      D3=38.71

       D1 - TOP DEPTH
       D2 - EXCAVATION BASE
       D3 - PILE TIP (20% increased, see EMBEDMENT Notes below)

   MOMENT BALANCE: M=0.00   AT DEPTH=33.59  WITH EMBEDMENT OF 25.59
   FORCE BALANCE: F=0.00   AT DEPTH=38.71  WITH EMBEDMENT OF 30.71
```

The program calculates an embedment for moment equilibrium, then increase the embedment by 20% to reach force equilibrium.
A Balance Force=145.79 is developed from depth=33.59 to depth=38.71
Total Passive Pressure = Total Active Pressure,   OK!

*********************************RESULTS***

* EMBEDMENT Notes *
Based on USS Design Manual, first calculate embedment for moment equilibrium, then increased by 20 to 40 % to get the total design depth.
The embedment for moment equilibrium is 25.59
* The 20% increased the total design depth is 30.71 (Used by Program)
The 30% increased the total design depth is 33.27
The 40% increased the total design depth is 35.83

Based on AASHTO 2002 Standard Specifications, first calculate embedment for moment equilibrium, then add safety factor of 30% for temporary shoring; add safety factor of 50% for permanent shoring.
The embedment for moment equilibrium is 25.59
Add 30% embedment for temporary shoring is 33.27
Add 50% embedment for permanent shoring is 38.39

* BASED ON USS DESIGN MANUAL (20% increased), PROGRAM CALCULATED MINIMUM EMBEDMENT = 30.71
TOTAL MINIMUM PILE LENGTH = 38.71

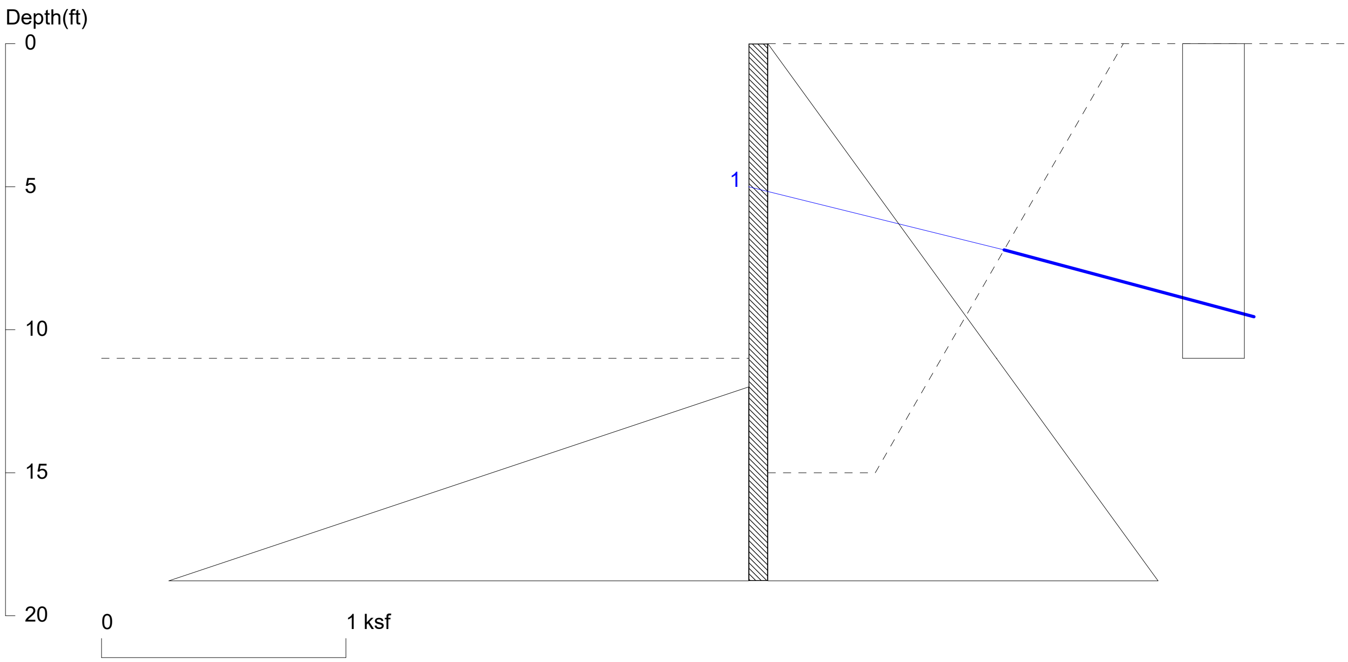
* MOMENT IN PILE (per pile spacing)*
Pile Spacing: sheet piles are one foot or one meter; soldier piles are one pile.
Overall Maximum Moment = 703.22 at 22.98
Maximum Shear = 145.10
Moment and Shear are per pile spacing: 8.0 foot or meter

* VERTICAL LOADING *
Vertical Loading from Braces = 0.00
Vertical Loading from External Load = 0.00
Total Vertical Loading = 0.00

```
***************************SPECIFIED PILE **********************************

   Overall Maximum Moment = 703.22 at 22.98
   The pile selection is based on the magnitude of the moment only. Axial force is
neglected.

   Request Min. Section Modulus = 255.72 in3/pile = 4190.45 cm3/pile, Fy= 50 ksi =
345 MPa, Fb/Fy=0.66

   W18X130 has been found in Soldier Pile list!
   (English Units):
   Area= 38.2 in.  Depth= 19.3 in.  Width= 11.2 in.  Height= 18 in.
   Flange thickness= 1.2 in.   Web thickness= 0.67 in.
   Ix= 2460 in4/pile   Sx= 256 in3/pile   Iy= 278 in4/pile   Sy= 49.9 in3/pile
   (Metric Units):
   Ix= 1023.85 x100cm4/pile   Sx= 4195.07 cm3/pile   Iy= 115.70 x100cm4/pile   Sy=
817.71 cm3/pile



   The pile selection is based on the magnitude of the moment only. Axial force is
neglected.

   W18X130 is capable to support the shoring!
   Top deflection = 1.133(in)
   Max. deflection = 1.133(in)

********************** LAGGING SIZE ESTIMATION **********************
Max. Pressure above base = 0.83
Piles are more rigid than timber lagging, due to arching, only portion of pressures
are acting to lagging, 30-50% loading is suggested.
     If 50% loading is used for lagging design, Design Pressure = 0.41
     Pile Spacing =8.0, Max. Moment in lagging = 3.31
     For 4"x12" Timber, Section Modules S=23.47 in3.  The request allowable bending
strength, fb=M/S=1.69
     For 6"x12" Timber, Section Modules S=57.98 in3.  The request allowable bending
strength, fb=M/S=0.69

     If 30% loading is used for lagging design, Design Pressure = 0.25
     Pile Spacing =8.0, Max. Moment in lagging = 1.99
     For 4"x12" Timber, Section Modules S=23.47 in3.  The request allowable bending
strength, fb=M/S=1.02
     For 6"x12" Timber, Section Modules S=57.98 in3.  The request allowable bending
strength, fb=M/S=0.41

Unit: Pressure: ksf,  Spacing: ft,  Moment: kip-ft, Bending Strength, fb: ksi
```

Dufrense Residence

11' - Permanent Soldier Pile - Siesmic Loading



<ShoringSuite> CIVILTECH SOFTWARE USA www.civiltech.com

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File: C:\Lucia Engineering\Lucia Engineering 2017\Misc. Small Projects\Margaret Dufresne\11' - Rear Wall - With Siesmic L

Wall Height=11.0 Pile Diameter=2.5 Pile Spacing=8.0 Wall Type: 2. Soldier Pile, Drilled

PILE LENGTH: Min. Embedment=7.78 (8~10ft is recommended!!!) Min. Pile Length=18.78 (in graphics and analysis)
MOMENT IN PILE: Max. Moment=111.17 per Pile Spacing=8.0 at Depth=11.09

PILE SELECTION:
Request Min. Section Modulus = 40.4 in3/pile=662.48 cm3/pile, Fy= 50 ksi = 345 MPa, Fb/Fy=0.66
W18X35 has Section Modulus = 57.6 in3/pile=943.89 cm3/pile. It is greater than Min. Requirements!
Top Deflection = -0.19(in) based on E (ksi)=29000.00 and I (in4)/pile=510.0

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

No. & Type	Depth	Angle	Space	Total F.	Horiz. F.	Vert. F.	L_free	Fixed Length
1. Tieback	5.0	15.0	6.0	49.8	48.1	12.9	8.5	21.1

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	0	11	0.935	0.085000
11	0.935	50	4.25	0.085000
*	Siesm	23H		
0	0.253	11	0.253	0.000000

PASSIVE PRESSURES:

Z1	P1	Z2	P2	Slope
12	0	19	2.45	0.3500

ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	8.00
2	15.00	2.50

PASSIVE SPACING:

No.	Z depth	Spacing
1	12.00	6.00

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

report.out
****

SHORING WALL CALCULATION SUMMARY
The leading shoring design and calculation software
Software Copyright by CivilTech Software
www.civiltech.com

****
ShoringSuite Software is developed by CivilTech Software, Bellevue, WA, USA.
The calculation method is based on the following references:
    1. FHWA 98-011, FHWA-RD-97-130, FHWA SA 96-069, FHWA-IF-99-015
    2. STEEL SHEET PILING DESIGN MANUAL by Pile Buck Inc., 1987
    3. DESIGN MANUAL DM-7 (NAVFAC), Department of the Navy, May 1982
    4. TRENCHING AND SHORING MANUAL Revision 12, California Department of Transportation, January 2000
    6. EARTH SUPPORT SYSTEM & RETAINING STRUCTURES, Pile Buck Inc. 2002
    5. DESIGN OF SHEET PILE WALLS, EM 1110-2-2504, U.S. Army Corps of Engineers, 31 March 1994
    7. EARTH RETENTION SYSTEMS HANDBOOK, Alan Macnab, McGraw-Hill. 2002
    8. AASHTO HB-17, American Association of State and Highway Transportation Officials, 2 September 2002

UNITS: Width/Spacing/Diameter/Length/Depth - ft, Force - kip, Moment - kip-ft, Friction/Bearing/Pressure - ksf, Pres. Slope - kip/ft3, Deflection - in
----
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Date: 10/15/2017 File: C:\Lucia Engineering\Lucia Engineering 2017\Misc. Small Projects\Margaret Dufresne\11' - Rear Wall - With Siesmic Loading - Piles 3 & 15.sh8

Title: Dufrense Residence
Subtitle: 11' - Permanent Soldier Pile - Siesmic Loading

**********************************INPUT DATA**********************************
Wall Type: 2. Soldier Pile, Drilled
        Wall Height: 11.00
        Pile Diameter: 2.50
        Pile Spacing: 8.00
        Factor of Safety (F.S.): 1.00
Lateral Support Type (Braces): 3. Tieback
        Top Brace Increase (Multi-Bracing): Add 15%*
        Brace Position (One Brace Case): Normal Brace*
No-Load Zone:
        Vertical Depth for No-Load Zone: 15.00
        H-Distance (Input H/V ratio) for No-Load Zone: 0.25
        Angle from H. Line for No-Load Zone: 60.00
Embedment Option: 1. Yes
        Friction at Pile Tip: No

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```
Pile Properties:
        Steel Strength, Fy:  50 ksi = 345 MPa
        Allowable Fb/Fy: 0.66
        Elastic Module, E: 29000.00
        Moment of Inertia, I: 184
        User Input Pile: W18X35

* DRIVING PRESSURE (ACTIVE, WATER, & SURCHARGE) *
 No.    Z1 top          Top Pres.       Z2 bottom       Bottom Pres.    Slope
-----------------------------------------------------------------------------
 1      0               0               11              0.935           0.085000
 2      11              0.935           50              4.25            0.085000
 3      *               Siesm           23H
 4      0               0.253           11              0.253           0.000000
-----------------------------------------------------------------------------

* PASSIVE PRESSURE *
 No.    Z1 top          Top Pres.       Z2 bottom       Bottom Pres.    Slope
-----------------------------------------------------------------------------
 1      12              0               19              2.45            0.3500
 2      19              2.45            50              7.10            0.1500
-----------------------------------------------------------------------------

* ACTIVE SPACE *
 No.    Z depth         Spacing
-----------------------------------------------------------------------------
 1      0.00            8.00
 2      15.00           2.50
-----------------------------------------------------------------------------

* PASSIVE SPACE *
 No.    Z depth         Spacing
-----------------------------------------------------------------------------
 1      12.00           6.00
-----------------------------------------------------------------------------

* BRACE: STRUT, TIEBACK, ANCHOR PLATE, DEADMAN, OR SHEET PILE AS ANCHOR*
 No.    Z brace         Angle           Spacing         Input1*         Input2*
Type
-----------------------------------------------------------------------------------
----------------------
 1      5.00            15.0            6.00            0.50            1.50
Tieback
-----------------------------------------------------------------------------------
----------------------
*For Tieback:  Input1 = Diameter;  Input2 = Bond Strength
*For Plate:  Input1 = Diameter;  Input2 = Allowable Pressure
*For Deadman:  Input1 = Horz. Width;  Input2 = Passive Pressure;
```

```
*For Sheet Pile Anchor:  Input1 = Horz. Width;  Input2 = Passive Slope;


********************************CALCULATION**********************************

   The calculated moment and shear are per pile spacing.  Sheet piles are per one
foot or meter; Soldier piles are per pile.

   Top Pressures start at depth =  0.00


   NUMBER OF BRACE LEVEL = 1


        |      D1=0.00
        |
        |<--   D2=5.00          R1=64.07
        |
        |
        |
      ==|==    D3=11.00
        |
        |      D4=18.78

        D1 - TOP DEPTH
        D2 - BRACE DEPTH         R1 - REACTION
        D3 - EXCAVATION BASE
        D4 - PILE TIP

   TOTAL REACTION: R1 = 64.07
   TOTAL PRESSURES ACTING ON WALL = 64.07
      Total Reactions = Total Pressures,    OK!
   __________________________________________________
   BRACE NO.1 AT DEPTH = 5.00
       R1 = Brace Load = 64.07
   __________________________________________________


*********************************RESULTS*****************************************

* EMBEDMENT *
MINIMUM EMBEDMENT = 7.78 (8~10ft recommended!!!),  TOTAL MINIMUM PILE LENGTH =
18.78

* MOMENT IN PILE (per pile spacing)*
Pile Spacing:  sheet piles are one foot or one meter; soldier piles are one pile.
 No.    Depth           M @ Brace       Mmax in Span    Depth of Mmax
-----------------------------------------------------------------------------
```

```
 1      5.00            38.96           111.17          11.09
-----------------------------------------------------------------------------
Overall Maximum Moment = 111.17 at 11.09
Maximum Shear = 45.37
Moment and Shear are per pile spacing: 8.0 foot or meter

* BRACE: STRUT, TIEBACK, ANCHOR PLATE, DEADMAN, OR SHEET PILE AS ANCHOR*
The calculated brace force are per brace spacing.

 No.    DEPTH           Tangle          SPACING         HORIZONTAL      VERTICAL
 TOTAL LOAD
-----------------------------------------------------------------------------------
-----------
 1      5.00            15.0            6.00            48.06           12.88
49.75

 No.    DEPTH           Free length     Brace Type
-----------------------------------------------------------------------------------
-----------
 1      5.00            8.54            Tieback, Bond length = 21.11
-----------------------------------------------------------------------------------
-----------

* VERTICAL LOADING *
Vertical Loading from Braces = 17.17
Vertical Loading from External Load = 0.00
Total Vertical Loading = 17.17


*****************************SPECIFIED PILE ***********************************

   Overall Maximum Moment = 111.17 at 11.09
   The pile selection is based on the magnitude of the moment only. Axial force is
neglected.

   Request Min. Section Modulus = 40.43 in3/pile = 662.48 cm3/pile, Fy= 50 ksi =
345 MPa, Fb/Fy=0.66

   W18X35 has been found in Soldier Pile list!
   (English Units):
   Area= 10.3 in.  Depth= 17.7 in.  Width= 6 in.  Height= 18 in.
   Flange thickness= 0.425 in.   Web thickness= 0.3 in.
   Ix= 510 in4/pile   Sx= 57.6 in3/pile   Iy= 15.3 in4/pile   Sy= 5.12 in3/pile
   (Metric Units):
   Ix= 212.26 x100cm4/pile   Sx= 943.89 cm3/pile   Iy= 6.37 x100cm4/pile   Sy=
83.90 cm3/pile
```

The pile selection is based on the magnitude of the moment only. Axial force is neglected.

W18X35 is capable to support the shoring!
Top deflection = -0.192(in)
Max. deflection = 0.196(in)

********************** LAGGING SIZE ESTIMATION **********************
Max. Pressure above base = 1.19
Piles are more rigid than timber lagging, due to arching, only portion of pressures are acting to lagging, 30-50% loading is suggested.
If 50% loading is used for lagging design, Design Pressure = 0.59
Pile Spacing =8.0, Max. Moment in lagging = 4.75
For 4"x12" Timber, Section Modules S=23.47 in3. The request allowable bending strength, fb=M/S=2.43
For 6"x12" Timber, Section Modules S=57.98 in3. The request allowable bending strength, fb=M/S=0.98

If 30% loading is used for lagging design, Design Pressure = 0.36
Pile Spacing =8.0, Max. Moment in lagging = 2.85
For 4"x12" Timber, Section Modules S=23.47 in3. The request allowable bending strength, fb=M/S=1.46
For 6"x12" Timber, Section Modules S=57.98 in3. The request allowable bending strength, fb=M/S=0.59

Unit: Pressure: ksf, Spacing: ft, Moment: kip-ft, Bending Strength, fb: ksi

GEOTECHNICAL REPORT

Dufresne Residence

11143 Rolling Bay Walk NE
Bainbridge Island, Washington

Prepared for: Ms. Margaret Dufresne

Project No. 170202-01 • July 20, 2017

e a r t h + w a t e r

Aspect
CONSULTING

GEOTECHNICAL REPORT

Dufresne Residence

11143 Rolling Bay Walk NE
Bainbridge Island, Washington

Prepared for: Ms. Margaret Dufresne

Project No. 170202-01 • July 20, 2017

Aspect Consulting, LLC

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1 Introduction

This report presents the results of a geotechnical engineering evaluation performed by Aspect Consulting, LLC (Aspect) for the proposed residence at 11143 Rolling Bay Walk NE, Bainbridge Island, Washington (Site). The Site location is shown on Figure 1, *Site Location Map*.

The proposed residence is located adjacent to Rolling Bay Walk NE and at the base of a tall, steep, northeast-facing slope that has been subject to landslide activity as recently as January of 1997 when a landslide destroyed the previous residence at the Site. A landslide debris catchment wall system was constructed on the steep slope across the width of the Site and above the proposed residence location in 2006. The design and construction documentation of the catchment wall system was completed by GeoEngineers, Inc (2005, 2006, 2009, 2010). Global stability evaluations for the Site and proposed residence, taking into account the catchment wall system, were completed by GeoEngineers, Inc (2009) and Perrone Consulting, Inc (2016). The Site addresses in the previous studies were 11129 and 11139 Rolling Bay Walk NE but were subsequently changed to 11131 and 11143 Rolling Bay Walk NE and then consolidated to the present single lot address of 11143 Rolling Bay Walk NE.

The steep slope is mapped as a geologically hazardous area by the City of Bainbridge Island (City). The proposed residence will be located within the identified geologically hazardous area.

The purpose of this evaluation is to characterize the subsurface conditions underlying the Site, assess the current slope conditions, review the catchment wall stabilization and global stability analyses completed by others, summarize our geotechnical engineering analyses, and to provide the design basis for the proposed residence including foundation and shoring alternatives.

1.1 Scope of Services

Our scope of services included a review of the existing Site data, a Site reconnaissance, subsurface explorations, and geotechnical engineering evaluations. This report includes:

- Site and project descriptions;
- Distribution and characteristics of subsurface soils and groundwater;
- Review of global slope stability analyses completed by others;
- Seismic design considerations in accordance with the current version of the International Building Code (IBC);
- Suitable foundation and excavation shoring types and associated design criteria;
- Site preparation recommendations and general construction recommendations; and

- A completed Step-1 Form as required by the City for improvements within a geologically hazardous area.

1.2 Project Understanding

The Site consists of Kitsap County Parcel No. 4156-001-004-1006 at 11143 Rolling Bay Walk NE in Bainbridge Island, Washington. The Project includes the construction of a new single-family residence and associated infrastructure at the Site.

A single-family residence previously occupied the Site but was destroyed by the landslide in 1997 and subsequently demolished. A landslide debris catchment wall system was constructed in 2006 and consisted of two tied-back (anchored) soldier pile retaining walls tiered at a location approximately one third of the way up the steep slope. We understand that the construction of the catchment wall system was observed on a full-time basis by GeoEngineers Inc. (2009 and 2010), who confirmed that the wall system was constructed in accordance with their recommendations.

The proposed residence will include a garage at the lowest level corresponding to the elevation of Rolling Bay Walk NE at about Elevation 10 feet (NAVD88) and two stories of living space above the garage. The approximate proposed building footprint is shown on Figure 2. An excavation (cut) at the toe of the steep slope on the order of 13 to 15 feet tall is required to construct the garage level. The western edge of the proposed residence is located approximately 16 feet northeast from the face of the lower wall of the landslide debris catchment wall system. Sewage effluent will be pumped to a new septic drainfield located upslope of the residence and between the tiered soldier pile catchment walls. At the time of this report, the project is in its conceptual design phase and we assume the proposed residence will be designed for relatively conventional residential construction with no unique or extraordinary loads.

Nearby subsurface exploration data includes two soil borings that were completed to depths of approximately 30 feet on the adjacent properties north and south of the Site in support of residential projects similar to what is being proposed at the Site.

2 Site Conditions

2.1 Site Description

The Site is a 0.44-acre rectangular parcel, approximately 125 feet wide (northwest to southeast direction) and 150 feet long (southwest to northeast direction). Rolling Bay Walk NE separates the Site from the Puget Sound to the northeast, as shown on Figure 2. On the northeast side of Rolling Bay Walk NE was a concrete bulkhead, approximately 1 foot tall, and on the southwest side of Rolling Bay Walk NE was a concrete retaining wall, approximately 1.5 feet tall.

Overall, the steep slope at the Site is inclined at approximately 100 percent or 1H:1V (Horizontal:Vertical) and is approximately 120 feet tall. The lower third of the steep slope is less steep than the overall slope with inclinations on the order of 50 to 80 percent from the level of Rolling Bay Walk NE up to the tiered soldier pile wall system. The wall system includes an upper and lower soldier pile retaining wall. Both walls are embedded well-below the surface of the steep slope and anchored by tieback anchors consisting of concrete-encased steel tendons. The space between the two walls is relatively flat and approximately 15 feet wide. Above the walls, the steep slope rises to the top of the slope at inclinations on the order of 115 percent.

The existing Site layout, topography, and the location of our subsurface explorations are shown on Figure 2.

2.1.1 Drainage

We did not observe standing water across the Site at the time of our reconnaissance nor did we observe significant/concentrated groundwater seepage emitting from the face of the steep slope. We observed scattered 4-inch-diameter black corrugated plastic pipes on the steep slope above the catchment wall system. The purpose and function of these pipes was not clear at the time of our reconnaissance, but they are/were likely related to drainage for the properties located at/near the top of the steep slope.

The drainage conditions as well as groundwater conditions at the Site will vary with fluctuations in precipitation, tidal conditions, Site usage (such as irrigation), and off-Site land use.

2.1.2 Vegetation

The lower portion of the steep slope is covered with blackberries, grasses, horsetail reeds, and small deciduous trees. The horsetail reeds are hydrophilic (water-loving) and indicative of perennial (year-round) wetness. The upper portion of the steep slope consist of mature deciduous trees and undergrowth consisting of grasses, blackberries, and ferns. The mature deciduous trees on the steep slope typically exhibit lean and trunk curvature in the downslope direction.

2.1.3 Slope Conditions

An Aspect geologist performed a Site reconnaissance on May 3, 2017, to observe the existing conditions and, to the extent possible, identify geologic and landslide-related features. The Site reconnaissance was performed by traversing the steep slope and noting visible features such as scarps, cracks, springs, vegetation, and general geomorphology that may be indicative of new or incipient ground movement.

The slope surface is covered with grasses, blackberry bushes, ivy, and young deciduous trees. We did not observe any signs of ongoing or incipient movement on the slope, such as bare soil, tension cracks, prominent groundwater seepage, or uncontrolled surface water. The soldier pile catchment wall system did not show any obvious signs of movement or distress, appeared plumb, and in relatively good condition.

2.2 Geologic Setting

2.2.1 Mapped Geology

The geologic map (Haugerud and Troost, 2011) of the Project vicinity shows the Site as underlain by Pleistocene Pre-Vashon-age deposits (Qpv). Northwest of the Site, a portion of the slope has been mapped as Quaternary landslide deposits (Qls); southeast of the Site, Holocene Beach deposits (Qb) have been mapped along the shore. The uplands southwest of the Site are mapped as Pleistocene Vashon till (Qvt).

Quaternary landslide deposits (Qls): Mapped landslide deposits are usually deep-seated and identified based on morphology. Commonly water-saturated and relatively less dense, they can consist of diamict, sand, gravel, silt, or any combination of the above.

Holocene beach deposits (Qb): Beach deposits are found along the shoreline and consist of sand, gravel, and logs deposited by wave action.

Pleistocene Vashon till (Qvt): Vashon till was deposited during the most recent glacial advance and were overridden and compacted by glacial ice, creating a dense/hard configuration. The Qvt unit is generally described as a dense diamict of sand, gravel, and silt. Shoreline bluff exposures indicate that most of the Vashon till is subglacial lodgment till.

Pleistocene Pre-Vashon-age deposits (Qpv): This unit was deposited prior to the most recent glacial advance, which was about 15,000 years ago. The Qpv unit is a broad category including both glacial and nonglacial sand, gravel, silt, peat, sandstone, mudstone, conglomerate, and diamict. According to the mapped unit description, areas mapped as Qpv may include minor amounts of the **Esperance Sand Member (Qve)** and the **Lawton Clay Member (Qvlc)** of the Pleistocene Vashon Drift, which are stratigraphically adjacent and above (just younger than) the Qpv unit. The Qve unit consists of dense, fine to medium sand deposited in glacial advance outwash settings; the Qvlc unit consists of thinly-bedded, dark gray silt and clay deposited in glacial lake settings.

2.2.2 Mapped Geologic Hazards

The City maps the Site and steep slope as a greater than 40 percent geologically hazardous area (COBI, 2015). No landslides have been formally mapped on the Site (COBI, 2015; McKenna et al., 2008; Ecology 1979), most likely due to the lack of a prominent headscarp produced by the 1997 and previous surficial landslides on the steep slope. Liquefaction susceptibility at and near the Site is mapped as very low (DNR, 2004).

2.3 Subsurface Conditions

2.3.1 Subsurface Explorations

We inferred subsurface conditions at the Site using the completed field explorations, existing subsurface exploration data from the adjacent properties, and our experience with the local geology. On May 3, 2017, borings B-01 and B-02 were advanced to 30.9 feet and 26.4 feet below ground surface (bgs), respectively. The locations of the borings are shown on Figure 2.

Our subsurface explorations correlate well with the geologic map of the area (Haugerud and Troost, 2011) and with subsurface conditions encountered by GeoEngineers, Inc. and Myers Biodynamics Inc. on adjacent properties. Our subsurface explorations at the Site indicate that landslide deposits (Qls) generally overlie advance outwash deposits (Qve). Near the toe of the slope, beach deposits (Qb) were found stratigraphically between the landslide deposits and the advance outwash deposits.

Detailed descriptions of the subsurface conditions encountered in our explorations, as well as the depths where characteristics of the soils changed, are on the soil boring logs presented in Appendix A. Soils were classified per the Unified Soil Classification System (USCS) in general accordance with the American Society for Testing and Materials (ASTM) D2488, Standard Practice for Description and Identification of Soils (Visual and Manual Procedure). A key to the symbols and terms used on the logs is provided in Figure A-1. The depths on the logs where conditions changed may represent gradational variations between soil types and actual transitions may be more gradual.

2.3.2 Stratigraphy

The soil conditions observed during the subsurface exploration are summarized as follows.

Landslide Deposits (Qls)

We encountered landslide deposits at the ground surface in both borings to depths ranging from 15 feet below the ground surface (bgs) in B-01 to 7.5 feet bgs in B-02. The landslide deposits typically consisted of very loose to loose, moist, brown to dark brown, gravelly, silty SAND (SM)[1] or sandy, slightly silty GRAVEL (GW-GM). Between 2.5 and 7.5 feet bgs in B-02, we encountered void space and residential debris such as carpet, pieces of plastic, concrete blocks, and Styrofoam within a sparse soil matrix of silty SAND (SM).

The Standard Penetration Test (SPT)[2] blow counts from the explorations in the landslide deposits ranged 7 to 14 blows per foot, indicating that the landslide deposits are typically loose. The landslide deposits can be expected to have low shear strength, moderate to high

---

[1] Soil Classification per the Unified Soil Classification System (USCS). Refer to American Society of Testing and Materials (ASTM) D2488.

[2]SPT blow count refers to standard penetration test (SPT) N-values, in accordance with ASTM D1586.

compressibility, moderate to high permeability, and are susceptible to liquefaction (when saturated) under seismic shaking.

Beach Deposits (Qb)

We encountered beach deposits underlying the landslide deposits in B-02 from 7.5 to 13.5 feet bgs. The beach deposits consisted of medium dense, wet, brown, sandy GRAVEL (GP) overlying very gravelly, well-graded SAND (SW), with trace amounts of silt. The gravel encountered was subrounded to well-rounded, with a characteristic flat shape that matches the shape of the present-day beach gravels on the adjacent exposed shoreline.

The SPT blow counts from the exploration in the beach deposits ranged from 12 to 20 blows per foot, indicating that the beach deposits are typically medium dense. The beach deposits can be expected to have moderate shear strength, low to moderate compressibility, high permeability, and are susceptible to liquefaction (when saturated) under seismic shaking.

Advance Outwash Deposits (Qve)

Both explorations were terminated in advance outwash deposits underlying the landslide deposits and beach deposits. The outwash deposits were encountered at 15 feet and 13.5 feet bgs in borings B-01 and B-02, respectively. These deposits typically consisted of dense to very dense, moist to wet, light brown to gray, relatively homogenous fine to medium SAND with varying amounts of silt (SP, SP-SM, and SM) and trace fine gravel.

The SPT blow counts from the explorations in the advance outwash deposits ranged from 27 to greater than 90 blows per foot, indicating that the advance outwash deposits are typically dense to very dense. The advance outwash deposits can be expected to have high shear strength, low compressibility, moderate permeability, and are not typically susceptible to liquefaction (when saturated) under seismic shaking.

2.3.3 Groundwater

In boring B-01, groundwater was first encountered during drilling at approximately 23 feet bgs, or at approximately Elevation 5 (NAVD88). In boring B-02, water balancing and the addition of bentonite slurry was required to drill through the residential debris and to combat sand heave within the boring; therefore, we were unable to directly observe the groundwater level during drilling at the toe of the slope. However, based on the stratigraphy and moisture content trends, the groundwater conditions near the toe of the steep slope are in hydraulic continuity with the Puget Sound and tide gauge records from the Seattle shoreline (station ID 9447130) show the tide at approximately Elevation 2.5 (NAVD88) at the time of drilling (NOAA, 2017).

Based on our observations, the groundwater levels recorded in the explorations on the adjacent properties, and our local hydrogeologic experience, the groundwater near the toe of the steep slope will generally mirror the adjacent water levels of the Puget Sound and fluctuate with the tide stages. We recommend a design groundwater level of Elevation 5 (NAVD88). The groundwater levels at the Site will also fluctuate seasonally with precipitation as well as with changes in usage.

3 Geologic Hazards

3.1 Seismic Considerations

The Site is located within the Puget Lowland physiographic province, an area of active seismicity that is subject to earthquakes on shallow crustal faults and deeper subduction zone earthquakes. The Site area lies about 3 miles north of the Seattle fault zone, which consists of shallow crustal tectonic structures that are considered active (evidence for movement within the Holocene [since about 15,000 years ago]) and is believed to be capable of producing earthquakes of magnitude 7.3 or greater. The recurrence interval of earthquakes on this fault zone is believed to be on the order of a thousand years or more. The most recent large earthquake on the Seattle fault occurred about 1,100 years ago (Pratt et al., 2015). There are also several other shallow crustal faults in the region capable of producing earthquakes and strong ground shaking.

The Site area also lies within the zone of strong ground shaking from earthquakes associated with the Cascadia Subduction Zone (CSZ). Subduction zone earthquakes occur due to rupture between the subducting oceanic plate and the overlying continental plate. The CSZ can produce earthquakes up to magnitude 9.3 and the recurrence interval is thought to be on the order of about 500 years. A recent study estimates the most recent subduction zone earthquake occurred around 1700 (Atwater et al., 2015).

Deep intra-slab earthquakes, which occur from tensional rupture of the sinking oceanic plate, are also associated with the CSZ. An example of this type of seismicity is the 2001 Nisqually earthquake. Deep intra-slab earthquakes typically are magnitude 7.5 or less and occur approximately every 10 to 30 years.

The following sections present descriptions of the seismic hazards present at the Site and seismic design considerations.

3.1.1 Seismic Design Criteria

Inertial seismic forces are expected to affect the Site and proposed residence. Appropriate design of residence in accordance with the current version of the International Building Code (IBC) and selection of an appropriate foundation system will mitigate the relevant seismic hazards.

The IBC requires design for a "Maximum Considered Earthquake (MCE)" with a 2 percent probability of exceedance (PE) in 50 years (2,475-year return period; IBC, 2015). The USGS has completed probabilistic ground motion studies and maps for Washington (USGS, 2016).

The IBC requires the determination of the seismic Site Class in accordance with Chapter 20 of American Society of Civil Engineers (ASCE)7-10, *Minimum Design Loads for Buildings and Other Structures*. Chapter 20 of ASCE 7-10 indicates that soils vulnerable to collapse under seismic loading, such as liquefiable soils, shall be classified as Site Class F, and a site response analysis shall be performed unless the project meets the listed exception criteria. For structures having fundamental periods of vibration equal to or less than 0.5 seconds, a site response analysis is not required and the Site Class can be determined in accordance with Section 20.3 of ASCE 7-10. A conservative estimate of the fundamental period of vibration for the proposed residence (a three-story timber-framed house) is 0.3 seconds; therefore, a site response analysis is not required and a seismic Site Class C is applicable based on the average and inferred soil density revealed by the completed soil borings at the Site and anticipation of foundation support from the dense to very dense advance outwash deposits.

Based on the latitude and longitude of the Site (47.66408°N, -122.50307°W), the mapped maximum considered earthquake spectral response accelerations for short period (S_s) = 1.352g and for 1-second period (S_1) = 0.531g. Site coefficients for this Site are F_a = 1.0, F_v = 1.3. The maximum considered earthquake spectral response accelerations adjusted for Site class effects are S_{ds} = 0.901g, S_{d1} = 0.460g.

Using methods presented in the IBC and ASCE Standard 7-10 we calculated a peak ground acceleration (PGA) of 0.55g during the MCE.

3.1.2 Liquefaction

Liquefaction occurs when loose, saturated, and relatively cohesionless soil deposits temporarily lose strength from seismic shaking. The primary factors controlling the onset of liquefaction include intensity and duration of strong ground motion, characteristics of subsurface soil, *in situ* stress conditions, and the depth to groundwater. Potential effects of soil liquefaction in the context of the project and Site include soil strength reduction during and after strong shaking, liquefaction-induced settlement and lateral spreading.

Although the Washington Department of Natural Resources (DNR) maps the Site as having a low susceptibility to liquefaction (DNR, 2004), the presence of saturated loose to medium dense landslide and beach deposits underlying a portion of the Site indicates that liquefaction is a potentially relevant hazard.

Liquefaction evaluations for the Site were conducted with the aid of LiquefyPro, a seismically induced liquefaction and settlement analyses software program developed by CivilTech Software (2009); and of WSliq, a liquefaction analysis software program that was created as part of an extended research project supported by the Washington State Department of Transportation (WSDOT) and authored by Steve Kramer (Kramer, 2008).

We utilized the data from boring B-02 and the ground motions for the MCE described above. The results of our analyses indicate that liquefaction is likely to occur from the ground surface to a depth between 5 and 10 feet below the level of Rolling Bay Walk NE (Elevation 5 to 0 NAVD88). A graphical summary of our liquefaction analyses is shown in Appendix C. The specific hazards associated with the predicted liquefaction are discussed below.

Seismic Settlement

Liquefaction-induced settlements were estimated using the recommended methods of Kramer (2008). We estimate that seismic settlements as the result of the predicted liquefaction of the Site soils could be up to 1 inch. Due to the stratigraphic variation across the Site, we estimate that differential seismic settlement across the building footprint could equal the total seismic settlement. It is our opinion that the seismic settlement hazard can be sufficiently mitigated through the use of specific foundation considerations; see Section 4.6, *Foundations* for specific recommendations.

Lateral Spreading

Lateral spreading is the horizontal movement of surficial soil down gentle slopes towards a body of water or towards a free face due to liquefaction. Because liquefaction of the underlying soil appears likely in the event of the design earthquake, and the sloping topography of the Site, there is potential for seismically-induced lateral spreading to occur.

The existing concrete bulkhead at the shoreline of the Site will provide some support to the slope; however, the age and condition of the bulkhead suggest it was not designed for current seismic considerations. Accordingly, our evaluations did not consider the bulkhead to protect or buttress the Site from lateral spreading.

We estimated horizontal displacement due to lateral spreading using the Site topography and our characterization of the subsurface conditions based on our explorations and lab testing. Using the methods developed by Youd et. al (2002) and Kramer and Baska (2009) within the WSliq computer program, we estimate that relatively minor horizontal displacements, ranging from negligible movement to a couple inches, could occur during the MCE.

It is our opinion that the lateral spreading hazard can be sufficiently mitigated through the use of specific foundation considerations; see Section 4.6, *Foundations*, for specific recommendations.

Reduction in Soil Shear Strength

Liquefied soil will temporarily experience a reduction in shear strength immediately following seismic shaking. This is likely to occur within the liquefiable soils at the Site. A discussion of the potential impacts on the global stability of the Site and steep slope is presented in Section 3.2.1, *Slope Stability*.

3.1.3 Seismically Induced Landslides

The dense to very dense (glacially overridden) advance outwash and glacial till deposits that comprise the majority of the steep slope and the results of the previously completed slope stability analyses (Perrone Consulting, Inc., 2016) that we reviewed indicate that the steep slope has a low risk of large-scale, deep-seated rotational landslide activity during an earthquake. Movement and landsliding of the surficial soils near the face of the steep slope during an earthquake is likely, but the landslide debris catchment wall system appropriately mitigates the hazard.

3.1.4 Surficial Ground Rupture

The nearest known fault to the Site is the Seattle Fault Zone. The Site is located approximately 3 miles north of the Seattle Fault Zone. The most recent large earthquake on the Seattle fault occurred about 1,100 years ago (Pratt et al., 2015). Due to the suspected long recurrence interval, and the proximity of the Site from the mapped fault trace, the potential for surficial ground rupture at the Site is considered low during the expected life of the improvements.

3.2 Landslide Hazard

The landslide hazard at the Site and subsequent mitigation is well documented in the previous studies (GeoEngineers, Inc., 2005, 2006, 2009 and Perrone Consulting, Inc., 2016). The Site is mapped as a greater than 40 percent steep slope geologically hazardous area and has known landslide history.

Landslides may be triggered by natural causes such as precipitation, freeze-thaw cycles, seismic event, or be man-made (e.g., broken water pipes or stormwater flow). Three types of landslides (Varnes, 1978) are common on steep slopes and bluffs in the Puget Sound: topples, deep-seated rotational slides, and shallow flows.

Topples

Tension cracks that are naturally present in soils near the face of steep slopes and bluffs may provide conduits for surface water migration and flow, and also promote growth of tree roots that can extend many feet downward into these cracks. As the roots grow and the face of the bluff progresses through freeze-thaw cycles, these cracks often become failure planes and a slab of soil will spall or topple off the slope or bluff face.

We did not observe tension cracks or evidence of recent/incipient topple activity on the steep slope during our reconnaissance. However, due to the near vertical inclinations on the steep slope, the possibility of topples does exist. Topples can impact the outer slope up to several feet but do not typically impact the underlying soils or overall stability of the steep slope. At the Site, topples would occur upslope of the landslide debris catchment wall system and the hazard of topples to the proposed residence is suitably mitigated by the wall system.

Deep-Seated Rotational Landslides

Rotational landslides consist of deep-seated failures that typically involve slip along a curved surface(s). Rotational landslides may transport large masses of semi-intact soil downslope, resulting in alternating steep headscarps along the upper portion of the failure, with more gently sloping benches composed of displaced soil.

We did not observe indicates of historical, recent, or incipient deep-seated rotational landslide activity at the Site. Based on the dense to very dense advance outwash deposits and glacial till deposits that comprise the majority of the steep slope, it is our opinion that the risk of large-scale, deep-seated rotational landslide activity is relatively low at the Site.

Shallow Flow Landslides

Flow-type landslides typically consist of movement of the upper colluvial soil layer and vegetation that typically mantles steep slopes. Flows are commonly triggered from a significant increase in the moisture content within the upper soil layer, typically from extended periods of heavy precipitation, near-surface groundwater seepage, or concentrated stormwater discharge onto the slope surface. Shallow flow failures on tall, steep slopes can generate large quantities of fast moving soil and pose a hazard to structures, roads, infrastructure, and the public.

The vegetation patterns, general steepness, presence of loose landslide deposits on the face of the steep slope, and documented landslide history indicate that there is a moderate to high risk of future shallow flow landslide activity on the steep slope. Any shallow failures would likely be limited to the upper soil and would not affect the global slope stability of the steep slope. Like with the topple hazard, shallow flow failures are most likely to occur above the landslide debris catchment wall and the hazard of shallow flow landslides to the proposed residence is suitably mitigated by the wall system.

3.2.1 Slope Stability

Based on our review of the design and construction documents related to the landslide debris catchment wall system and the results of our subsurface explorations and reconnaissance, it is our opinion that the topple and shallow flow landslide hazard to the proposed residence has been appropriately mitigated at the Site. The catchment wall system is reasonable and consistent with local engineering practice for these types of landslide mitigation and appears to have been well-constructed and is performing satisfactorily.

Similarly, we reviewed the slope stability analyses completed by Perrone Consulting, Inc. (2016) and found the analyses to be accurate. The results of our subsurface explorations confirm the stratigraphy, soil engineering properties, assumed groundwater level, and model configuration used in the previous analysis. The psuedostatic seismic coefficient used in the seismic analyses is roughly one half of the PGA for the MCE under the current version of the IBC, consistent with typical seismic slope stability analyses for residential projects.

One condition that was not directly considered in the previous slope stability analyses is the post-earthquake condition with residual (reduced) shear strengths within the liquefiable soils at the Site. Due to the location of these soils beyond the toe of the steep slope, their shear strength characteristics do not have a significant effect on global stability. Their bulk weight and buttressing is their primary contribution to the global stability analysis and that will be relatively unchanged during the post-earthquake condition.

The results of the previously completed slope stability analyses meet or exceed the minimum factors of safety required by the City and conservatively ignore any buttressing/stabilizing effects that the proposed residence and shoring walls would provide in the development of the Site.

4 Conclusions and Recommendations

4.1 General

Based on our geotechnical evaluation of the Site that included data review, Site reconnaissance, subsurface explorations, and geotechnical engineering analyses, it is our opinion that the proposed project will not negatively affect the global stability of the geologically hazardous area (steep slope) located at and near the Site and it can be constructed in a relatively stable manner provided the recommendations contained in this report are incorporated into the design and construction of the project. The key findings and conclusions include:

- The landslide debris catchment wall system is appropriate mitigation for the landslide hazard at the Site and can provide an acceptable level of protection to the proposed residence.
- We agree with the slope stability analyses completed by others that indicates the factors of safety for global stability at the Site for static and seismic conditions meet or exceed 1.5 and 1.0, respectively.
- The subsurface conditions beneath the majority of the proposed residence footprint consist of loose sand and gravel landslide deposits, overlying medium dense sand and gravel beach deposits, and dense sand advance outwash deposits. Subsurface conditions at the southwestern-most extent of the proposed residence consist of sand and gravel landslide debris directly overlying dense sand advance outwash deposits.
- Site-wide grading to create a flat building area and for the partially below-grade parking level will require excavation into the toe of the steep slope and a shoring wall along the southwest extent of the proposed residence. Shoring walls may also be needed along the northwest and southeast extents of the proposed residence, depending on the final building footprint and proximity to adjacent property lines and structures.
- The shoring wall(s) may be incorporated into the proposed residence as a permanent below-grade wall and/or foundation support.
- The landslide deposits beneath the majority of the proposed residence footprint contains debris, voids, and are prone to settlement and liquefaction hazards. Therefore, a deep foundation system is recommended with supporting elements driven or drilled into the underlying dense advance outwash sand to bypass the landslide deposits and provide adequate support for the residence.
- For exterior flatwork and landscaping walls, we recommend the use of flexible systems like pavers, modular blocks or rockeries, instead of cast-in-place concrete.

The grading and final development plans for the project had not been completed when this report was prepared. Once completed, Aspect should be engaged to review the project plans to confirm our recommendations have been appropriately incorporated and/or to update our recommendations as necessary.

4.2 Steep Slope Management

The most likely impact from slope instability to the Site would be shallow flow landslides triggered by saturation of the near-surface soils. These failure-types are typically limited to the outer several feet of soil on the steep slope. Table 1 includes factors that can affect stability of the near-surface soil layer:

Table 1. Vegetation Related Stabilizing and Destabilizing Factors for Slope Management (Gray and Leiser, 1982)

Stabilizing Factors	Destabilizing Factors
Root Reinforcement Roots mechanically reinforce a soil by transfer of shear stresses in the soil to tensile resistance in the roots.	**Surcharge** Weight of vegetation on a slope exerts both a downslope (destabilizing) stress and a stress component perpendicular to the slope, which tends to increase resistance to sliding.
Soil Moisture Modification Evapotranspiration and interception in the foliage lower soil moisture content.	**Root Wedging** Alleged tendency of roots to invade cracks, fissures, and channels in a soil and thereby cause local instability by a wedging or prying action.
Buttressing and Arching Anchored and embedded stems can act as buttress piles or arch abutments in a slope, counteracting shear stresses.	**Windthrowing** Destabilizing influences from an overturning moment exerted on a slope as a result of strong winds blowing downslope through trees.

Other causes of surficial slope instability include improperly managed storm and surface water runoff flowing near or over the top of the slope. Uncontrolled runoff or surface water should never be allowed to flow across the Site slopes.

We recommend maintaining dense vegetative groundcover on the Site slopes outside of the building area. If soils on or near the slopes become exposed through erosion and/or shallow failures, we recommend immediately covering and aggressively revegetating the exposed area. This may require the placement of plastic sheeting replaced by a woven jute-mat to provide temporary ground cover while vegetation takes root.

For specific vegetation recommendations, Ecology has several good publications on the subject including:

- Vegetation Management: A Guide for Puget Sound Bluff Property Owners, Ecology Publication 93-31, at http://www.ecy.wa.gov/programs/sea/pubs/93-31/intro.html.

- Slope Stabilization and Erosion Control Using Vegetation: A Manual of Practice for Coastal Property Owners, Ecology Publication 93-30, at http://www.ecy.wa.gov/programs/sea/pubs/93-30/index.html.

We recommend working with your neighbors to identify and monitor the function of any active drainage pipes on the steep slope. If possible, work to identify alternatives to drain lines on the steep slope.

It is important to note that the landslide debris catchment wall system requires the removal of any landslide debris that accumulates behind in order to operate effectively and to provide the intended landslide mitigation for the proposed residence. In the event of a landslide, debris should be removed from behind the wall system to allow for catchment of debris from future landslide events. The wall system should also be inspected and repaired, as necessary, annually and after each landslide event. We recommend annual inspection of the wall system each fall prior to the wet winter season.

4.3 Earthwork

It is our opinion that basic excavation and grading necessary for the project can be completed with standard construction equipment. The contractor should be prepared to excavate oversized debris associated with the previous landslide activity and subsequent demolition/removal of the previous residence. A layer of quarry spalls was reportedly placed over the lower steep slope to facilitate construction of the landslide debris catchment wall and should be expected during excavation. Excavation activities would be significantly easier and more cost-effective during the drier summer months. Appropriate erosion control measures should be implemented prior to beginning earthwork activities in accordance with the local regulations.

4.3.1 Wet Weather Earthwork

If earthwork is to be performed in wet weather or under wet conditions when soil moisture content is difficult to control, the following recommendations apply:

- Track-mounted equipment may be required.
- Earthwork should be performed in small areas to minimize exposure to wet weather. Excavation or the removal of unsuitable soils should be followed promptly by the placement and compaction of clean structural fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance.
- The ground surface within the construction area should be graded to promote runoff of surface water and to prevent the ponding of water.
- The ground surface within the construction area should be covered or sealed, and under no circumstances should be left uncompacted and exposed to moisture. Soils that become too wet for compaction should be removed and replaced with clean granular materials.
- Cut and fill slopes should be covered/protected by plastic sheeting or equivalent to help reduce the potential for erosion and sloughing.

- Reuse of on-Site soil for structural fill may not be feasible due to the high fines content and moisture sensitivity of the Site soils.
- Excavation and placement of fill should be observed by Aspect to verify that all unsuitable materials are removed and suitable compaction and Site drainage is achieved.
- Appropriate erosion and sedimentation best management practices (BMPs) should be strategically implemented in accordance with City BMPs.

4.3.2 Site Preparation

Site preparation within the proposed construction area footprint should include removal of all debris and any other deleterious material. Additionally, all soils with significant root debris within the proposed building footprint and areas to receive structural fill should be removed. The contractor must use care during Site preparation and excavation operations so that any bearing surfaces are not disturbed. If this occurs, the disturbed material should be removed to expose undisturbed material or compacted in-place.

We recommend that excavations and subgrade preparation for the project be observed by Aspect to verify that the recommendations of this report have been followed.

If excavations are open during the winter season or periods of wet weather, we recommend providing a layer of crushed rock or gravel to help preserve the subgrade until structural fill or concrete is placed.

4.3.3 Structural Fill

Material excavated for the project may be suitable for use as structural fill, provided it is relatively free of organics and other deleterious materials and can be moisture-conditioned to meet the compaction requirements for structural fill. Material excavated that is not suitable for use as structural fill may be used as fill in landscape areas or other areas not sensitive to settlement.

Imported material for structural fill should be granular material with less than 10 percent fines such as Select Borrow as specified in Section 9-03.14(2) of the WSDOT Standard Specifications, similar to well-graded pit run. In wet weather conditions or situations requiring free-draining backfill, material meeting the criteria for Gravel Borrow as specified in Section 9-03.14(1) of the WSDOT Standard Specifications should be imported for use as fill. Crushed Surfacing Base Course as specified in Section 9-03.9(3) of the WSDOT Standard Specifications, similar to 1.25-inch minus crushed rock, should be used as structural fill under exterior flatwork or auxiliary structures not supported by deep foundations, such as landscape retaining walls.

In general, suitable structural fill material for the project is fill placed within 3 percent of its optimum moisture content per ASTM D-1557 (modified Proctor test) and does not contain deleterious materials or particles larger than 3 inches in diameter. Structural fill material should be compacted to a minimum of 95 percent of the maximum dry density (MDD) based on ASTM D-1557. Structural fill adjacent to a wall should be compacted to a minimum of 90 percent of the MDD based on ASTM D-1557.

4.4 Temporary and Permanent Slopes

Maintaining safe working conditions, including temporary excavation stability, is the responsibility of the contractor. All temporary cuts in excess of 4 feet in height that are not protected by trench boxes, or otherwise shored, should be sloped in accordance with Occupational Safety and Health Administration (OSHA) guidelines.

In general, the Site soils classify as OSHA Soil Classification Type C. Temporary excavation side slopes (cut slopes) are anticipated to stand as steep as 1.5H:1V (Horizontal:Vertical) within the landslide deposits up to a maximum height of 20 feet (with no backslope). For cut slopes greater than 20 feet tall or those with a significant backslope, the slopes should be reduced to 2H:1V or flatter or appropriately shored. The cut slope inclinations estimated above are for planning purposes only and are applicable to excavations without inflowing perched groundwater or runoff. Surcharge loads such as construction equipment or material stockpiles should be maintained behind the cut slopes a minimum distance equal to the height of the cut.

With time and the presence of seepage and/or precipitation, the stability of temporary unsupported cut slopes can be significantly reduced. We recommend planning the construction schedule to have excavation occur during the summer months and to minimize the amount of time that the temporary slopes will be unsupported during construction. The contractor should monitor the stability of the temporary cut slopes and adjust the construction schedule and slope inclination accordingly. Vibrations created by traffic and construction equipment may cause caving and raveling of the face of the temporary slopes. Cut slopes made to construct footings or other elements requiring worker foot traffic should be sloped or shored as necessary to prevent soil from raveling into the excavation.

Permanent fill slopes for the project should be no steeper than 2H:1V. Permanent cut slopes for the project should be no steeper than 2H:1V.

4.5 Shoring

Shoring is anticipated for the excavation into the toe of the steep slope. We understand the lower level of the structure will be used for parking and will be near the elevation of the adjacent Rolling Bay Walk NE. This will require an excavation (cut) on the order of 13 to 15 feet tall along the southwestern edge of the building footprint. Shoring is required to temporarily retain the Site soils to accommodate the planned excavation and associated construction activities. Depending on the final layout of the building footprint and whether sufficient space is available for sloping the excavation cuts, shoring walls may also be required along the northwestern and southeastern edges of the excavation. The shoring may be incorporated as permanent walls for the proposed residence and/or as foundation support.

For shoring at the Site, we recommend soldier pile walls.

4.5.1 Soldier Pile Wall Design Criteria

The soldier piles should be drilled and cast-in-place, and we assume they will be cantilevered or anchored with one row of tieback anchors. Based on our characterization of the Site and our observations, we anticipate the walls will support and retain loose landslide deposits. The embedded portion of the walls will be drilled into and gain support from medium dense beach deposits and dense advance outwash deposits.

We recommend a maximum center-to-center spacing of approximately 8 feet between individual soldier piles. We recommend a minimum pile embedment below the base of the planned excavation of 8 feet to achieve an adequate global factor of safety for the wall system and/or to prevent “kickout” of the base of the piles. Detailed design of the shoring wall may require additional pile embedment to provide sufficient passive earth pressure support for the system.

We recommend designing the soldier pile wall based on the following earth pressures:

Table 2. Summary of Earth Pressures for Static Design of the Soldier Pile Shoring Walls

Wall Segment	Elevation Range(1)	Active Earth Pressure(2)(3) (pcf)	Passive Earth Pressure(3) (pcf)
Parallel to Slope Contours	Above Elevation 5	85	350
	Below Elevation 5		150
Perpendicular to Slope Contours	Above Elevation 5	40	250
	Below Elevation 5		150

Notes: pcf = pounds per cubic foot.

1) NAVD88. Elevation 5 is the design groundwater level.
2) Presented as an equivalent fluid pressure, assumes a cantilever wall system or a single row of tieback anchors.
3) Assumes a maximum backslope of 30 degrees behind the wall and ignores any passive resistance from the tide-flat deposits in front of the wall.

The recommended earth pressures are equivalent fluid earth pressures that assume the wall is allowed to yield slightly and invoke the “active” condition. The use of free-draining lagging and wall facing is anticipated such that there will be no buildup of unbalanced hydrostatic pressures behind the wall.

The active earth pressures act over the full soldier pile spacing above the base of the wall and over the soldier pile hole diameter (B) below the base of the wall. The passive pressures act over two and half times the soldier pile hole diameter (2.5B) for the embedded portion of the wall. The passive resistance provided is allowable and includes a factor of safety of 1.5.

The recommended earth pressures may be reduced by 50 percent for the design of the lagging due to soil arching effects between the soldier piles.

The southwestern shoring wall will be located approximately 15 feet (horizontally) northeast of the lower wall of the existing landslide debris catchment wall system. To ensure minimal effect and interaction between the shoring for the proposed residence and the existing landslide debris catchment wall system, we recommend top-down construction of the shoring and methods to minimize soil loss (caving or sloughing) during drilling of the soldier piles and subsequent lagging between the soldier piles, as described in Section 4.5.3, *Soldier Pile Wall Construction Considerations*. Otherwise, we understand the existing lower wall does not rely significantly on passive resistance (GeoEngineers, 2010) and achieves the majority of its lateral restraint through the tieback anchors. Considering the configuration of the lower catchment wall, it is our opinion that the risk of adverse impact from a downslope shoring wall can be suitably mitigated through incorporation of the construction considerations described in Section 4.5.3, Soldier Pile Wall Construction Considerations. We have also recommended an elevated active lateral earth pressure for the proposed shoring to reflect the loose retained soils and anticipated backslope condition.

If the soldier pile shoring wall is to be incorporated into the proposed residence as a permanent wall, we recommend a seismic lateral earth pressure equal to 23H (where H is the exposed height of the wall) be applied to account for a temporary lateral seismic soil pressure increment for the MCE. The seismic lateral earth pressure was calculated using limit equilibrium modeling and methods presented by the National Cooperative Highway Research Program (NCHRP, 2008). Using the modeling software package SLIDE, a force balance of the active wedge of soil behind the wall was modeled considering the soil types present, backslope geometry, and ground shaking representing the MCE and a PGA of 0.55g.

If the soldier pile shoring wall is to be permanent, the soldier piles should be epoxy coated (or equal) for corrosion protection a minimum of 2 feet below the projected final grades. Alternatively, the piles may be oversized to compensate for loss of material over the wall design life.

If inclined tie-back anchors are utilized for support of the soldier pile wall or a concrete facing is installed, they could impose a vertical load on the soldier piles. These loads will be resisted by a combination of end bearing and side friction from the embedded portion of the soldier pile. Side friction for the portion of the piles above the embedded portion should be neglected. In determining the resistance of the piles to vertical loads, we recommend assuming an allowable end bearing value of 10,000 psf. Side friction should be taken as 500 psf (allowable) starting at a depth of 10 feet below the embedded portion of the soldier pile.

4.5.2 Tieback Anchors

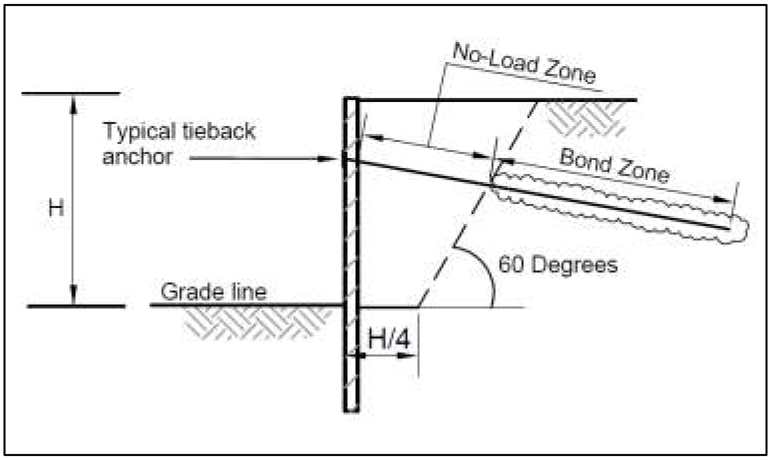
Tieback anchors or permanent ground anchors can be used to provide additional horizontal support to the soldier pile wall. Tieback anchors typically consist of a drilled hole ranging from 4 to 8 inches in diameter with a steel tendon encapsulated in grout. The tendon is secured to a steel plate or waler at the face of the wall and actively prestressed to limit deflections. Ultimate pullout capacity of grouted anchors is dependent on the diameter and length of the bond zone (where the grout is in direct contact with the soil).

For preliminary planning and design purposes, we recommend assuming an allowable tieback adhesion of 1,500 psf for any portion of the bond zone. This recommended value assumes low pressure (gravity) grout injection methods. The adhesion value should be verified in the field using a grouted anchor testing program, as described below. We recommend a minimum installation inclination of 15 degrees below horizontal to help ensure gravity flow of the grout to the base of the drilled hole. Tieback anchors should be spaced a minimum of 4 feet apart, assuming a maximum anchor diameter of 6 inches.

The tieback anchors should be designed to withstand a moderately corrosive environment over their design life. Corrosion protections should follow the recommendations presented by WSDOT (2016).

Potential conflicts between new tieback anchors and the existing soldier piles for the catchment wall system should be investigated and account for in the design and layout of any anchored shoring system.

Tieback-anchor bond zones should be entirely beyond the "no-load" zone illustrated on the inset figure below. Tiebacks should carry no loads within the "no-load" zone. A bond-breaker should be utilized between the anchor tendons and the grout in the "no-load" zone.



Tieback Anchor No-Load Zone Diagram

Tieback anchor testing should be performed in general accordance with the recommendations presented by WSDOT (2016). All of the tieback anchors should be proof-tested to 100 percent of the design load. At least two anchors should be verification tested to 150 percent of the design load at locations selected by the geotechnical engineer. Testing should be accomplished using the same equipment that will be used for all production anchors. Testing loads should not exceed 80 percent of the ultimate tensile strength of the tendon. The design lock-off load should be 80 percent of the tieback-anchor design load to minimize wall deflections. Lift-off tests should be performed at the geotechnical engineer's request to verify the proper lock-off load.

An anchor should be deemed acceptable, if it meets the following criteria:

- The total elastic movement obtained from the verification tests and proof tests exceed 80 percent of the theoretical elastic elongation of the nonbonded length.
- Total anchor movement between the 1- and 10-minute intervals should not exceed 0.04 inches, regardless of the tendon length or load. Total anchor

movement between the 6- and 60-minute intervals (if required) should not exceed 0.08 inches.

- The "lift-off measurement" indicates an anchor load within 5 percent of the design lock-off load.

4.5.3 Soldier Pile Wall Construction Considerations

All soldier pile installation activities should be observed by Aspect. If permanent wood lagging is used, the wood should be treated with preservatives to limit degradation over time.

The soldier piles will penetrate relatively loose landslide deposits and dense advance outwash deposits. The landslide deposits may contain obstructions such as, buried logs, stumps, or oversized debris associated with the previous residence at the Site. Groundwater seepage should be anticipated at or above the interface between the landslide deposits and underlying advance outwash deposits. The contractor should be prepared to drill soldier piles such that caving is prevented.

Preventative measures to mitigate for caving and hole collapse for a drilled scenario may include temporary casing and/or drilling fluids. If groundwater is encountered and/or drilling fluids are used, we recommend concrete be placed with a tremie pipe. Soldier piles with center-to-center spacing of less than three pile diameters should not be drilled in sequence. Rather, every other pile should be drilled, and the concrete should be placed and allowed to cure at least 24 hours before adjacent piles are drilled. Lean concrete may be used to backfill the soldier pile shafts unless the soldier piles are used to support axial foundation loads, then structural concrete should be used.

Once the soldier piles have been installed and accepted by the geotechnical engineer, excavation may begin. Excavation should proceed along the entire face of the wall to approximately the same elevation and should not exceed an open-face depth of 5 feet before installing lagging. After lagging is installed, excavation should not exceed an open-face depth of 2 feet below the planned tieback elevations to limit wall deflection. Upon completion of the tieback anchors installation and successful testing and lock-off of the tieback anchors, excavation should proceed along the entire face of the wall to approximately the same elevation to the final grade.

Any voids behind the lagging should be replaced with self-compacting and free-draining materials such as pea gravel. For a wall facing system, drainage should be provided for to avoid the buildup of unbalanced hydrostatic pressure behind the wall.

Monitoring

A monitoring program should be developed to verify the performance of the wall system during construction. Monitoring will be used as a basis for providing an early warning for possible problems and allow for mitigation measures to correct problems that may be encountered.

Optical surveying targets should be established for measuring horizontal and vertical displacements on the tops of selected soldier piles. Measurement techniques should be accurate to the nearest 0.01 feet for vertical and horizontal displacements.

Targets on the soldier piles should be established after they are installed and prior to excavation and grading of the downhill slope. Targets should be measured twice weekly during excavation activities until the final grade of the downhill slope is achieved. Once the complete shoring system is in place, measurements should occur twice-monthly until the project is completed.

Action level should be developed in the specifications for appropriately protecting the adjacent catchment wall system and stability of the steep slope. The contractor should submit a contingency plan to implement, if action levels are exceeded.

4.6 Foundations

The majority of the Site is underlain by loose landslide deposits that contain debris, voids, and are prone to settlement and liquefaction. Therefore, a deep foundation system is recommended with supporting elements driven or drilled into the underlying dense advance outwash sand to bypass the landslide deposits and provide adequate support for the residence.

Due to the liquefaction hazard, the area of the proposed residence may deform towards the shoreline during the MCE due to lateral spreading. Resisting and fully mitigating this lateral spreading deformation would require significant foundation elements including large diameter concrete shafts and possibly additional permanent ground anchors for lateral restraint. Full mitigation of liquefaction induced lateral spreading is often a cost-prohibitive scenario for residential development.

As an alternative, less robust deep foundation systems may be used to partially mitigate the lateral spreading (and other liquefaction related hazards) with the primary goal of maintaining the structural integrity of the residence such that life safety is preserved during and following the MCE. Under this approach, the anticipated earth movements would not be fully resisted and the residence would likely require structural and cosmetic repairs following a large earthquake. In our opinion support of the residence with small diameter steel “pin” piles and grade beams as the primary foundation system would achieve the goal of preserving structural integrity and life safety during the MCE.

Design considerations for pin piles are provided below. If desired, Aspect can provide detailed design considerations for a more robust cast-in-place concrete shaft foundation system.

4.6.1 Pin Piles

Normal pin piles typically consist of 2- to 6-inch-diameter steel pipe piles driven to refusal using a pneumatic or hydraulic hammer. Smaller 2-inch-diameter pin piles can be driven with a standard 90-pound jackhammer while larger diameter pin piles typically require a machine-mounted hammer. Refusal criteria varies by the diameter of the pin pile, but is typically defined as less than 1-inch of penetration into the ground during a specified time period of continuous driving with the specified hammer. Specific refusal and load capacity information is shown below in Table 3.

Table 3. Pin Pile Capacities and Refusal Criteria

Pin Pile Diameter (in)	Hammer Weight$^{(1)}$ (lbs)	Allowable Capacity$^{(2)}$ (kips)	Refusal Criteria$^{(3)}$ (s)$^{(4)}$
2	90	4	60
3	550	12	12
4	850	20	16
6	1,100	30	20

Notes:
1) Minimum hammer weight recommended.
2) Includes a factor of safety of 2.0.
3) Time to drive pile less than 1 inch during continuous driving.
4) (s) = Second.

If a larger hammer than what is indicated in Table 3 is utilized to drive the piles, an alternate refusal criteria (different from above) may be needed. For pin piles greater than 2 inches in diameter and for 2-inch-diameter piles that exceed 30 feet in length, the piles should be load tested in general accordance with the Quick Load Test Method described in American Society for Testing and Materials (ASTM) ASTM D-1143-81. The Quick Load Test Method may also be used to calibrate alternate driving refusal criteria.

Pin piles should be utilized for compressive support only and battered pin piles may be required to provide lateral support. The maximum design batter for pin piles should be limited to about 15 degrees based on our experience with local pile driving contractors' equipment limitations.

Based on the results of our explorations, we anticipate that small diameter pin piles will achieve practical refusal when embedded approximately 10 feet into the very dense advance outwash deposits at the Site. This results in estimated pile lengths ranging from 15 to 30 feet long with longer piles required along the eastern edge of the residence footprint. We estimate settlement of pin piles installed to the refusal criteria described above will be less than ½ inch for long-term static loading conditions.

4.6.2 Foundation Lateral Resistance

Lateral forces may be resisted through the use of battered pin piles, as described above, and/or through passive earth pressure against the buried portions of the structures. Lateral resistance will be impacted by the presence of liquefiable beach deposits adjacent to the buried portions of the structures. Whether or not the effects of liquefaction will occur concurrently to inertial shaking forces associated with the design earthquake is dependent on a number of factors including the duration of the design earthquake, and density and extent of the liquefiable soil.

Guidance from the WSDOT Geotechnical Design Manual (GDM) cites the best available science and suggests that for short-duration ground motions, liquefaction is typically triggered near the end of the inertial shaking and lateral loading from the inertial shaking can be considered separately (de-coupled) from the effects of liquefaction. Short-duration earthquakes are considered those with a magnitude 7.5 or less (WSDOT, 2015). At the Site, the deaggregation for the MCE earthquake indicates that the modal contributing event to the deaggregation is a 7.09 magnitude earthquake, and the mean contributing event is a 7.2 magnitude earthquake (USGS, 2016). Therefore, we recommend the foundation design consider the lateral forces from inertial shaking separate from the effects of liquefaction. Passive earth pressures may be relied upon before and during the design earthquake event. Following an earthquake event, minor grading and backfilling may be required to fill any developed voids against the buried portions of the structures to reestablish the soil-structure contact for future passive earth pressure resistance.

We recommend an allowable passive earth pressure of 200 pounds per cubic foot (pcf) for use in the foundation design.

4.6.3 Floor Slabs

We anticipate concrete slabs will be structural slabs relying on support from grade beams and deep foundations. To provide a capillary break, we recommend the floor slab be underlain by a capillary break section. The capillary break material should consist of a minimum of 6 inches of free-draining, crushed rock or well-graded sand and gravel compacted to at least 95 percent MDD (ASTM D1557). The capillary break material should have a maximum particle size of 3/4 inch, with no more than 80 percent passing the No. 4 sieve and less than 5 percent fines (material passing the U.S. Standard No. 200 sieve). In areas where moisture will be detrimental to floor coverings or equipment inside the proposed structures, a 10-mil polyethylene vapor barrier should be placed directly over the capillary break. The vapor barrier should be installed in accordance with the manufacturer’s recommendations.

4.6.4 Cast-in-Place Concrete Walls

We anticipate that cast-in-place concrete walls may be used for the lower level of parking/basement. Nonyielding or restrained walls, such as basement walls, should be designed for an equivalent fluid weight of 60 pcf. A level backslope and adequate drainage is assumed for the recommended earth pressure value. For nonlevel backslopes, up to a maximum of 30 degrees, we recommend an equivalent fluid weight 105 pcf.

Earthquake shaking will subject concrete retaining walls to a temporary additional earth pressure. We estimated the lateral seismic soil pressure increment using the Mononobe-Okabe method (for level backslope conditions), with consideration of the possible backfill soil properties and MCE. For level backslope conditions, we recommend an average seismic soil pressure increment of 10H (where H is the height of the wall) represented by a uniform rectangular pressure along the height of the wall. For non-level backslope conditions, up to a maximum of 30 degrees, we recommend an average seismic soil pressure increment of 23H.

Overcompaction of the backfill behind walls should be avoided. In this regard, we recommend compacting the backfill to about 90 percent of the MDD (ASTM D1557). Heavy compactors and large pieces of construction equipment should not operate within 5 feet of any embedded wall to avoid the buildup of excessive lateral pressures. Compaction close to the walls should be accomplished using hand-operated vibratory plate compactors.

Lateral forces that may be induced on the wall due to other surcharge loads should be considered by the structural engineer.

4.7 Drainage and Moisture Considerations

Final grades around the proposed residence should be sloped such that water drains away from the structure. Water from hard surfaces should be collected and diverted to the stormwater outfall system. Downspouts and roof drains should not be connected to the foundation drains and under-slab drains in order to reduce the potential for flooding foundation drains and clogging. Drains should include clean-outs to allow periodic maintenance and inspection.

The outside edge of all perimeter footings and upslope side of all walls should be provided with a drainage system consisting of 4-inch diameter, perforated, rigid plastic pipe embedded in a clean, free-draining sand and gravel meeting the requirements of Section 9-03.12(4) of the WSDOT Standard Specifications (WSDOT, 2016) for Gravel Backfill for Drains. The drainpipe and surrounding drain rock should be wrapped in filter fabric to minimize the potential for clogging and/or ground loss due to piping. A washed rock drain curtain at least 1 foot-thick should extend from the footing continuously upward to within 1 foot of the ground surface. A layer of low permeability soils should be used on the upper foot to reduce potential for surface water to enter the drain curtain.

If the soldier pile shoring is used as a permanent wall, a prefabricated drain mat should be installed at the face of the soldier pile wall prior to casting the concrete wall. The drain mat should be hydraulically connected to a conveyance pipe for direction to the primary Site stormwater system and outfall.

We recommend the garage floor slab be underlain by a sub-slab drainage system consisting of perforated, 4-inch-diameter, polyvinyl chloride (PVC) pipes placed in the capillary break with a minimum spacing of 10 feet center-to-center.

Drainage should be directed away from the slope and into an appropriate containment system for conveyance from the Site.

The sloping topography and relatively shallow groundwater conditions indicate the Site is not suitable for concentrated stormwater infiltration. We recommend that all onsite stormwater be collected and conveyed a suitable discharge location as determined by the Civil Engineer. At no time should stormwater or runoff be allowed to collect or discharge onto the slope.

We are available to review and comment on the Site drainage plans as they are developed.

4.8 Septic System Considerations

We understand the proposed on-site septic system will include a drainfield located up slope and between the existing soldier pile catchment walls. This will require the pumping of effluent up slope to that location. Based on our geotechnical engineering judgement, this approach is feasible and will not increase the landslide hazard at the Site or surrounding areas provided the system is designed for the specific soil conditions at that location.

We recommend a system that is metered or ‘dosed’ for controlled flows and installed at shallow depths. We anticipate the effluent conveyance line will be buried for frost protection approximately 12 inches below the surface of the steep slope and we recommend the trench backfill consist of material that meets the requirements for on-Site structural fill, described above. The trench backfill should be compacted until relatively firm and unyielding and the surface of the backfilled trench should be left slightly higher than the surrounding ground surface to avoid concentration of surface runoff along the trench alignment. Trench dams should be incorporated in strategic locations to prevent shallow water from migrating through and along the trench backfill, as needed. The trench dam should consist of concrete, controlled density fill (CDF), bentonite, or similar low-permeability material.

We recommend the pump conveyance line be sleeved with a larger diameter high density polyethylene (HDPE) or thick-walled PVC pipe to provide added protection against rupture from tree falls on the steep slope. The system should also include an automatic shutoff feature to detect any loss in pressure and stop flow to the conveyance line and steep slope in the event of pipe failure.

The introduction of effluent water between the landslide debris catchment walls will incrementally raise the overall unit weight of the soil and result in minor increases in lateral earth pressures on the lower catchment wall. However, we understand the lower wall was designed to support surcharge loads from the heavy construction equipment needed to construct the upper catchment wall and it is our opinion that the lower wall can accommodate incremental additional pressure that could result from the introduction of effluent water to the retained soils. The wood lagging facing of the lower catchment wall is free-draining and will prevent the buildup of unbalanced hydrostatic pressures on the wall.

5 Additional Project Design and Construction Monitoring

At the time of this report, site plans, site grading, structural plans, and construction methods have not been finalized, and the recommendations presented herein are based on preliminary project information. If project developments result in changes to the assumptions made herein, we should be contacted to determine if our recommendations should be revised. We recommend that, once design plans are fully developed, Aspect is consulted in order to verify that our recommendations were properly interpreted and applied.

This report is issued with the understanding that the information and recommendations contained herein will be brought to the attention of the appropriate design team personnel and incorporated into the project plans and specifications, and that the necessary steps will be taken to verify that the contractor and subcontractors carry out such recommendations in the field. We do not direct the contractor's operations, and we cannot be responsible for the safety of personnel other than our own on the Site; the safety of others is the responsibility of the contractor. The contractor should notify the property owner if he considers any of the recommended actions presented herein unsafe.

We are available to provide geotechnical engineering and monitoring services during construction. The integrity of the foundation depends on proper site preparation and construction procedures. In addition, engineering decisions may have to be made in the field in the event that variations in subsurface conditions become apparent.

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7 Limitations

Work for this project was performed for Ms. Margaret Dufresne (Client) for specific application to the proposed project as described herein, and this report was prepared in accordance with generally accepted professional practices for the nature and conditions of work completed in the same or similar localities, at the time the work was performed. This report may be used only by the Client and for the purposes stated, within a reasonable time from its issuance. This report does not represent a legal opinion. No other warranty, expressed or implied, is made.

All reports prepared by Aspect Consulting for the Client apply only to the services described in the Agreement(s) with the Client. Any use or reuse by any party other than the Client is at the sole risk of that party, and without liability to Aspect Consulting. Aspect Consulting's original files/reports shall govern in the event of any dispute regarding the content of electronic documents furnished to others.

Recommendations presented herein are based on data that we acquired, our geotechnical engineering calculations, and judgment in accordance with our mutually agreed-upon scope of work. Variations may exist between soil and groundwater conditions reported, and those actually underlying the Site. The nature and extent of such soil variations may change over time and will not be evident before construction begins. If any soil conditions are encountered at the Site that are different from those described in this report, we should be notified immediately to review the applicability of our recommendations.

Our conclusions and recommendations made herein rely and take into account previous geotechnical reports at and near the Site and their associated data, analyses, and conclusions. Unless stated specifically above, we did not perform independent analyses or technical reviews to verify the accuracy of the data, analyses, and conclusions presented in the previous reports.

It is the Client's responsibility to see that all parties to this project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk.

Our scope of our work does not include services related to construction safety precautions. Our recommendations are not intended to direct the contractors' methods, techniques, sequences or procedures. Our scope of our work also excludes the assessment of environmental characteristics, particularly those involving potentially hazardous substances in soil or groundwater.

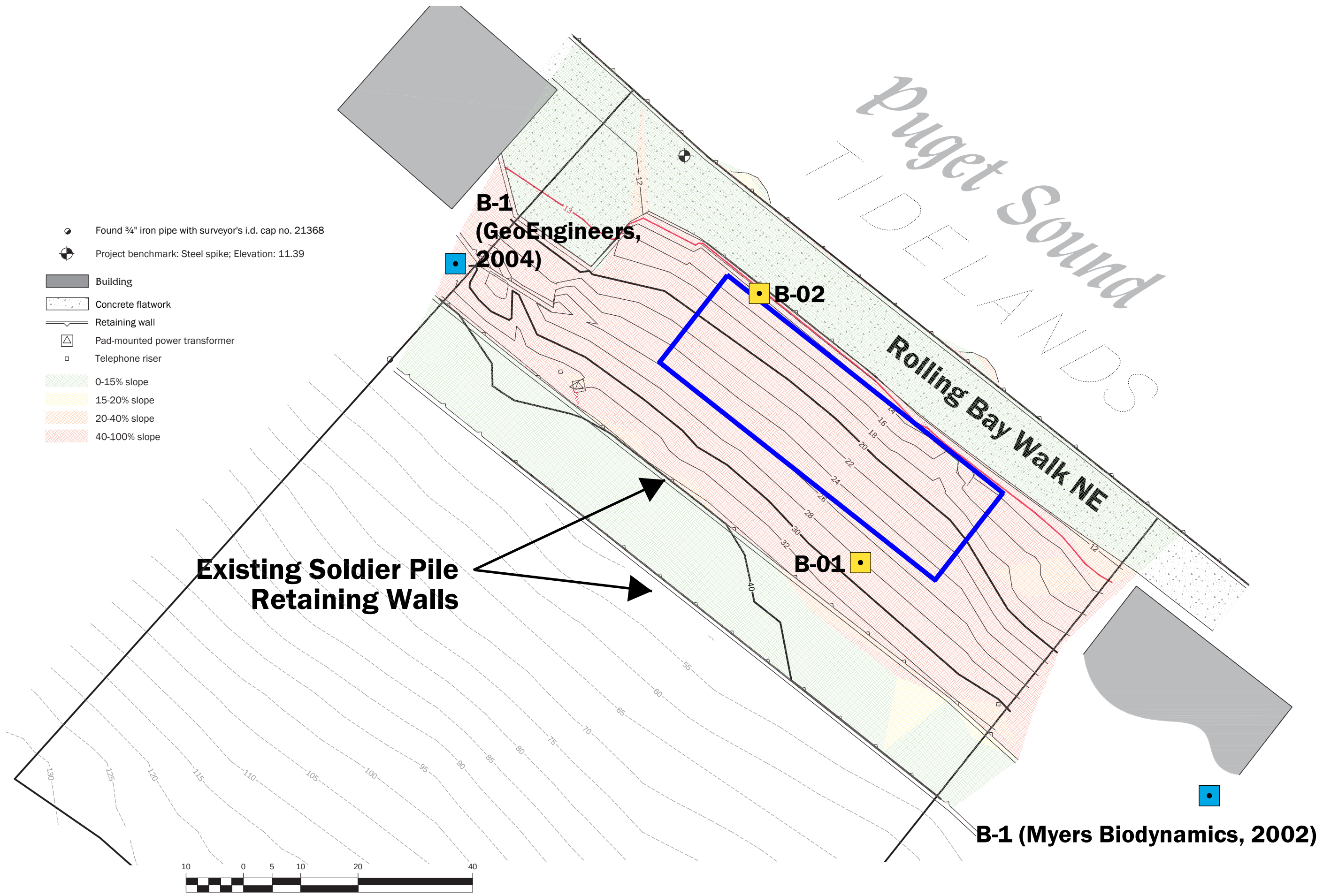
FIGURES

Site Location Map

Dufresne Residence
11143 Rolling Bay Walk NE
Bainbridge Island, Washington

Aspect CONSULTING	JUL-2017	BY: NHC / KES	FIGURE NO.
	PROJECT NO. 170202	REVISED BY: - - -	1

Basemap Layer Credits || Sources: Esri, HERE, DeLorme, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, MapmyIndia, © OpenStreetMap contributors, and the GIS User Community



Notes:
- Map Source is MacLearnsberry, Inc.
- Solid contours were ground-measured by MacLearnsberry, Inc. Dashed, gray contours are from Puget Sound LiDAR Consortium data and are included only to show the general magnitude of the slope beyond the ground-mapped area.
- Elevation 13 is the upper limit of the FEMA flood zone for this locale.
- Vertical Datum: NAVD88

Boring (Consultant, Year)

Aspect Boring

Proposed Residence

N

Site Exploration Plan

Dufresne Residence
11143 Rolling Bay Walk NE
Bainbridge Island, Washington

Aspect CONSULTING	JULY-2017	BY: EAC / NHC	FIGURE NO.
	PROJECT NO. 170202	REVISED BY: - - -	2

GIS Path: T:\projects_8\DufresneResidence_170202\Delivered\02 Site Plan Layout.mxd || Coordinate System: NAD 1983 StatePlane Washington North FIPS 4601 Feet || Date Saved: 5/18/2017 || User: ecrumbaker || Print Date: 5/18/2017

APPENDIX A

Soil Boring Logs

A. Field Exploration Program

A.1. Soil Borings

Two machine drilled borings, B-01 and B-02 were advanced on the Site on May 3, 2017. The machine drilled borings were advanced using hollow-stem auger methods by Boretec, Inc. under subcontract to Aspect using an Acker Drill that was equipped with a 140-pound cathead-safety hammer. Samples were obtained every 2.5 feet to 15 feet bgs and then every 5 feet to the depths explored using the Standard Penetration Test (SPT) in general accordance with ASTM Method D1586.

The SPT method involves driving a 2-inch-outside-diameter split-barrel sampler with a 140-pound hammer free-falling from a distance of 30 inches. The number of blows for each 6-inch interval is recorded and the number of blows required to drive the sampler the final 12 inches is known as the Standard Penetration Resistance ("N") or blow count. The resistance, or N-value, provides a measure of the relative density of granular soils or the relative consistency of cohesive soils. If a total of 50 blows are recorded for a single 6-inch interval, the test is terminated and the blow count is recorded as 50 blows for the total inches of penetration. Samples were placed in labeled plastic jars and taken to a laboratory for further classification.

The locations of explorations are shown on Figure 2 and were collected in the field using a global positioning system (GPS). The borings were backfilled with bentonite chips and capped with about 1 foot of excavated soils, in accordance with Washington State Department of Ecology regulations.

Group		Fines	Symbol	Description
Coarse-Grained Soils - More than 50% (1) Retained on No. 200 Sieve	Gravels - More than 50% (1) of Coarse Fraction Retained on No. 4 Sieve	≤5% Fines (5)	GW	Well-graded gravel and gravel with sand, little to no fines
			GP	Poorly-graded gravel and gravel with sand, little to no fines
		≥15% Fines (5)	GM	Silty gravel and silty gravel with sand
			GC	Clayey gravel and clayey gravel with sand
	Sands - 50% (1) or More of Coarse Fraction Passes No. 4 Sieve	≤5% Fines (5)	SW	Well-graded sand and sand with gravel, little to no fines
			SP	Poorly-graded sand and sand with gravel, little to no fines
		≥15% Fines (5)	SM	Silty sand and silty sand with gravel
			SC	Clayey sand and clayey sand with gravel
Fine-Grained Soils - 50% (1) or More Passes No. 200 Sieve	Silts and Clays Liquid Limit Less than 50		ML	Silt, sandy silt, gravelly silt, silt with sand or gravel
			CL	Clay of low to medium plasticity; silty, sandy, or gravelly clay, lean clay
			OL	Organic clay or silt of low plasticity
	Silts and Clays Liquid Limit 50 or More		MH	Elastic silt, clayey silt, silt with micaceous or diatomaceous fine sand or silt
			CH	Clay of high plasticity, sandy or gravelly clay, fat clay with sand or gravel
			OH	Organic clay or silt of medium to high plasticity
Highly Organic Soils			PT	Peat, muck and other highly organic soils

Terms Describing Relative Density and Consistency

	Density	SPT (2) blows/foot
Coarse-Grained Soils	Very Loose	0 to 4
	Loose	4 to 10
	Medium Dense	10 to 30
	Dense	30 to 50
	Very Dense	>50

	Consistency	SPT (2) blows/foot
Fine-Grained Soils	Very Soft	0 to 2
	Soft	2 to 4
	Medium Stiff	4 to 8
	Stiff	8 to 15
	Very Stiff	15 to 30
	Hard	>30

Test Symbols

- FC = Fines Content
- G = Grain Size
- M = Moisture Content
- A = Atterberg Limits
- C = Consolidation
- DD = Dry Density
- K = Permeability
- Str = Shear Strength
- Env = Environmental
- PiD = Photoionization Detector

Component Definitions

Descriptive Term	Size Range and Sieve Number
Boulders	Larger than 12"
Cobbles	3" to 12"
Gravel	3" to No. 4 (4.75 mm)
Coarse Gravel	3" to 3/4"
Fine Gravel	3/4" to No. 4 (4.75 mm)
Sand	No. 4 (4.75 mm) to No. 200 (0.075 mm)
Coarse Sand	No. 4 (4.75 mm) to No. 10 (2.00 mm)
Medium Sand	No. 10 (2.00 mm) to No. 40 (0.425 mm)
Fine Sand	No. 40 (0.425 mm) to No. 200 (0.075 mm)
Silt and Clay	Smaller than No. 200 (0.075 mm)

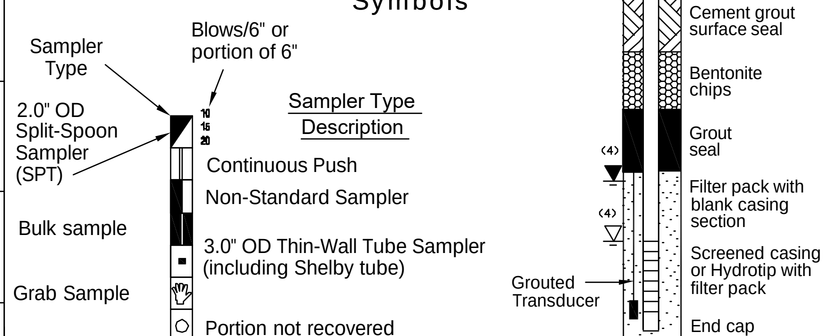
(3) Estimated Percentage

Percentage by Weight	Modifier
<5	Trace
5 to 15	Slightly (sandy, silty, clayey, gravelly)
15 to 30	Sandy, silty, clayey, gravelly)
30 to 49	Very (sandy, silty, clayey, gravelly)

Moisture Content

- Dry - Absence of moisture, dusty, dry to the touch
- Slightly Moist - Perceptible moisture
- Moist - Damp but no visible water
- Very Moist - Water visible but not free draining
- Wet - Visible free water, usually from below water table

Symbols



(1) Percentage by dry weight
(2) (SPT) Standard Penetration Test (ASTM D-1586)
(3) In General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)
(4) Depth of groundwater
▽ ATD = At time of drilling
▼ Static water level (date)
(5) Combined USCS symbols used for fines between 5% and 15% as estimated in General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)

BGS = below ground surface

Classifications of soils in this report are based on visual field and/or laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field or laboratory testing unless presented herein. Visual-manual and/or laboratory classification methods of ASTM D-2487 and D-2488 were used as an identification guide for the Unified Soil Classification System.

Aspect CONSULTING

Exploration Log Key

DATE:
DESIGNED BY:
DRAWN BY:
REVISED BY:
PROJECT NO.
FIGURE NO.
A-1

Q:_ACAD Standards\Standard Details\Exploration Log Key A1.dwg

Aspect CONSULTING	Dufresne Residence - 170202-01		Geotechnical Exploration Log	
	Project Address & Site Specific Location		*Coordinates (Lat,Lon WGS84)*	*Exploration Number*
	11143 Rolling Bay Walk NE, Bainbridge Island, WA, upper-mid slope		47.664, -122.503	**B-01**
Contractor	*Equipment*	*Sampling Method*	*Ground Surface (GS) Elev. (NAVD88)*	
Boretec1, Inc.	Modified Acker Drill	cathead safety hammer	28'	
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*	*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Maclen	4.5-in OD / 2.5-in ID Hollow-stem Auger	5/3/2017	NA	23' (ATD)

Exploration Completion and Notes: Borehole backfilled with bentonite chips.

Sample Type/ID	Blows/6"	Tests
S1	2 4 4	GS FC=32%
S2	3 4 5	MC
S3	3 3 5	GS FC=26%
S4	6 5 5	MC
S5	15 8 6	MC
S6	6 11 16	MC

Blows/foot ▲ Water Content (%) ● 0 10 20 30 40 50

Description

LANDSLIDE DEPOSITS

Loose, moist, dark brown, gravelly, silty SAND (SM); fine to coarse sand, fine to coarse gravel, numerous cobbles, trace rootlets and wood fragments.

Becomes brown.

Loose, moist, brown, very gravelly, silty SAND (SM); fine to coarse sand, subangular to rounded fine to coarse gravel, trace wood fragments, trace iron-oxide staining.

Loose, very moist, brown, sandy, slightly silty GRAVEL (GW-GM); subangular to rounded fine to coarse gravel, fine to coarse sand.

Becomes medium dense.

ADVANCE OUTWASH DEPOSITS

Medium dense, moist, light brown to light gray SAND (SP); fine to medium sand, trace silt and fine gravel, trace iron-oxide staining, homogeneous.

Legend

Sample Method: ◯ No Soil Sample Recovery; ◩ Split Barrel 2" X 1.375" (SPT)

Plastic Limit ├──┤ Liquid Limit

Water Level: ∇ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: NHC
Approved by: AJH 5/31/2017

Exploration Log B-01

Aspect CONSULTING	Dufresne Residence - 170202-01			Geotechnical Exploration Log	
	Project Address & Site Specific Location			Coordinates (Lat,Lon WGS84)	Exploration Number
	11143 Rolling Bay Walk NE, Bainbridge Island, WA, upper-mid slope			47.664, -122.503	**B-01**
Contractor	Equipment	Sampling Method		Ground Surface (GS) Elev. (NAVD88)	
Boretec1, Inc.	Modified Acker Drill	cathead safety hammer		28'	
Operator	Exploration Method(s)	Work Start/Completion Dates		Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Maclen	4.5-in OD / 2.5-in ID Hollow-stem Auger	5/3/2017		NA	23' (ATD)

Depth (feet)	Elev. (feet)	Exploration Completion and Notes	Sample Type/ID	Blows/foot ▲ Water Content (%)●	Blows/6"	Tests	Material Type	Description	Depth (ft)
			S7		19 26 30	MC		Very dense, very moist, light gray, slightly silty SAND (SP-SM); fine to medium sand, homogeneous.	
	5	▽ 5/3/2017						Very dense, wet, light gray, silty SAND (SM); fine to medium sand, homogeneous, sand heaving noted.	
25			S8		22 32 38	MC			25
	0								
30			S9		36 50/5	MC			30
								Bottom of exploration at 30.9 ft. bgs.	
	-5								
35									35
	-10								

Legend

Sample Method: No Soil Sample Recovery; Split Barrel 2" X 1.375" (SPT)

Plastic Limit ├──┤ Liquid Limit

Water Level: ▽ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: NHC
Approved by: AJH 5/31/2017

Exploration Log B-01

Aspect CONSULTING	Dufresne Residence - 170202-01		Geotechnical Exploration Log	
	Project Address & Site Specific Location		Coordinates (Lat,Lon WGS84)	Exploration Number
	11143 Rolling Bay Walk NE, Bainbridge Island, WA, toe of slope		47.664, -122.503	B-02
Contractor	Equipment	Sampling Method	Ground Surface (GS) Elev. (NAVD88)	
Boretec1, Inc.	Modified Acker Drill	cathead safety hammer	14'	
Operator	Exploration Method(s)	Work Start/Completion Dates	Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Maclen	4.5-in OD / 2.5-in ID Hollow-stem Auger	5/3/2017	NA	11.5' (ATD)

Depth (feet)	Elev. (feet)	Exploration Completion and Notes	Sample Type/ID	Blows/6"	Tests	Description
0	14	Borehole backfilled with bentonite chips.				**LANDSLIDE DEPOSITS** Very loose, moist, dark brown, gravelly, silty SAND (SM); numerous cobbles.
2.5			S1	29, 5, 4		Loose, moist, dark brown, silty SAND (SM); largely comprised of pieces of plastic, carpet, and concrete (residential debris).
4	10	Water added to borehole to break up concrete.				Concrete block encountered around 4 ft bgs.
5			S2	8, 3, 4	MC	Only a piece of styrofoam recovered at 5 ft bgs.
7.5			S3	7, 5, 7	MC	**BEACH DEPOSITS** Medium dense, wet, brown, sandy GRAVEL (GP); subrounded to well rounded predominantly fine gravel, fine to coarse sand, trace silt, trace pieces of plastic and wood.
9.5	5					Medium dense, wet, light gray to brown, very gravelly SAND (SW); trace silt, fine to coarse sand, subrounded to well rounded fine to coarse gravel.
10			S4	9, 13, 7	GS FC=4%	
11.5		∇ 5/3/2017 Inferred from tide record. Drilling mud added to borehole to combat soil heave at 11.5 ft bgs.				
12.5			S5	11, 18, 20	MC	Becomes gray.
14	0					**ADVANCE OUTWASH DEPOSITS** Dense, very moist to wet, gray, silty SAND (SM); trace gravel, fine sand.
15			S6	19, 31, 36	GS FC=14%	Becomes very dense; 1" bed of very silty fine sand.
17						Sand heaving noted.
19	-5					

Blows/foot ▲ Water Content (%) ● 0 10 20 30 40 50

Plastic Limit ├──┤ Liquid Limit

Legend

Sample Method: ○ No Soil Sample Recovery; ◪ Split Barrel 2" X 1.375" (SPT)

Water Level: ∇ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: NHC
Approved by: AJH 5/31/2017

Exploration Log B-02

Sheet 1 of 2

ASPECT STANDARD EXPLORATION LOG TEMPLATE P:\GINTW\PROJECTS\DUFRESNE RESIDENCE - 170202.GPJ June 7, 2017

Dufresne Residence - 170202-01

Geotechnical Exploration Log

ASPECT CONSULTING

Project Address & Site Specific Location	Coordinates (Lat,Lon WGS84)	Exploration Number
11143 Rolling Bay Walk NE, Bainbridge Island, WA, toe of slope	47.664, -122.503	**B-02**

Contractor	Equipment	Sampling Method	Ground Surface (GS) Elev. (NAVD88)
Boretec1, Inc.	Modified Acker Drill	cathead safety hammer	14'

Operator	Exploration Method(s)	Work Start/Completion Dates	Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Maclen	4.5-in OD / 2.5-in ID Hollow-stem Auger	5/3/2017	NA	11.5' (ATD)

Depth (feet)	Elev. (feet)	Sample Type/ID	Blows/foot ▲ Water Content (%) ●	Blows/6"	Tests	Material Type	Description
		S7		23, 31, 36	MC		**ADVANCE OUTWASH DEPOSITS** Dense, very moist to wet, gray, silty SAND (SM); trace gravel, fine sand. (continued) Becomes light gray.
	-10						
25		S8		26, 40, 50/5	MC		Becomes mica-rich ; 5 ft of sand heaving noted.
							Bottom of exploration at 26.4 ft. bgs.
	-15						
30							
	-20						
35							
	-25						

Legend

Sample Method: No Soil Sample Recovery; Split Barrel 2" X 1.375" (SPT)

Plastic Limit ⊢—⊣ Liquid Limit

Water Level: Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: NHC
Approved by: AJH 5/31/2017

Exploration Log B-02

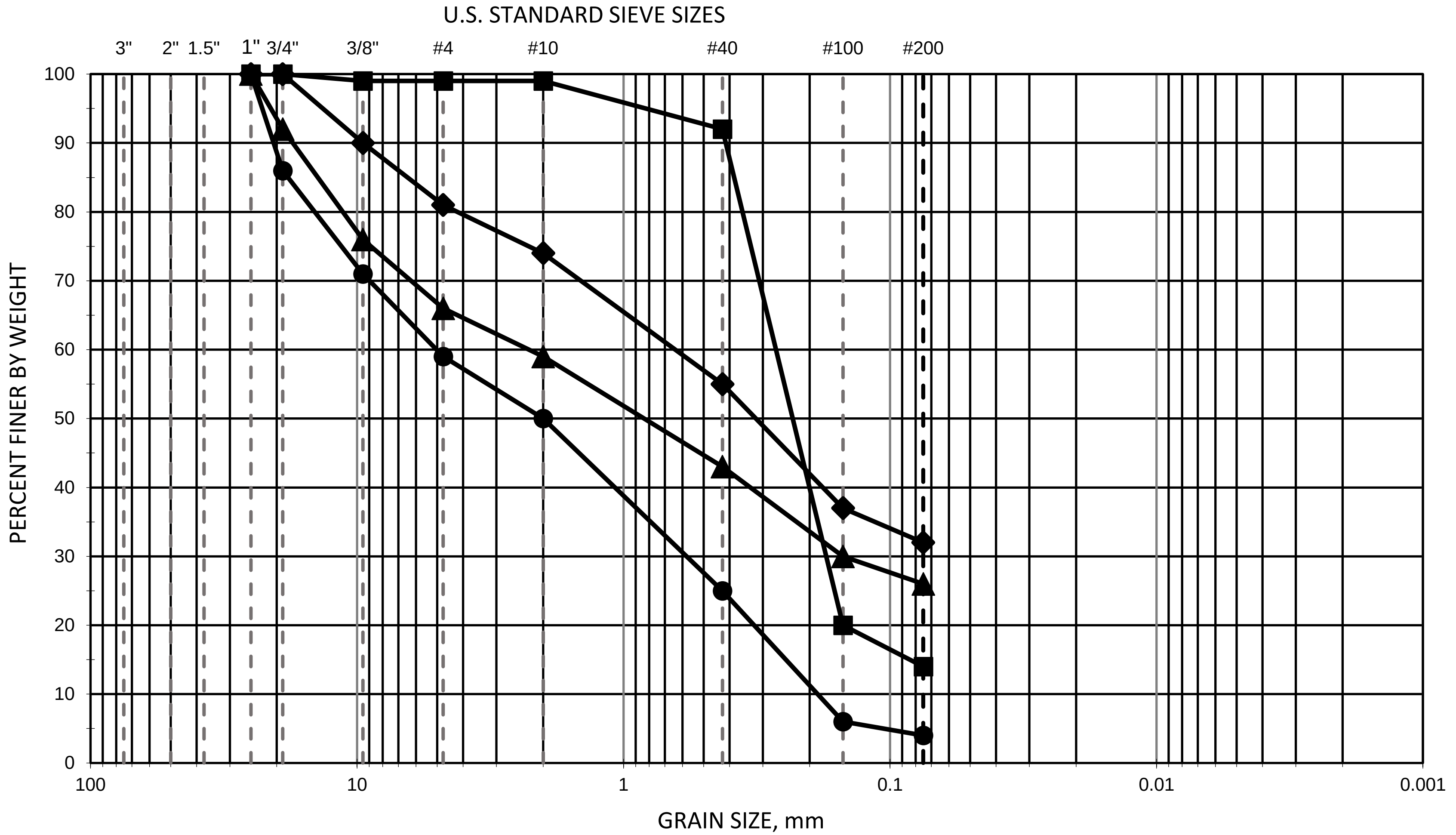
APPENDIX B

Lab Testing Results

B. Geotechnical Laboratory Testing

Laboratory tests were conducted on selected soil samples to characterize certain engineering (physical) properties of the soils at the Site. Laboratory testing included determination of grain-size distribution and moisture content. The laboratory tests were conducted in general accordance with appropriate ASTM test methods. The results of the moisture content tests are included on the exploration logs in Appendix A. The results of the grain-size distribution tests are included in this appendix. Test procedures are discussed below.

The grain size distribution of selected samples was analyzed in general accordance with ASTM D6913, *Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis*. The moisture content of selected samples was analyzed in general accordance with ASTM D2216, *Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass*.



Symbol	Exploration	Sample	Depth (ft)	Soil Description	Moisture Content %	Gravel %	Sand %	Fines %
◆	B-1	S1	2.5	Moist, dark brown, gravelly, very silty SAND (SM)	20%	19%	49%	32%
▲	B-1	S3	7.5	Moist, brown, very gravelly, silty SAND (SM)	11%	34%	40%	26%
●	B-2	S4	10	Wet, light gray to brown, very gravelly SAND (SW); trace silt	10%	41%	55%	4%
■	B-2	S6	15	Very moist to wet, gray, silty SAND (SM); trace gravel	20%	1%	85%	14%

Project Name: Dufresne Residence
Project No.: 170202-01
P:_GEOTECH\Dufresne Residence_Rolling Bay Walk\Data\Lab Data\Aspect Grain Size Distribu

Aspect CONSULTING

Particle Size Analysis of Soils
Method ASTM D6913
Figure B-1

APPENDIX C

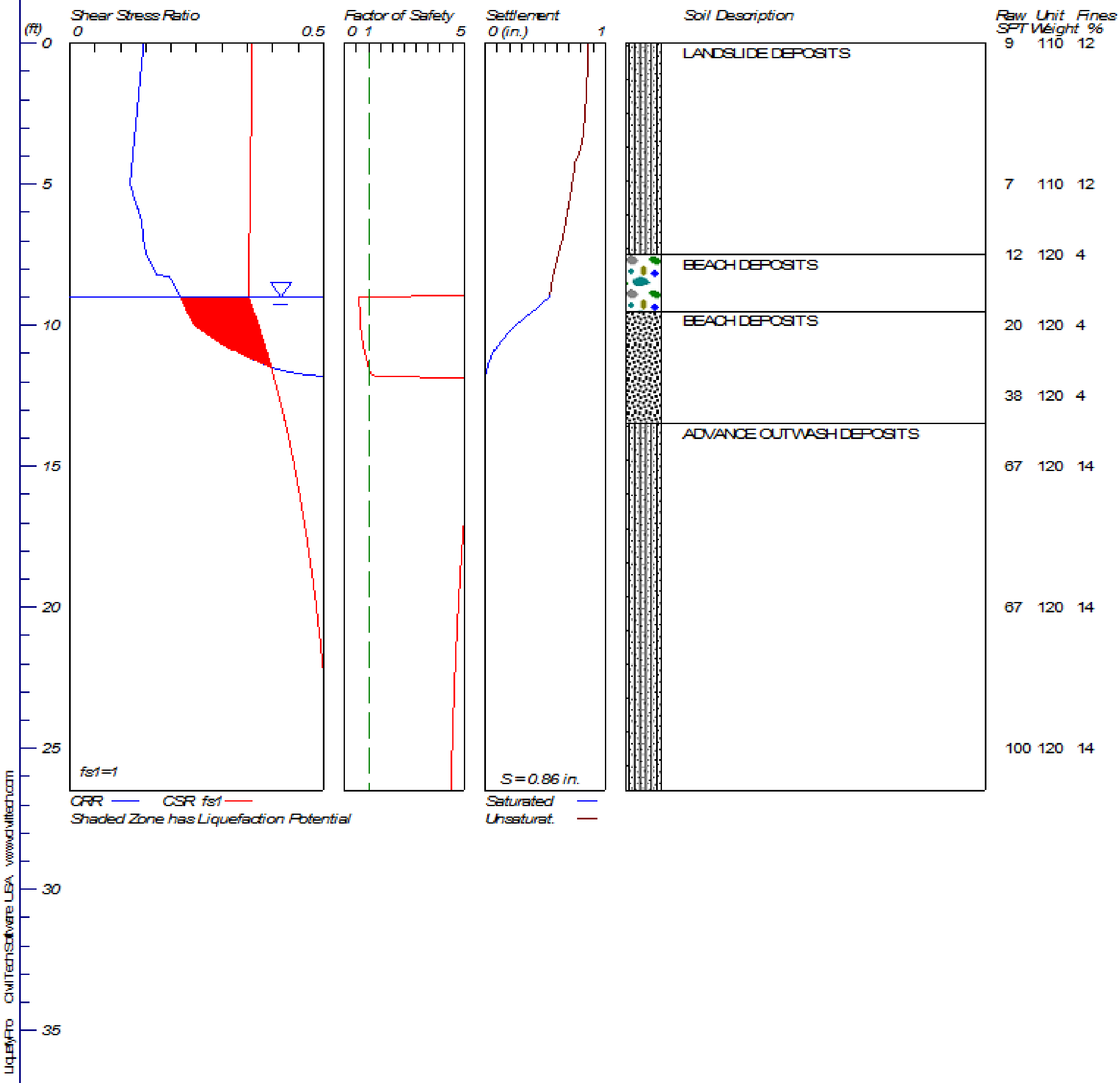
Liquefaction Analysis

LIQUEFACTION ANALYSIS

Dufresne Residence

Hole No.=B-02   Water Depth=9 ft   Surface Elev.=14

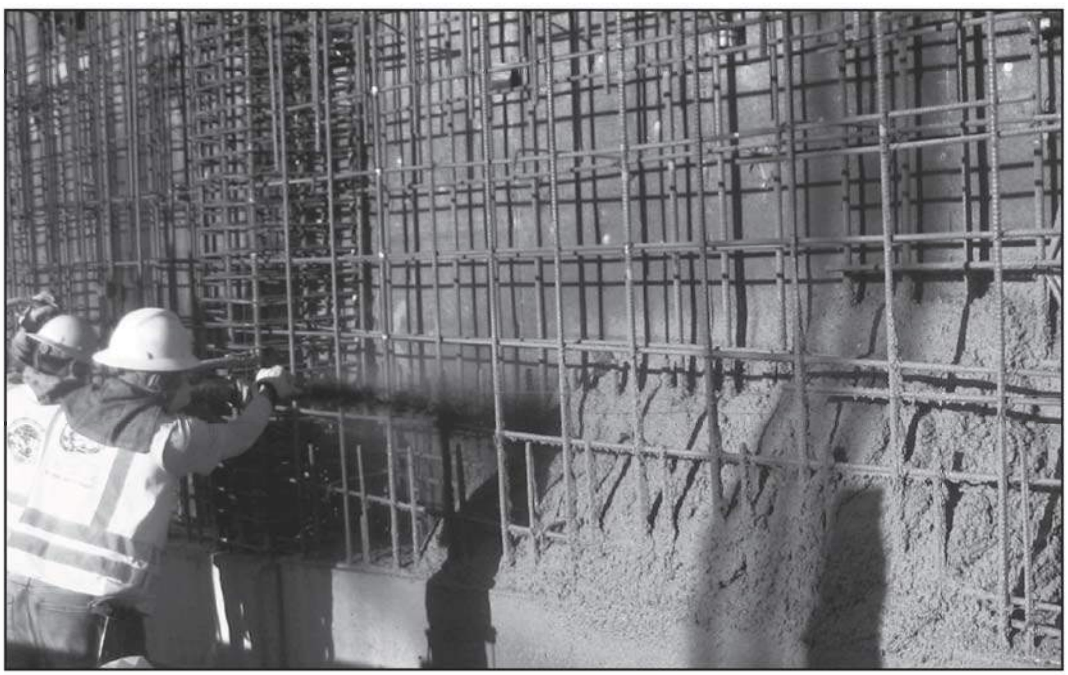
Magnitude=7.09
Acceleration=.554g



Aspect Consulting, LLC   170202   Figure C-1

SHOTCRETE APPLICATION MANUAL

Voltex



THIS INSTALLATION MANUAL IS SPECIFICALLY FOR SHOTCRETE FOUNDATION WALL APPLICATIONS. THE MANUAL IS DIVIDED INTO TWO SECTIONS: HYDROSTATIC AND NON-HYDROSTATIC CONDITIONS.

CETCO®

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VOLTEX DS SHOTCRETE MANUAL

THIS MANUAL CONTAINS THE INSTALLATION GUIDELINES FOR THE VOLTEX DS WATERPROOFING SYSTEM ON FOUNDATION SHORING WALLS, WHERE SHOTCRETE WILL BE APPLIED AS THE STRUCTURAL WALL. THIS MANUAL DOES NOT COVER THE INSTALLATION OF VOLTEX DS WITH CAST-IN-PLACE CONCRETE APPLICATIONS. FOR APPLICATIONS NOT COVERED IN THIS MANUAL, CONTACT CETCO FOR SPECIFIC INSTALLATION GUIDELINES. BEFORE INSTALLING VOLTEX DS, READ THIS INSTALLATION MANUAL TO GAIN FAMILIARITY WITH SPECIFIC PROCEDURES AND APPLICATIONS.

CONTENTS

SHOTCRETE SUMMARY

Shotcrete is concrete pneumatically projected onto a surface at high velocity. It is typically blown through a hand-held nozzle, after all the ingredients are mixed to project specification. Using shotcrete for constructing property line below-grade structural walls has become more common over the last ten years.

However, application of shotcrete can create difficult conditions for insuring the performance of below-grade waterproofing systems. Improperly mixed or placed shotcrete can, among other problems, create voids between the shotcrete and the waterproofing membrane. Shadow voids or poor consolidation behind steel reinforcement is another common problem. Shotcrete's high cement to water ratio can promote excessive cracking and increased efflorescence. Improper cleaning of shotcrete during application can produce rebound pockets, unbonded areas, sags and other defects; shotcrete overspray contamination can reduce the excellent mechanical bond that normally occurs between the Voltex DS shotcrete.

Unlike cast-in-place concrete, the success in applying shotcrete is mostly dependent on the skill and experience of the shotcrete installation crew, especially the nozzleman. Although it may be reassuring to have properly qualified nozzlemen on the project, it cannot be assured that the quality of in-place shotcrete will be consistently high unless there is continued attention to, and inspection of, shotcrete placement. The inspection process should include substrate preparation, steel reinforcement (position, size and anchorage), shotcrete material quality, mixing of materials, equipment operation and gunning technique, encapsulation of steel reinforcement and proper curing.

Furthermore, even properly applied shotcrete is more likely to develop shrinkage cracks than cast-in-place concrete during curing; due to shotcrete's lower water to cement ratio, and the lack of actual field practice of moisture curing the shotcrete for several days per American Concrete Institute (ACI) 506.

To address the difficult conditions of waterproofing shotcrete, CETCO has developed special product installation guidelines and drafted specification coordination directives for design professionals to use when specifying the Voltex DS System to waterproof below-grade, shotcrete foundation walls.

The special installation guidelines covered in this manual are further segmented by hydrostatic and non-hydrostatic conditions. Refer to and follow the correct section that meets the conditions on your project.

HYDROSTATIC / NON-HYDROSTATIC CONDITIONS

Hydrostatic conditions exist when elevation of the below-grade foundation is lower than the project site ground water level or historical high water table. Hydrostatic conditions are typically continual but may be intermittent with the fluctuation of the natural ground water table.

VOLTEX DS SHOTCRETE MANUAL

Non-hydrostatic conditions exist when site soil testing has determined that no ground water table exists or the elevation of the below-grade foundation is well above the expected historical high water table elevation. Intermediate temporary hydrostatic pressure conditions may exist after precipitation or irrigation but is not a continual or prolonged condition.

PRODUCT DESCRIPTION

Voltex DS is a highly effective waterproofing membrane comprised of 1.10 pounds of sodium bentonite per square foot, two polypropylene geotextiles, and an integral polymeric liner bonded to the outside surface of the non-woven geotextile. The two geotextiles are interlocked by a proprietary needle punching process which encapsulates and confines the sodium bentonite. The polymeric liner provides extremely low permeability for water vapor and gas transmission.

Sodium bentonite is a non-toxic mineral of volcanic origin that expands and seals upon hydration with water. Bentonite stops water by forming a dense monolithic membrane on the exterior of the structure that is impervious to water.

Voltex DS is excellent for waterproofing below-grade horizontal and vertical surfaces. Typical applications are underslab and property line construction, including soldier pile and lagging, metal sheet piling, shotcrete soil retention and concrete caisson retaining walls.

Installation of Voltex DS is fast and easy. Simply position the product into place and fasten. Voltex DS can be installed on green concrete, in virtually any weather, without the need for primers or adhesives. Voltex DS can be easily cut on site to form around corners and penetrations. Voltex DS is installed with an accompaniment of accessory and associated system products to provide a waterproofing system.

VOLTEX DS SPECIFICATIONS	
Roll Length	**14.5 Ft. (4.4 m)**
Roll Width	**4 ft. (1.2 m)**
Bentonite per Sq. Ft.	**1.10 lbs (5.3 kg/m²)**
Typ. Roll Weight	**68 lbs (30.8 kg)**

ACCESSORIES

BENTOSEAL®: trowel grade compound used to detail around penetrations, corner transitions and terminations.
HYDROBAR TUBES®: water soluble film tubing filled with bentonite, used at the footing/wall intersection.
TB-BOOT®: Pre-formed thermoplastic cover installed over both cable and rod type tie-back heads.
WATERSTOPPAGE®: active granular product used at detail areas that require additional waterproofing protection.
TERMINATION BAR: Min. 1" (25 mm) wide aluminum bar with pre-punched holes on 12" (300 mm) centering for fastening.
CEMENTITIOUS BOARD: ½" (12 mm) thick cementitious wall board for protection of waterproofing during the removal of steel soldier pile cap and top lagging boards.
GF-40SA: 40-MIL thick UV resistant flashing membrane for grade terminations and thru-wall flashing.
CETSEAL: multi-purpose, single component polyether moisture cure sealant/adhesive. CETSEAL is a low VOC, 100% solids, non-shrinking product with excellent UV resistance.

Additional accessory products not listed herein may be required for site specific details.

ASSOCIATED SYSTEM PRODUCTS

CORTEX: 4-ft x 25-ft roll of Active Polymer Core barrier material to provide high performance substrate system.
WATERSTOP-RX®: active, swelling concrete joint waterstop used around penetrations and applicable concrete joints. Swells upon hydration.
AQUADRAIN®: foundation drainage composite consisting of a molded profile core and a filter fabric.

LIMITATIONS OF VOLTEX DS

Use Voltex DS with reinforced shotcrete walls, conforming to ACI 506 Core Grade 1 or 2; minimum 8" (200 mm) thick, applied from the bottom up to their full design thickness in a single application with lift heights limited to a maximum 4 feet (1.2 m). Do not use stay-in-place concrete forming; use removable forming products only.

Voltex DS is not designed for above grade or unconfined waterproofing applications. Waterproofing products should not be installed in standing water or over ice. If ground water contains strong acids, alkalies, or is of a conductivity of 2,500 μmhos/cm or greater, water samples should be submitted to the manufacturer for compatibility testing. Ultraseal or CoreFlex may be required if contaminated ground water or saltwater conditions exist.

Voltex DS is not designed for split-slab deck construction or to waterproof expansion joints. Expansion joints are the responsibility of others. Do not use Voltex DS on masonry block foundation walls. Refer to other product manuals for installation instructions and limitations regarding underslab and cast-in-place concrete applications. Refer to CETCO's warranty documents for guidelines, eligibility, coverage and protocol. NOTE: Illustrations herein are not to scale.

SECTION - H1
GENERAL GUIDELINES

Install Voltex DS Waterproofing System with the woven geotextile side (dark gray) facing the installer so that the shotcrete will be against the dark gray geotextile side. On shoring walls to receive shotcrete, install Voltex DS with minimum 6" (150 mm) sheet edge overlaps fastened with both washer-head fasteners placed maximum 24" (600 mm) on center and pneumatic staples placed 6" (150 mm) on center. Install pneumatic staples within 1" (25 mm) of sheet edge to tightly secure membrane overlap to the shoring wall. Secure center line of Voltex DS sheets to shoring wall with pneumatic staples or washer-head fasteners as required to hold membrane tight against shoring wall (Figure H2-2).

Prior to installing adjacent Voltex DS sheets, apply continuous ¼" (6 mm) thick by 3" (75 mm) wide trowel of Bentoseal along top and side edges of the installed Voltex DS sheet. Install Bentoseal so it will be confined within the 6" (150 mm) membrane edge overlap (Figure H2-4).

Protect waterproofing products from hydrating before material is contained with concrete, shotcrete, or backfill. After any precipitation, standing water should be pumped off waterproofing as soon as possible.

Voltex DS waterproofing is not an expansion joint filler or sealant, but may be used as an expansion joint cover over properly installed expansion joint material placed during substrate preparation.

Protect adjacent work areas and finish surfaces from contamination from waterproofing products during installation operations.

Substrate Preparation: Excavation contractor should provide shoring wall in good condition to receive waterproofing system. Shoring should extend to the lowest level of the waterproofing installation with any voids or cavities exterior of the shoring filled with compacted soil or cementitious grout. With lagging, interior surface of boards should be uniform and tight together with gaps less than 1" (25 mm). Gaps in excess of 1" (25 mm) should be filled with cementitious grout or CETCO approved 2-part spray closed cell polyurethane foam, minimum 20 PSI compressive strength. Irregular lagging may require a liner prior to waterproofing installation to smooth out substrate prior to waterproofing installation received. Contact CETCO for recomendations based on the severity of condition.

SECTION - H2
CORTEX BASE

At the underslab-to-wall transition, first install Voltex DS membrane horizontally oriented (dark gray geotextile side facing the installer) with a minimum 12" (300 mm) of the bottom portion extending out onto the substrate (Figure H2-1). Secure Voltex DS membrane edges to shoring wall with washer-head fasteners maximum 24" (600 mm) on center. Apply Bentoseal along membrane sheet edges and then overlap all adjacent sheets edges a minimum 6" (150 mm). The top membrane edge of this Voltex DS transition installation must extend a minimum 30" (750 mm) above the finished slab elevation. Thus, as required, install a second full sheet of Voltex DS to meet the 30" (750 mm) requirement above the slab elevation.

Starting at base corner of the shoring wall, install Cortex membrane horizontally oriented over Voltex DS corner transition and continue up shoring wall to finished grade detail. Overlap Cortex roll edges minimum 4" (100 mm) and secure along each edge with washer-head fasteners maximum 24" (600 mm) on center (Figure H2-1). Stagger overlap joints of succeeding courses and with previously installed Voltex DS.

For HydroShield Warranty eligibility, the slab-to-wall transition requires that a minimum 24" (600 mm) of the shotcrete wall base be applied inside a temporary, removable cast-in-place kicker form and vibrated for proper consolidation (Figure H2-2). The balance of the wall can be constructed with wet method shotcrete spray installation techniques conforming to ACI 506 Core Grade 1 or 2.

The minimum 24" (600 mm) kicker form shotcrete application technique shall be used at the slab-to-wall transition for all below-grade levels of the structure that will be constructed within the historical high ground water elevation as determined by the projects geotechnical report. Additionally, the kicker form technique shall be used one floor level above the ground water elevation when one or more levels of the structure will be constructed above the ground water elevation.

Employ substrate methods to stop water flowing through shoring wall prior to Cortex installation. If only water seepage, install 6-mil polyethylene sheeting over the seepage area prior to installing the Cortex. Polyethylene sheeting should extend from seepage elevation to base of wall to protect entire Cortex installation at that area.

Cementitious Board: Prior to installing Cortex to finished grade detail, install ½" (12 mm) thick cementitious wall board centered over steel soldier pile from finished grade elevation to specified depth that the top of steel soldier pile and wood lagging will be removed (Figure H2-3).

SECTION - H3
SLAB TO WALL TRANSITION

At base of shoring wall, install Voltex DS waterproofing sheet over the Cortex previously installed onto the shoring wall following procedure in Section H2, page 4. Install Voltex DS sheet horizontally oriented (dark gray geotextile side facing the installer) with the bottom edge extending down to the wall/slab transition corner as shown in Figure H3-1. Secure Voltex DS to lagging wall with washer-head fasteners maximum 24" (600 mm) on center along edges and down center of sheet to secure firmly. Overlap adjacent Voltex DS sheet edges a minimum 6" (150 mm). Maintain a minimum 2" (50 mm) spacing between Voltex DS and reinforcement steel.

Prior to installing adjacent Voltex DS sheets, apply continuous ¼" (6 mm) thick by 3" (75 mm) wide trowel of Bentoseal along top and side edges of the installed Voltex DS sheet. Install Bentoseal so it will be confined within the 6" (150 mm) membrane edge overlap.

Section continued on page 6.

VOLTEX DS SHOTCRETE HYDROSTATIC

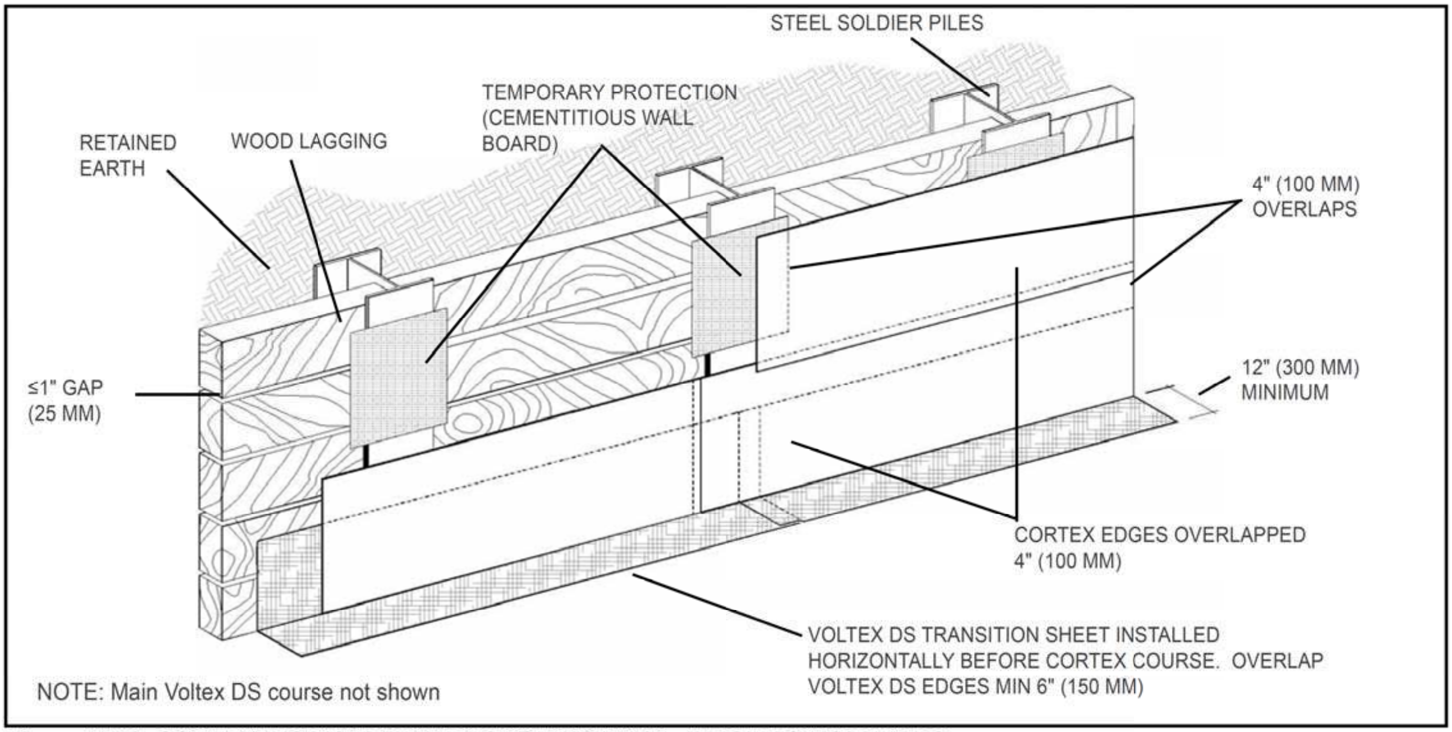


Figure H2-1: CORTEX INSTALLED ONTO SHORING WALL PRIOR TO VOLTEX DS
Install Cortex onto shoring wall prior to installing Voltex DS waterproofing membrane.

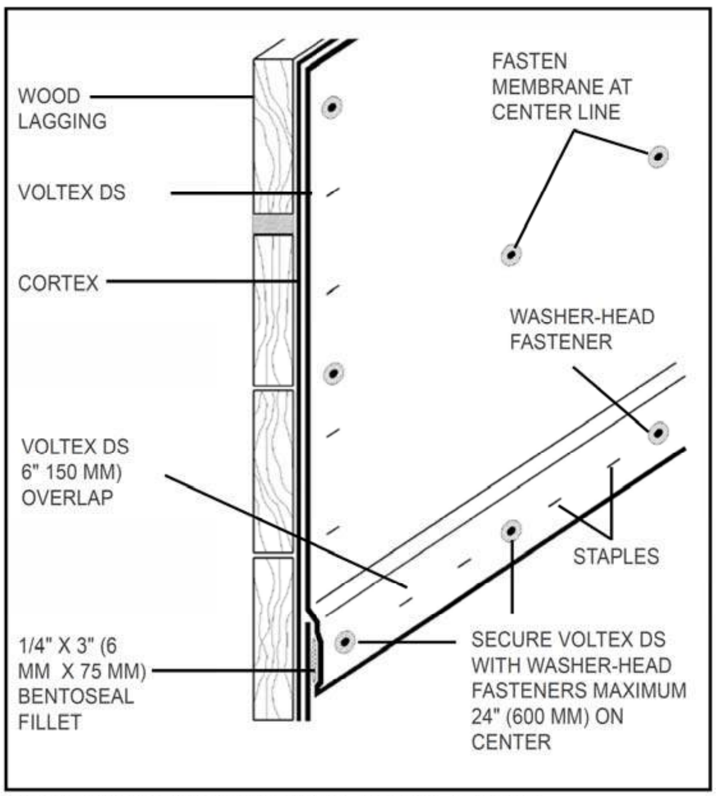


Figure H2-2: GENERAL WALL INSTALLATION
Install Cortex layer onto shoring then install Voltex DS layer directly over Cortex with washer-head fasteners placed maximum 24" (600 cm) on center and staples placed 6" (50 mm) on center.

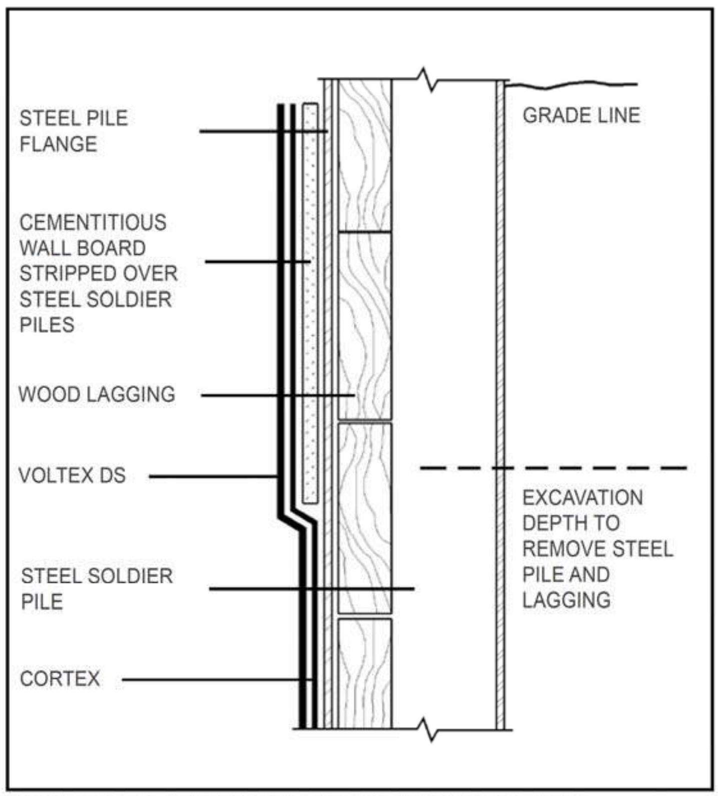


Figure H2-3: CEMENTITIOUS BOARD AT GRADE
Install cementitious board strip over steel piling at grade to protect waterproofing during removal of top lagging timbers and top of steel pile (typically with acetylene torch). Remove cement board before backfilling.

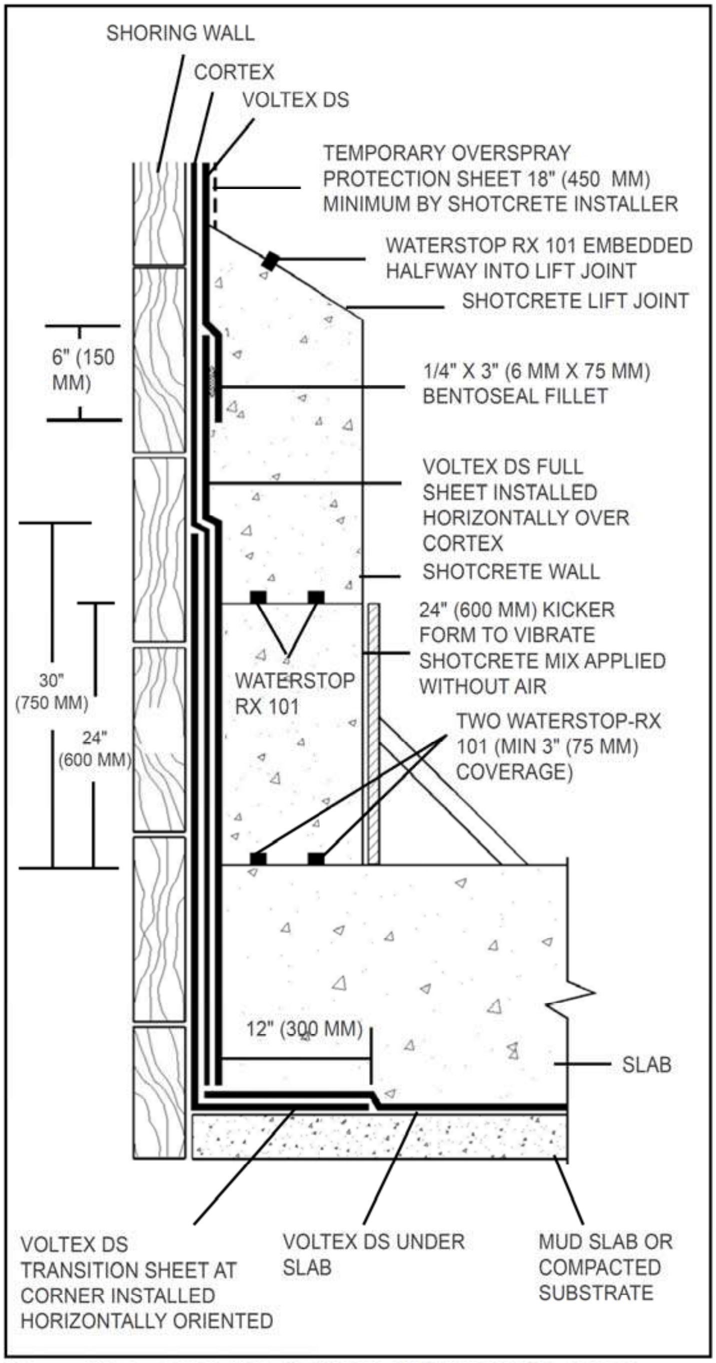


Figure H3-1: SHOTCRETE WALL WITH KICKER FORM
Install base Voltex DS sheet horizontally oriented onto shoring wall over Cortex and extend the underslab Voltex DS sheet to the slab/wall corner overlapping the Voltex DS transition sheet a minimum 12" (300 mm).

Continued from page 4.
Continue Voltex DS installation along base of wall with sheets horizontally oriented. Overlap adjacent Voltex DS sheet edges a minimum 6" (150 mm). Further secure membrane overlap edges with pneumatic staples placed 6" (150 mm) on center within 1" (25 mm) of sheet edge.

Install under slab Voltex DS membrane extending to corner (gray geotextile side up), fully overlapping the 12" (300 mm) horizontal tail of the Voltex DS corner membrane installed at the wall base. Secure corner edge of membrane with washer-head fasteners or pneumatic staples 12" (300 mm) on center.

For HydroShield Warranty eligibility, bottom 24" (600 mm) of wall shall be temporarily formed so that the project's approved shotcrete mix can be installed inside the kicker form without air velocity and consolidated by vibration per ACI industry standards.

If slab is greater than 24" (600 mm) thick, consult CETCO for guidelines.

NOTE: Reinforced shotcrete walls shall conform to ACI 506 Core Grade 1 or 2. Do not use stay-in-place concrete forming; use removable forming products only.

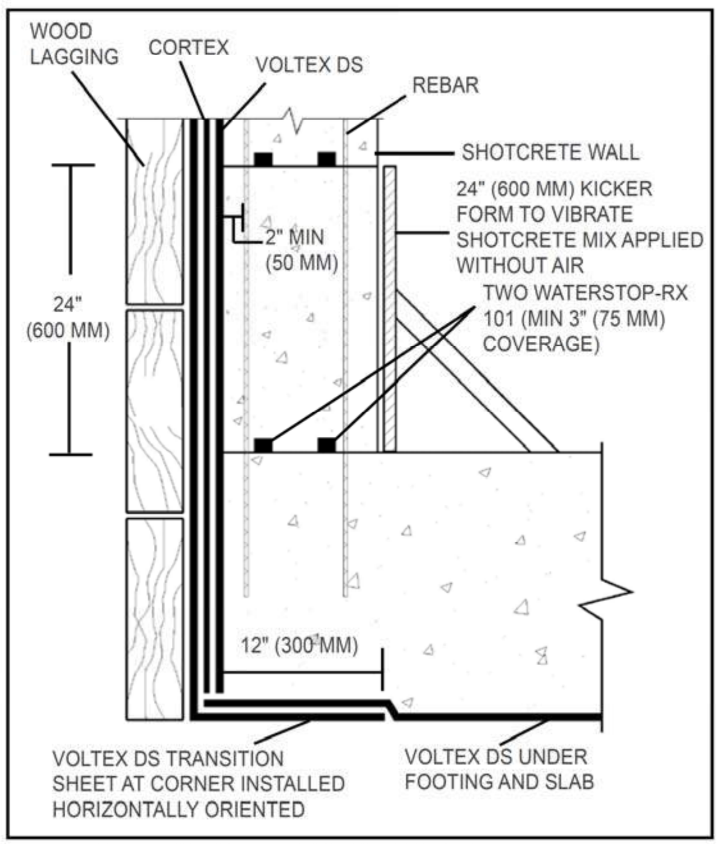


Figure H3-2: SLAB TO WALL TRANSITION
Voltex DS sheet installed at the corner should extend past the height of the top of the finished slab level a minimum 12" (300 mm) and extend under the slab 12" (300 mm).

VOLTEX DS SHOTCRETE HYDROSTATIC

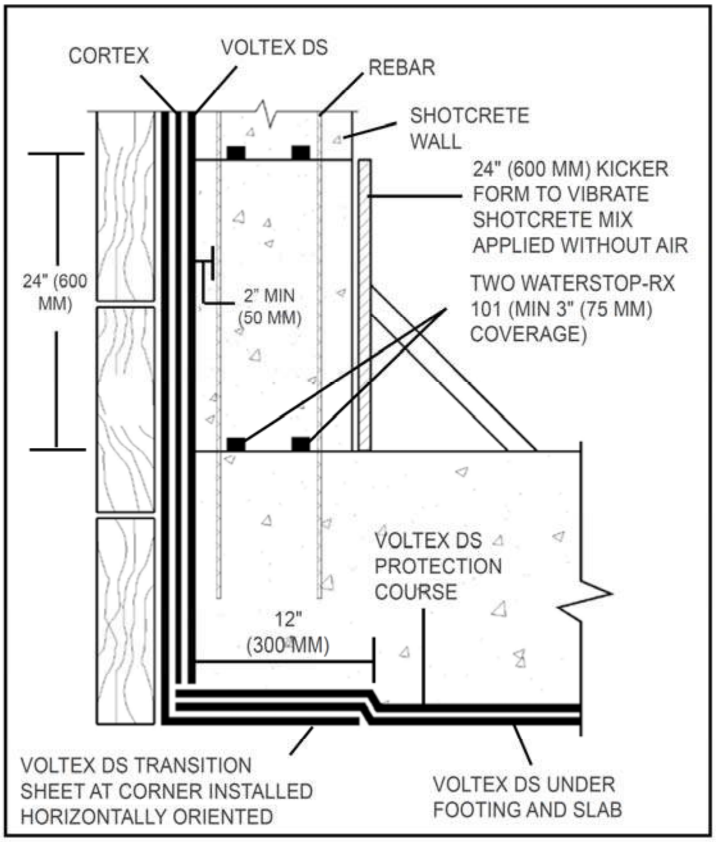


Figure H3-2A: VOLTEX DS PROTECTION COURSE
As specified or to provide additional waterproofing protection, install a second course of Voltex DS over the first course of Voltex DS in lieu of a concrete protection slab.

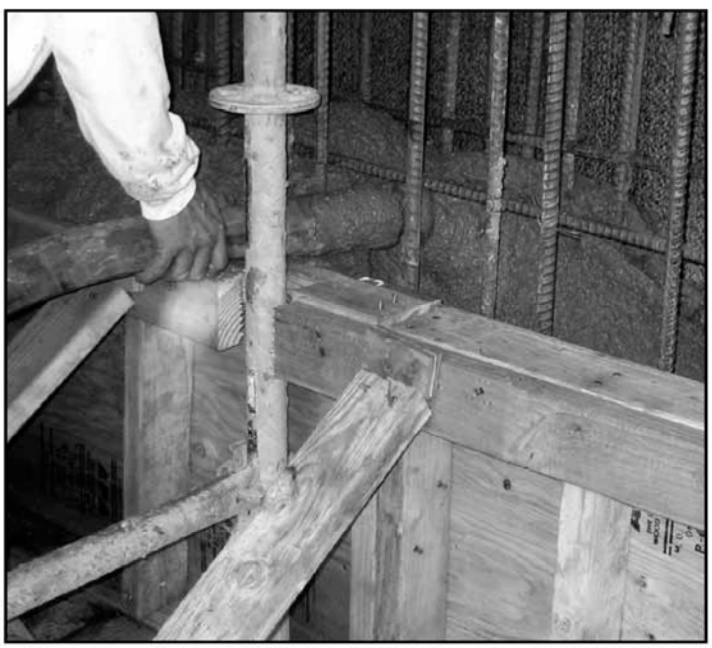


Photo H3-3: SHOTCRETE PLACEMENT IN KICKER FORM
Apply project approved shotcrete mix without air velocity into minimum 24" (600 mm) high kicker form and then consolidate by vibration.

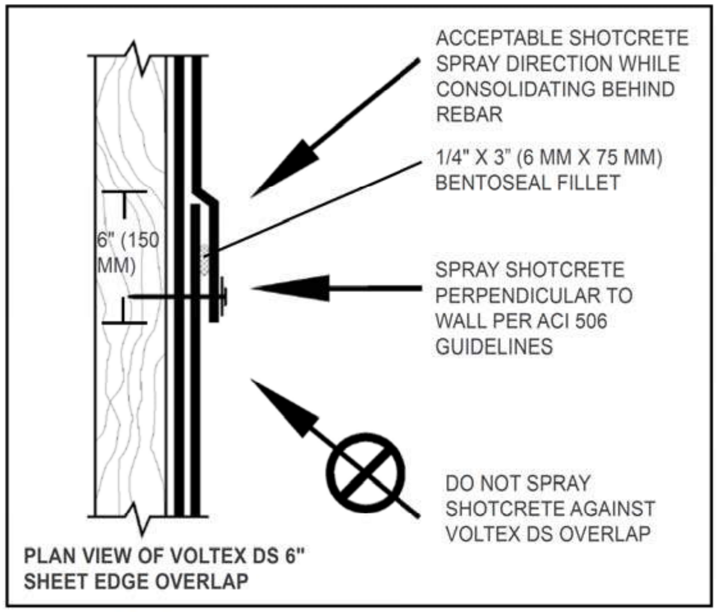


Figure H3-4: SHOTCRETE SPRAY DIRECTION
Shotcrete gunning should be applied straight against the wall per ACI 506. Do not allow shotcrete to be sprayed against Voltex DS overlap as illustrated above.

SECTION - H4
SHORING WALL

Cast-in-Place Columns/Pilasters: As it is extremely difficult to place and consolidate dry-mix shotcrete behind heavy or closely placed steel reinforcement typical within integrated or attached columns and pilasters, CETCO HydroShield eligibility requires that these structural components be cast-in-place (Figure H4-1). Sequence integrated cast columns and pilasters prior to spraying the connecting or adjoining shotcrete walls in order to minimize the impact of overspray and maintain the cleanest surface possible to pour against in these locations. Sequencing in this manner will also create a much more stable/defined surface to spray the shotcrete against when construction adjacent walls. If the shotcrete walls are constructed prior to the cast-in-place columns, use removable forming and not stay-in-place forming. Install Voltex DS sheet to shoring wall (dark gray geotextile side facing installer) with the bottom edge overlapping top edge of the Voltex DS corner transition work a minimum 6" (150 mm). Secure sheet edges to shoring with both washer-head fasteners placed maximum 24" (600 mm) on center and pneumatic staples placed 6" (150 mm) on center within 1" (25 mm) of sheet edge.

Membrane Installation: Prior to installing adjacent Voltex DS sheets, apply continuous ¼" (6 mm) thick by 3" (75 mm) wide trowel of Bentoseal along top and side edges of the installed Voltex DS sheet. Install Bentoseal so it will be confined within the 6" (150 mm) membrane edge overlap.

Secure center line of Voltex DS sheets to shoring wall with pneumatic staples or washer-head fasteners as required to hold membrane tight against shoring wall.

After the bottom horizontal transition sheet course, Voltex DS sheets can be installed either vertically or horizontally oriented. Continue Voltex DS installation up wall to finished grade elevation detail, or as specified, staggering all sheet roll ends of adjacent courses a minimum 12" (300 mm). Do not allow horizontal Voltex DS overlap joints to run at same elevation as the shotcrete lift joints. Plan by chalk lining the location of shotcrete lift joint lines.

Inspect completed waterproofing installation and repair any damaged material prior to shotcrete placement. Assure Voltex DS overlap is not separated during shotcrete placement.

Tie-Back Covers: Select appropriate size TB-Boot to fit over tie-back plate and allow proper concrete coverage per project requirements. TB-Boot should fit entirely over tie-back head without the tie-back plate or cables in direct contact with the TB-Boot. Prior to TB-Boot installation, fill voids in retention wall substrate and tie-back head assembly with spray foam (min. 20 PSI) or non-shrink grout. Prior to TB-Boot placement, install waterproofing membrane strip over soldier pile.

Fill pre-formed of TB-Boot with 2-part urethane spray foam (min. 20 PSI) and place over tie-back head before foam sets up. Secure TB-Boot to soil retention system with washer-head fasteners along the outside edge of the flat base. Apply ¼" (6mm) thick by 3" (75mm) wide continuous ring of Bentoseal onto the flat base just to the outside of the ½" (12mm) raised collar. Install Cortex and then Voltex DS membrane overlapping the entire flat base to the ½" (12mm) raised collar. Secure both the Cortex and Voltex DS sheet edge with washer-head fasteners just outside the ½" (12mm) raised collar so that the fasteners pass through the Bentoseal ring; typical fastener spacing 6" (150mm) on center. Do not install fasteners or puncture TB-Boot inside of the ½" raised collar. Complete detail by applying continuous counter flashing of Bentoseal along Voltex DS field sheet edge.

For soil nail rod and plate assemblies, install applicable TB-Boot over assembly and fasten to shoring wall. Install Cortex and then Voltex DS with Bentoseal detailing per TB-Boot installation guidelines herein.

Penetrations: Install a cut collar of Cortex tightly around the penetration; extending Cortex around penetration a minimum 12" (300 mm) radius.

Apply Bentoseal over Cortex collar around penetration; extending Bentoseal a minimum 3" (75 mm) radius at minimum ¼" (6 mm) thickness. Then install main course of Voltex DS membrane tightly around the penetration over Cortex. Finally, detail around penetration with ¾" (18 mm) thick cant of Bentoseal mastic. With sleeved penetrations, fill the gap between the pipe and the sleeve with Department of Transportation (DOT) non-shrink grout or 2-part polyurethane spray foam (min. 20 psi) and install mechanical seal (by Others) around the pipe (Figure H4-5A).

Rebar Anchorage: Install Bentoseal ¾" (18 mm) thick around all rebar anchorage penetrating Voltex DS. Then install a length of Waterstop-RX around the shaft of the rebar anchorage (Figure H4-6) securing it with zip tie or rebar wire.

Waterstop: As part of Division 3 Shotcrete Work, the shotcrete contractor should install one strip of Waterstop-RX 101 into each shotcrete lift joint, and two strips of Waterstop-RX 101 at all construction cold joints regardless of whether the joint will encounter hydrostatic or non-hydrostatic conditions. Applicable construction cold joints include the shotcrete wall to slab/footing; vertical formed work edge joints; shotcrete to cast-in-place columns; and daily shotcrete lift joint stops. Refer to Waterstop-RX product literature for installation guidelines.

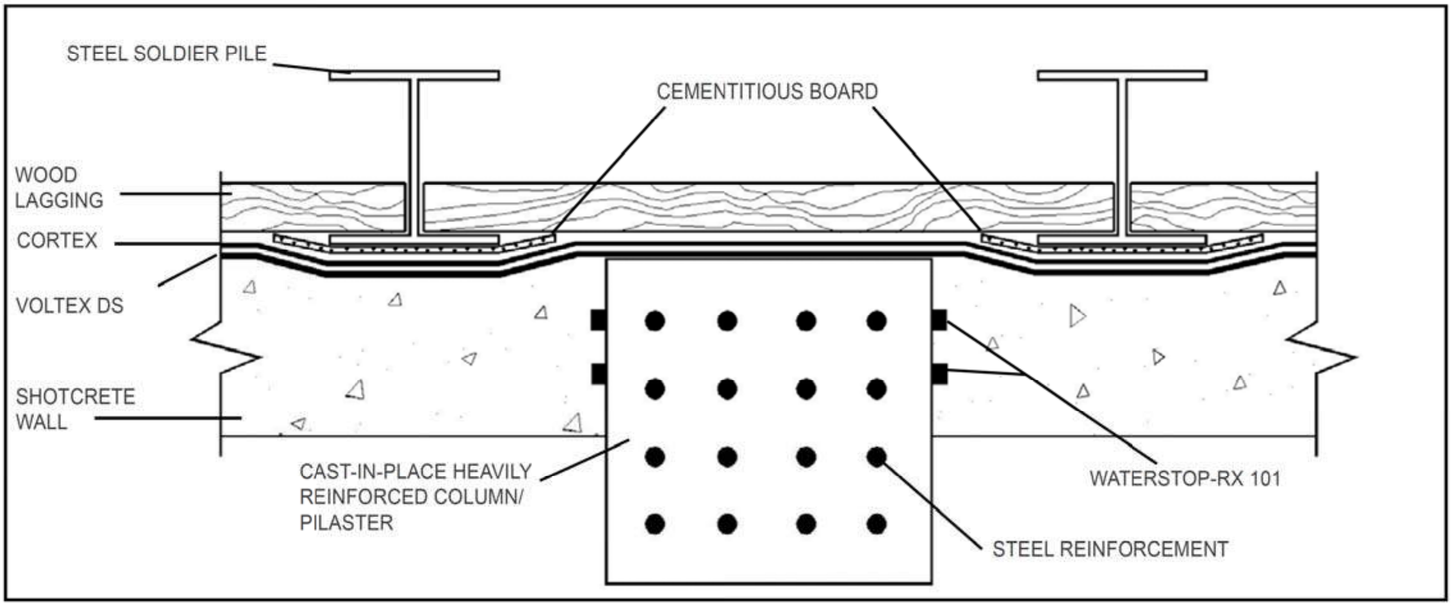


Figure H4-1: CAST-IN-PLACE STRUCTURAL COLUMNS AND PILASTERS
For HydroShield Warranty eligibility, CETCO requires that structural columns and pilasters be constructed with cast-in-place concrete. It is extremely difficult to properly apply and consolidate shotcrete, behind heavy or closely spaced steel reinforcement.

VOLTEX DS SHOTCRETE HYDROSTATIC

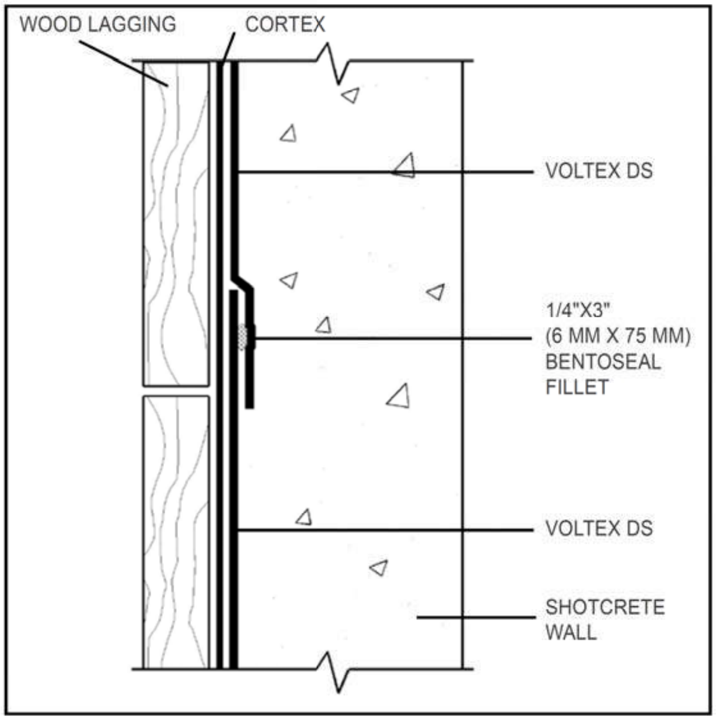


Figure H4-2: APPLY BENTOSEAL IN OVERLAPS
Prior to installing adjacent Voltex DS sheets, apply continuous 1/4" x 3" trowel of Bentoseal along top and side edges of previously installed Voltex DS sheet

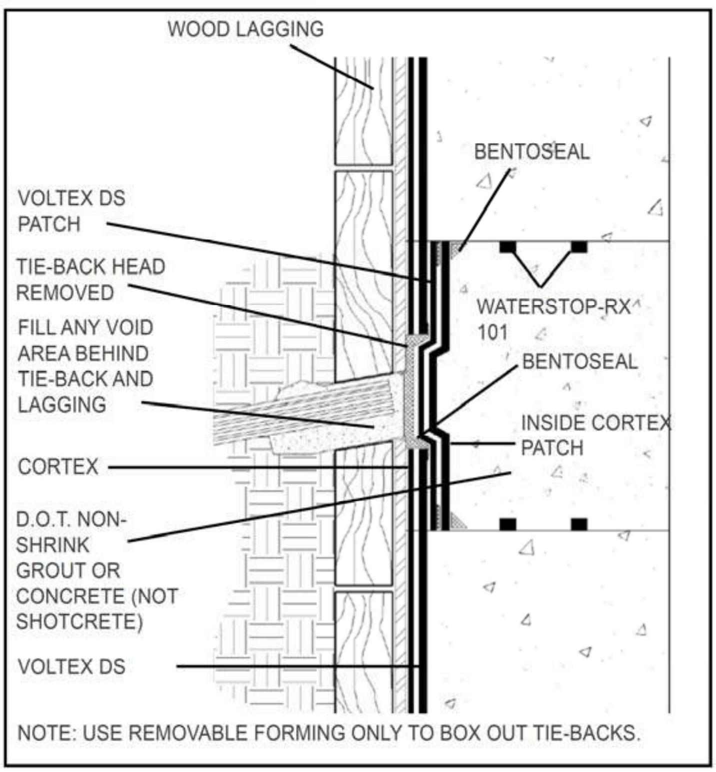


Figure H4-4: TIE-BACK BOX OUT DETAIL
After tie-back head removal, complete detail by installing Voltex DS patch, Bentoseal, inside Cortex patch, and Waterstop-RX. Only use D.O.T. approved non-shrink grout or concrete to fill box out (no shotcrete).

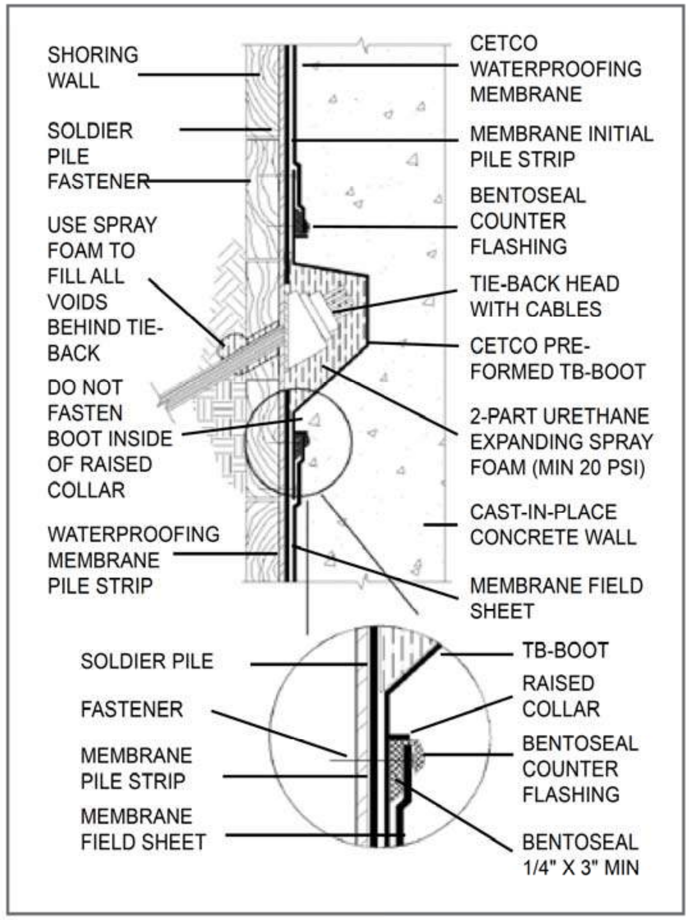


Figure H4-3 TIE-BACK DETAIL
Install TB-Boot centered over tie-back then install Voltex DS with Bentoseal detailing. Do not fasten boot inside of raised collar around center formed area.

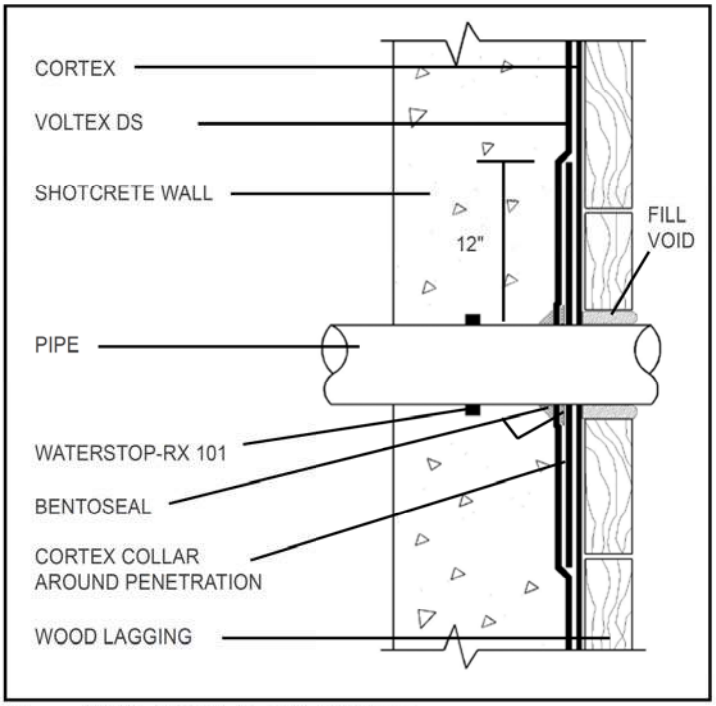


Figure H4-5: WALL PENETRATION
Cut and secure Voltex DS tightly around penetrations and then apply Bentoseal ¾" (18 mm) ring around penetration and extend over membrane a minimum 3" (75 mm) radius at minimum ¼" (6 mm) thickness.

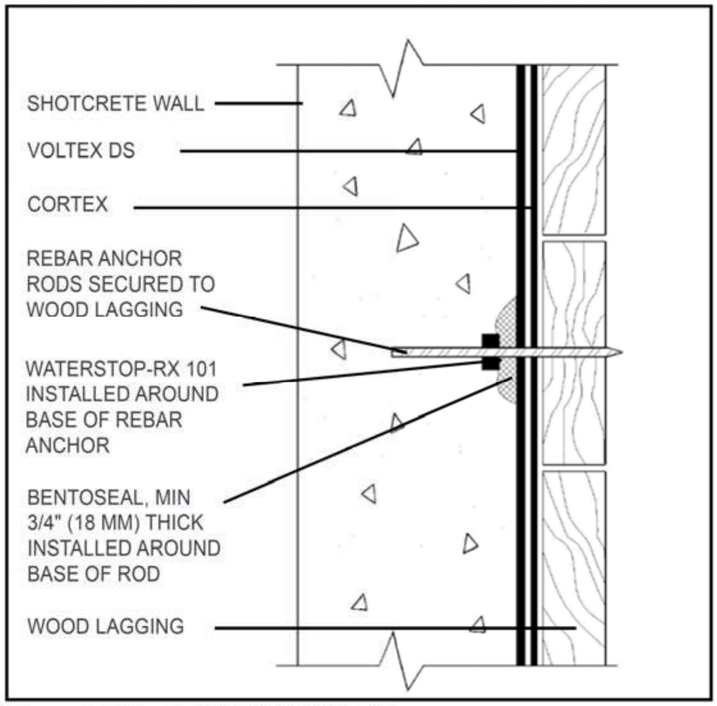


Figure H4-6: REBAR ANCHORAGE
Install Bentoseal ¾" (18 mm) thick around all rebar anchorage penetrating Voltex DS. Then install a length of Waterstop-RX around the shaft of the rebar anchorage secured with plastic zip tie or rebar wire.

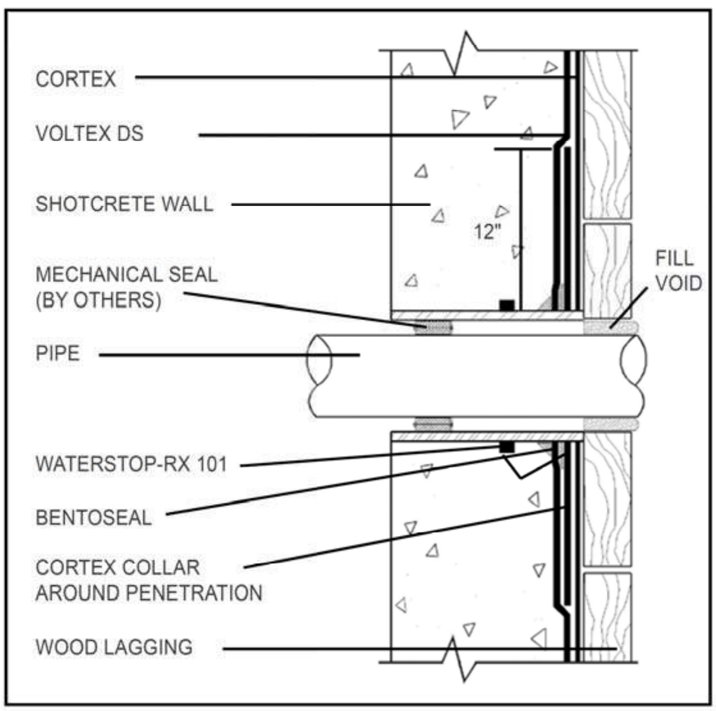


Figure H4-5A: SLEEVED WALL PENETRATION
Cut and secure Voltex DS tightly around penetrations and then apply Bentoseal ¾" (18 mm) thick ring around penetration and extend over membrane a minimum 3" (75 mm).

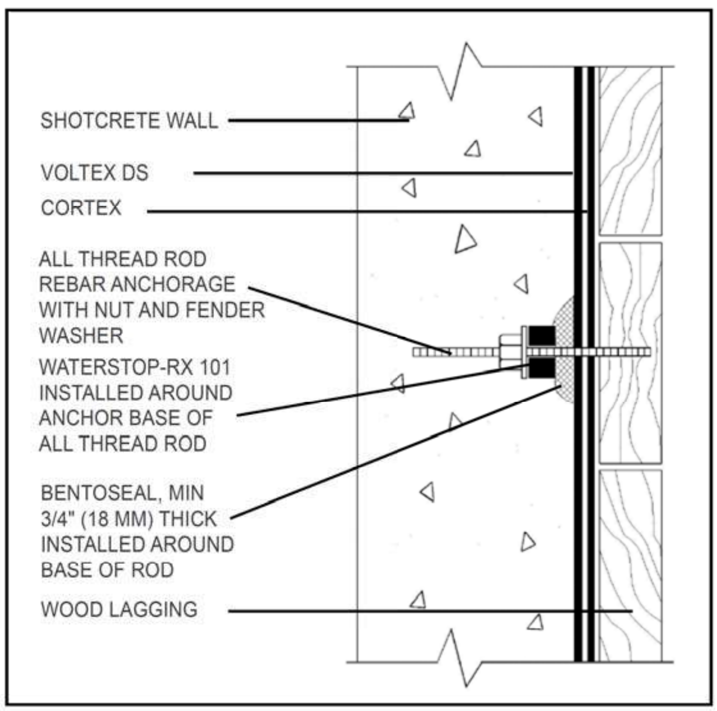


Figure H4-6A: REBAR ANCHORAGE (ALL THREAD)
Install Bentoseal ¾" (18 mm) thick and Waterstop-RX 101 around all rebar anchorage penetrating Voltex DS. Tighten nut and fender washer down all thread rod until compressing RX-101.

VOLTEX DS SHOTCRETE HYDROSTATIC

CETCO®

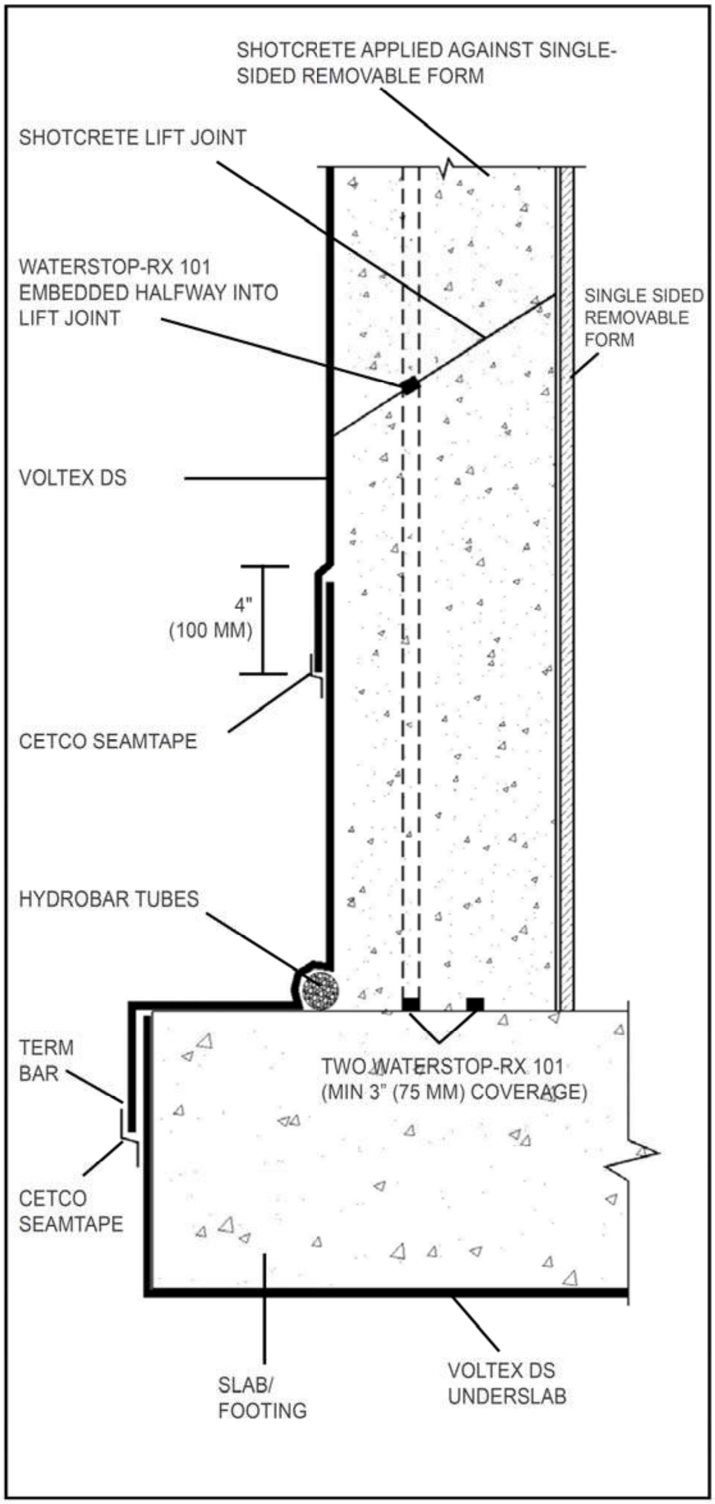


Figure H5-1: HYDROSTATIC BACKFILLED SHOTCRETE WALL

After form removal and surface prep, install Voltex DS to exterior surface of shotcrete wall with all seams overlapped minimum 4" (100 mm). Apply CETCO Seamtape to all Voltex DS overlap seams.

SECTION - H5 BACKFILLED WALLS

For backfilled walls constructed with shotcrete applied against single-sided removable form, install course of Voltex DS (dark gray geotextile side against concrete) to the exterior surface of the shotcrete wall. Install waterproofing only after inspection and repair of shotcrete wall surface. Shotcrete wall should be sound and without any defects including non-consolidated or irregular surface areas.

Prior to waterproofing, install line of Hydrobar Tubes at the wall/footing corner. Install base course of Voltex DS membrane horizontally oriented with the bottom edge extending to overlap underslab waterproofing a minimum of 6" (150 mm). Secure Voltex DS with washer-head fasteners and overlap sheet edges minimum 4" (100 mm) during the installation of both courses. Apply CETCO Seamtape to all Voltex DS overlap seams.

After the bottom horizontal course, Voltex DS sheets can be installed either vertically or horizontally oriented. Continue Voltex DS installation up wall to finished grade elevation detail, or as specified, staggering all sheet roll ends of adjacent courses a minimum 12" (300 mm). Do not allow horizontal Voltex DS overlap joints to run at same elevation as the shotcrete lift joints. Plan by chalk lining the location of shotcrete lift joint lines.

Refer to Section - H6 for applicable grade termination detailing and backfill operation guidelines.

Inspect completed waterproofing installation and repair any damaged material prior to backfill placement.

NOTE: Reinforced shotcrete walls shall conform to ACI 506 Core Grade 1 or 2. Do not use stay-in-place concrete forming; use removable forming products only.

SECTION - H6 EXCAVATION, BACKFILL & GRADE TERMINATION

Coordinate with excavation and backfill operations conducted under Division 31 Work to remove the top few wood lagging members and top of the steel soldier piles. Identify and repair any waterproofing damaged by excavation and removal of soldier pile heads and lagging.

Terminate Voltex DS and Cortex at grade detail with metal termination bar fastened 12" (300 mm) on center to exterior of shotcrete wall (Figure H5-2). Install 1/2" (12 mm) thick, continuous bead of CETSEAL centered on top edge of Voltex DS/Cortex system. Fully adhere 18" wide GF-40SA grade flashing strip to concrete wall with bottom edge overlapping top of Voltex DS/Cortex termination minimum 4" (100 mm). Complete detail with 3/8" bead of CETSEAL along top edge and overlap seams of GF-40SA flashing strip.

Care should be exercised during backfill operation to avoid damage to the waterproofing system. Division 31 backfill Work should follow generally accepted practices for backfilling and compaction of soil. Backfilled soils should be added in 6” to 12” (150 - 300 mm) lifts and compacted to a minimum 85% Modified Proctor density. Compacted aggregate backfill should be limited to 3/4” (18 mm) or less in size; non-washed gravel with fines included.

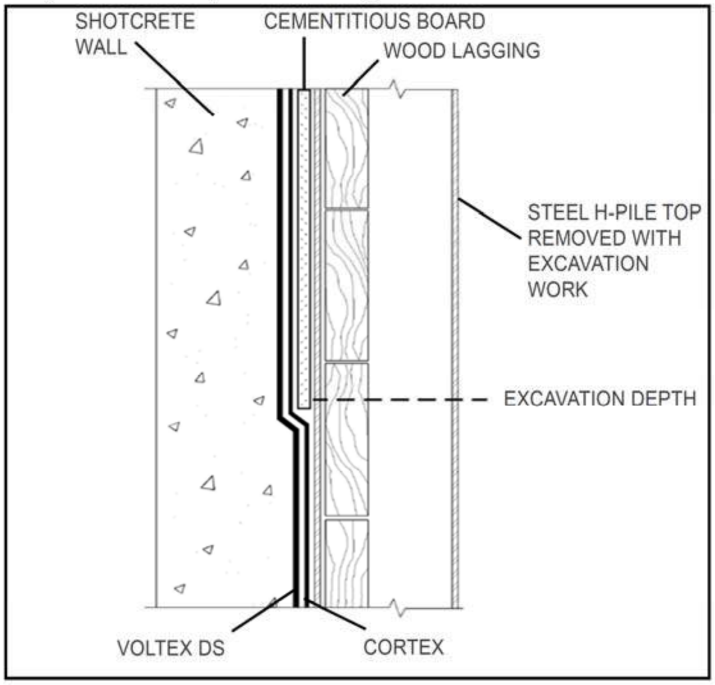


Figure H5-1: WALL EXCAVATION AT GRADE
Cementitious board protects waterproofing during excavation and removal of steel pile top and wood lagging.

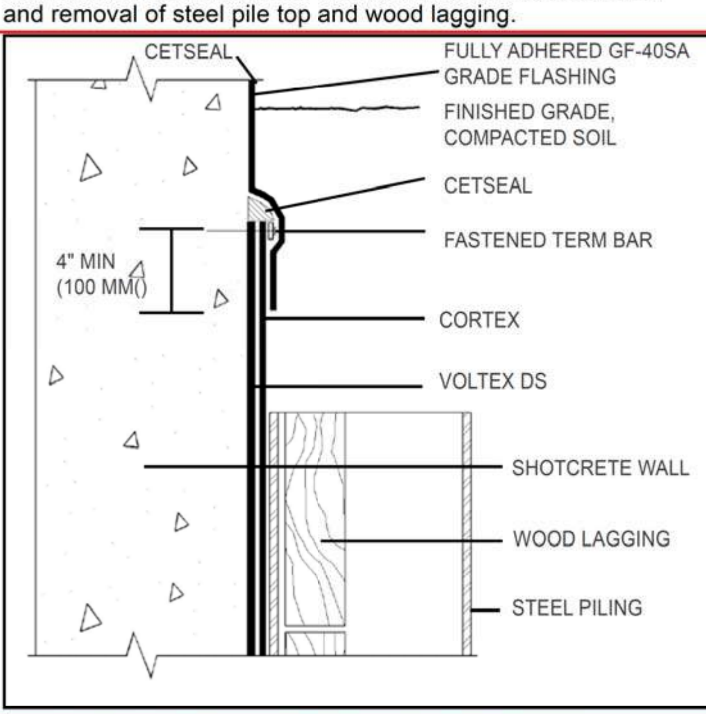


Figure H5-2: FULLY ADHERED GRADE FLASHING
Fully adhere 18" (450 mm) wide GF-40SA flashing strip to concrete wall with bottom 4" (100 mm) overlapping top of Voltex DS/Cortex installation.

INSTALLATION GUIDELINES FOR NON-HYDROSTATIC CONDITIONS

This section of the manual only covers the installation of the Voltex DS waterproofing system on foundation shoring walls where shotcrete will be applied as the structural wall under non-hydrostatic conditions with an operable water collection and discharge system. Non-hydrostatic condition means that the entire structure will be constructed above the historical high ground water elevation as determined by the projects geotechnical report. Before installing Voltex DS, read this installation section to gain familiarity with specific procedures and applications.

SECTION - NH1
GENERAL GUIDELINES

Install Voltex DS Waterproofing System with the dark gray geotextile side facing the installer so that the shotcrete will be shot against the dark gray geotextile side.

On shoring walls to receive shotcrete, install Voltex DS with minimum 6" (150 mm) sheet edge overlaps fastened with both washer-head fasteners placed 24" (600 mm) on center and pneumatic staples placed 6" (150 mm) on center. Install pneumatic staples within 1" (25 mm) of sheet edge to tightly secure entire overlap assembly to the shoring wall.

Secure center line of Voltex DS sheets to shoring wall with pneumatic staples or washer-head fasteners as required to hold membrane tight against shoring wall.

Voltex DS waterproofing system installation on a non-hydrostatic shotcrete foundation wall requires that the Aquadrain sheet and 100BD base drain composite system be connected to an operative water discharge system (sump pump or gravity to daylight). If the drainage system will not be connected to a operable water discharge system, instead of Aquadrain, install a course of Cortex over the shoring wall prior to installation of the Voltex DS waterproofing system.

Protect waterproofing products from hydrating before material is contained with concrete, shotcrete, or backfill. After any precipitation, standing water should be pumped off waterproofing as soon as possible.

Voltex DS waterproofing is not an expansion joint filler or sealant, but may be used as an expansion joint cover over properly installed expansion joint material placed during substrate preparation.

Protect adjacent work areas and finish surfaces from damage or contamination from waterproofing products during installation operations.

VOLTEX DS SHOTCRETE NON-HYDROSTATIC

Shoring Wall: Excavation contractor should provide shoring wall in good condition to receive waterproofing system. Shoring should extend to the lowest level of the waterproofing installation with any voids or cavities exterior of the shoring filled with compacted soil or cementitious grout. With lagging, interior surface of lagging boards should be uniform and tight together with gaps less than 1" (25 mm). Gaps in excess of 1" (25 mm) should be filled with cementitious grout or CETCO approved polyurethane foam. Irregular lagging may require liner prior to waterproofing installation.

SECTION - NH2 AQUADRAIN INSTALLATION

At the base of the lagging wall, install Aquadrain 100BD base-drain horizontally oriented with the open core edge up and the 2" (50 mm) fabric flap side away from the lagging wall. Secure the bottom edge of 100BD to the lagging wall with washer-head fasteners every few feet. Use couplers and corner fittings, as required, to form a continuous 100BD installation. Install discharge outlet fittings to connect with operable discharge pipes as required for the project.

Install the bottom course of Aquadrain sheet drainage horizontally oriented (geotextile side against the lagging wall) with the sheet drain bottom edge fabric flap tucked behind the top edge of the 100BD against the lagging to prevent the passage of soil into the core at the connection. Bottom edge of sheet drain core should be in contact with open top core edge of 100BD. Place the 2" (50 mm) fabric flap of the 100BD over the back of the sheet drain core and secure it with CETCO Seamtape. Secure the top edge of 100BD to the shoring wall with washer-head fasteners 24" (600 mm) on center.

Install subsequent rolls of Aquadrain sheet drainage to within 12" (300 mm) of finished grade or as shown on the project drawings. Tightly abut adjoining sheet drain core edges and tuck the extra fabric flaps behind the adjacent roll edge to keep soil from entering the sheet drain. Another installation method is to overlap the drain sheet core edges in a manner that sheds water to the outside. Secure sheet drain to shoring wall with fasteners.

Prior to installing Aquadrain sheet drainage composite near grade detail, install ½" (12 mm) thick cementitious wallboard centered over metal soldier pile from finished grade elevation to specified depth of soldier pile removal (Figure NH2-2). Cementitious wall board (Durock) will protect drainage and waterproofing when top of soldier pile is excavated and removed. Remove cementitious board with removal of soldier pile top and lagging.

Around penetrations and tie-back heads, cut sheet drainage composite to fit and wrap extra filter fabric around open core edge to prevent soil from entering core.

At the top of the sheet drain installation, wrap the filter fabric flap behind the exposed top core edge to prevent intrusion of soil into the core and secure sheet drain to wall with termination bar fastened 12" (300 mm) on center with the fabric wrapped.

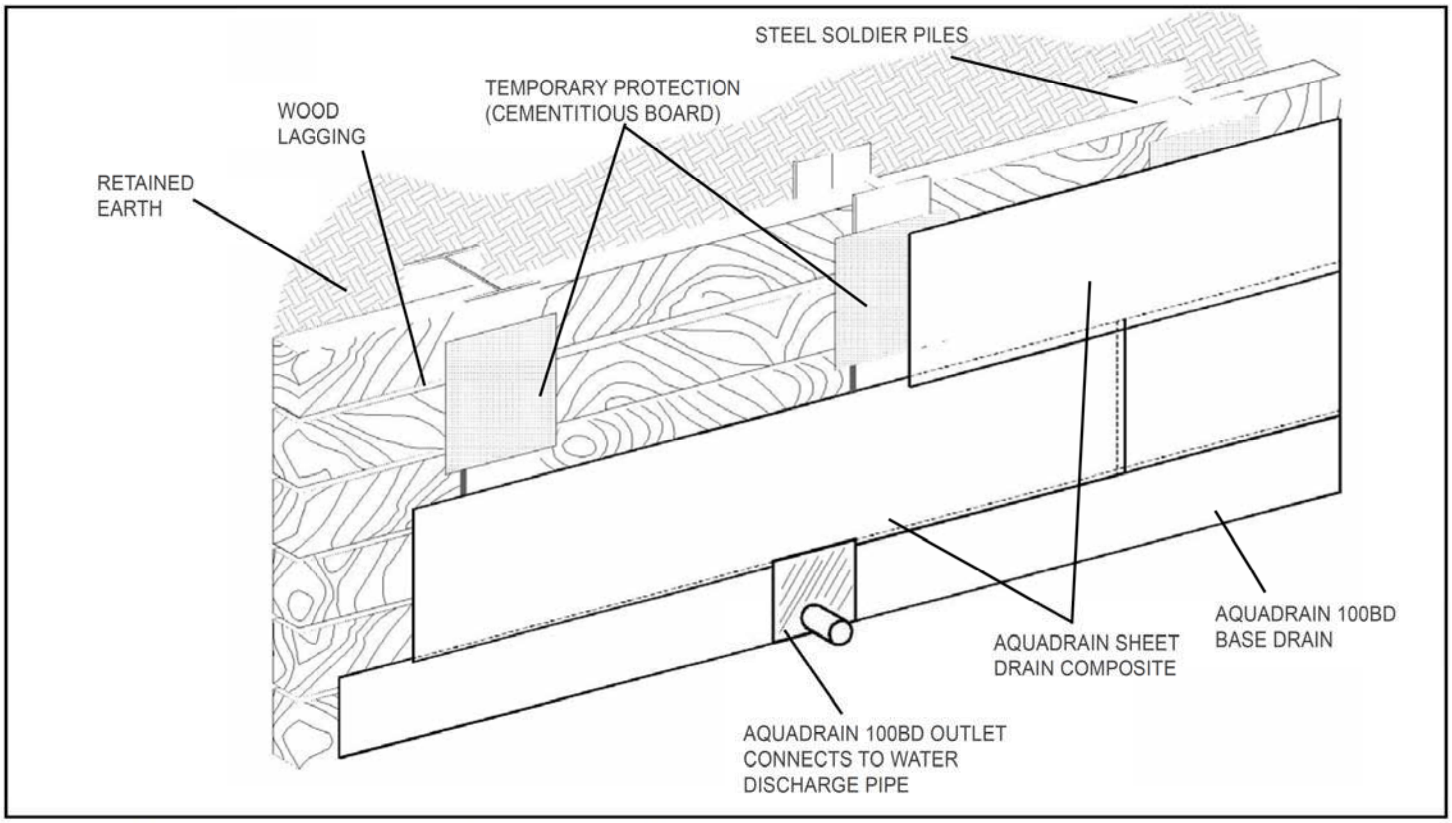


Figure NH2-1: AQUADRAIN SHEET DRAINAGE INSTALLED ONTO SHORING WALL PRIOR TO VOLTEX DS
Install Aquadrain sheet drainage over shoring wall prior to installing Voltex DS waterproofing membrane. Aquadrain should be applied from base of wall to grade level unless otherwise specified per project documents.

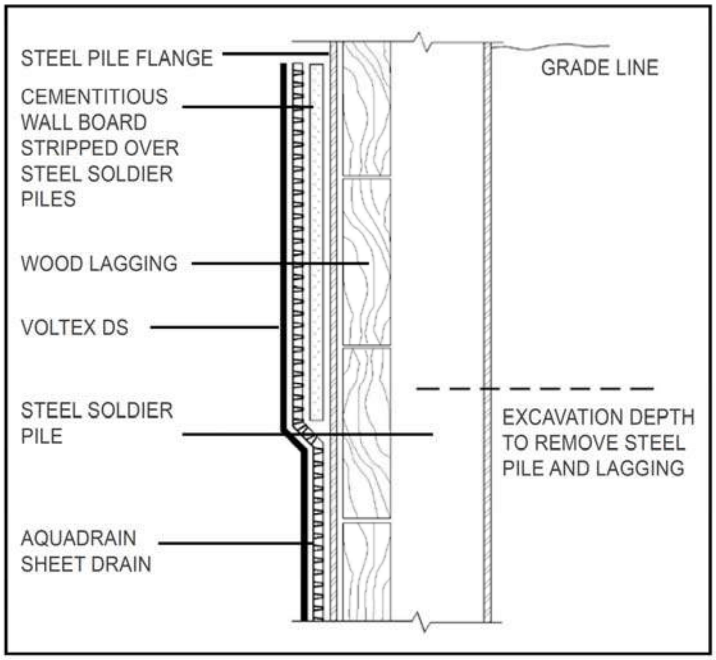


Figure NH2-2: CEMENTITIOUS BOARD AT GRADE
Install cementitious board strip over steel piling at grade to protect waterproofing during removal of top lagging boards and top of steel pile.

SECTION - NH3
SLAB TO WALL TRANSITION

Starting at the base of the shoring wall, install Voltex DS waterproofing over the Aquadrain drainage system previously installed in accordance with Section NH2.

For the slab to wall corner transition, install Voltex DS sheet horizontally oriented (dark gray geotextile side facing installer) with a minimum 12" (300 mm) of the bottom edge extending out onto the horizontal substrate. The top edge of the sheet must extend a minimum 12" (300 mm) above the finished slab surface. Secure Voltex DS sheet to lagging wall with washer-head fasteners maximum 24" (600 mm) on center. Overlap edges of adjacent Voltex DS sheets a minimum 6" (150 mm).

Install second Voltex DS sheet course horizontally oriented (dark gray geotextile side facing installer) onto the lagging wall over the corner transition sheet, with the bottom edge extending down to the wall/slab transition corner as shown in Figure NH3-1. Secure Voltex DS to lagging wall with washer-head fasteners maximum 24" (600mm) on center. Overlap edges of adjacent Voltex DS sheets a minimum 6" (150 mm). Further secure membrane overlap edges with pneumatic staples placed 6" (150 mm) on center within 1" (25 mm) of sheet edge.

Install underslab Voltex DS membrane extending to corner transition (dark gray geotextile side up), overlapping the 12" (300 mm) Voltex DS tail extending from Voltex DS corner transition sheet installed at the wall base. Secure corner edge of membrane with washer-head fasteners or pneumatic staples 12" (300 mm) on center.

If slab is greater than 24" (600 mm) thick, consult CETCO for guidelines.

NOTE: Reinforced shotcrete walls shall conform to ACI 506 Core Grade 1 or 2. Do not use stay-in-place concrete forming; use removable forming products only.

VOLTEX DS SHOTCRETE NON-HYDROSTATIC

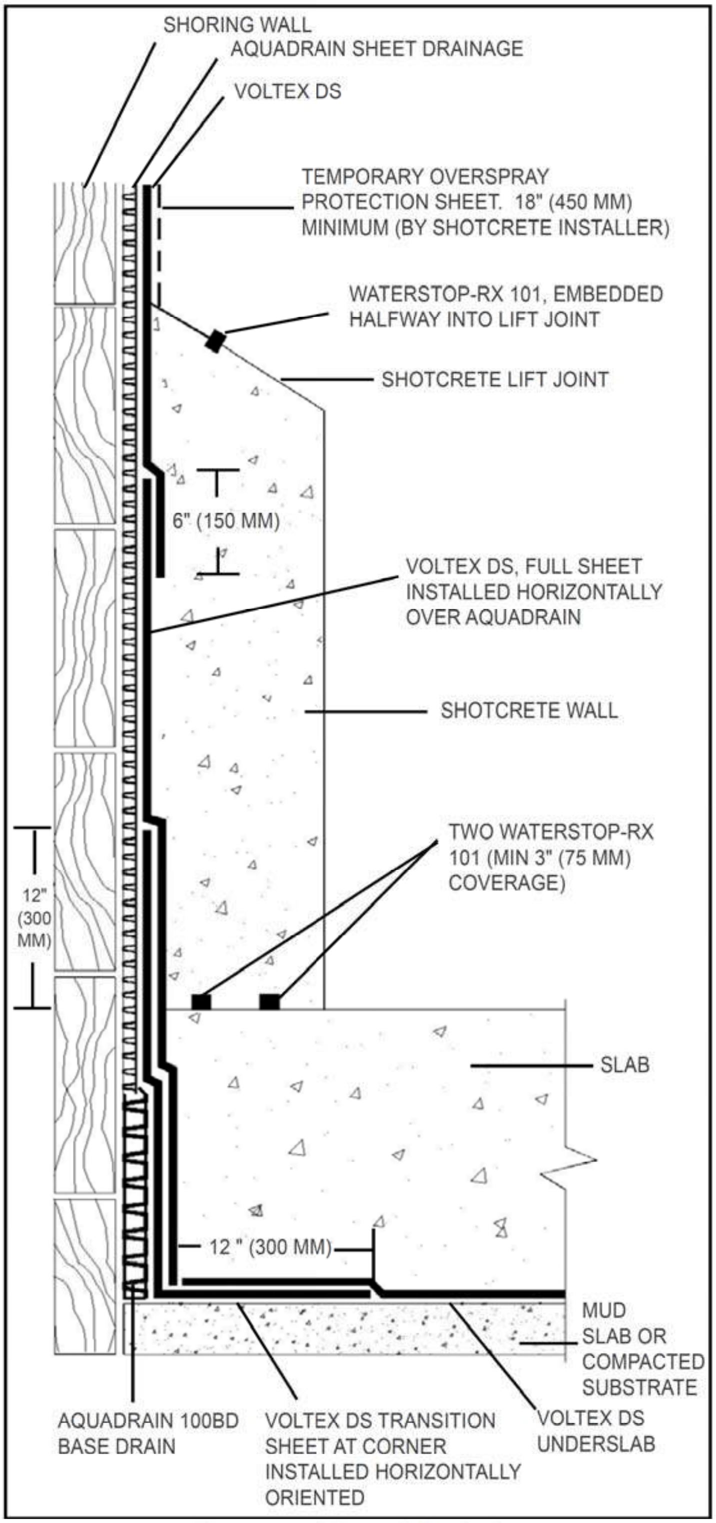


Figure NH3-1: NON-HYDROSTATIC SHOTCRETE WALL
Transition requires Voltex DS sheet installed horizontally over Aquadrain at the corner to extend 12" (300 mm) onto underslab substrate. Then install another Voltex DS sheet to lagging wall (horizontally oriented) over corner transition sheet. Install the underslab Voltex DS sheet to the slab/wall corner overlapping the Voltex DS transition sheet a minimum 12" (300 mm).

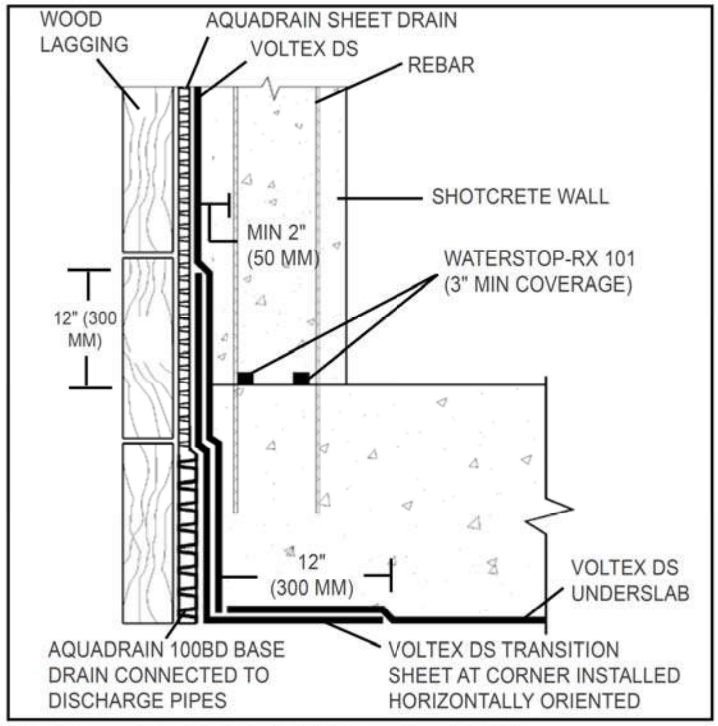


Figure NH3-2: SLAB TO WALL TRANSITION
Voltex DS sheet installed at the corner should extend past the height of the top of the finished slab level a minimum 12" (300 mm) and extend under the slab 12" (300 mm).

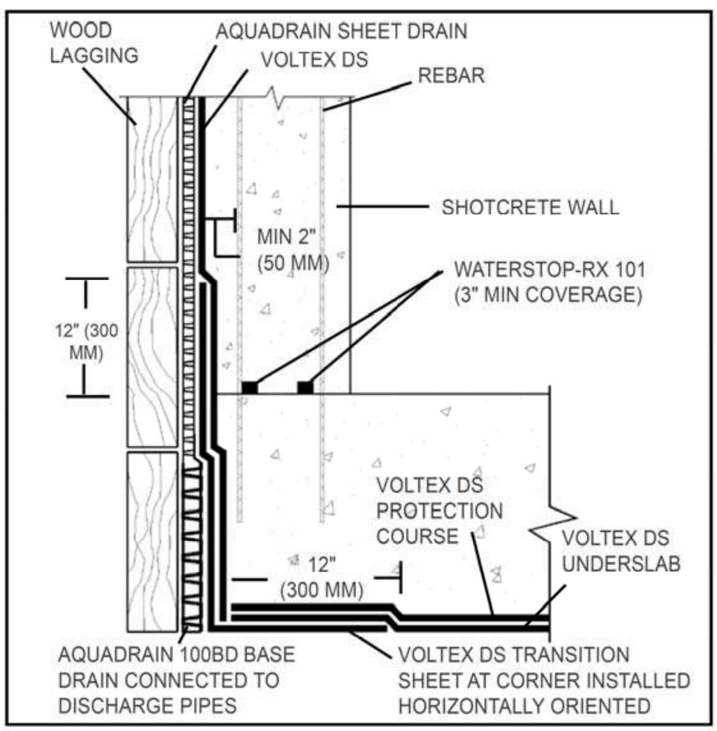


Figure NH3-2A: VOLTEX DS PROTECTION LAYER
As specified to provide protection, install a second course of Voltex DS over the first course of Voltex DS in lieu of concrete protection slab

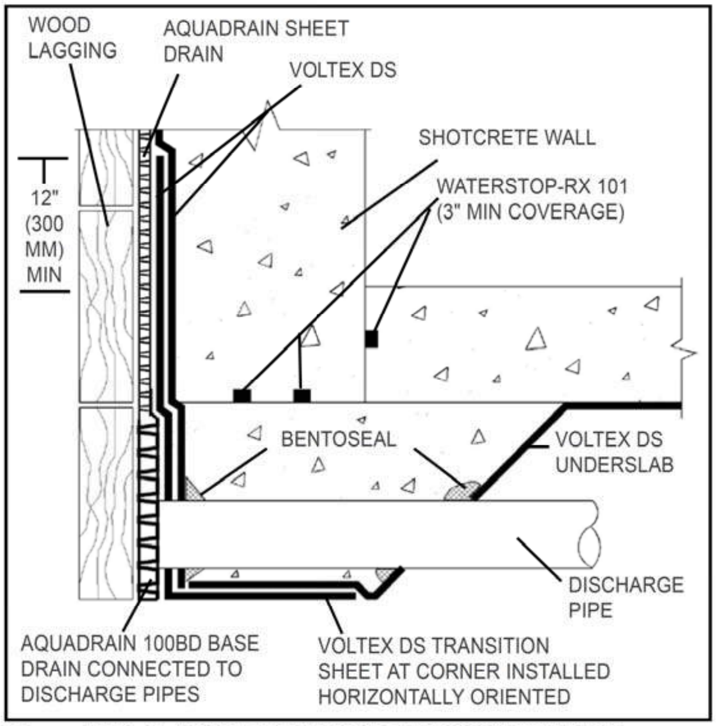


Figure NH3-3: AQUADRAIN 100BD DISCHARGE PIPE
Connect Aquadrain 100BD to water discharge pipes using 100BD accessory connectors.

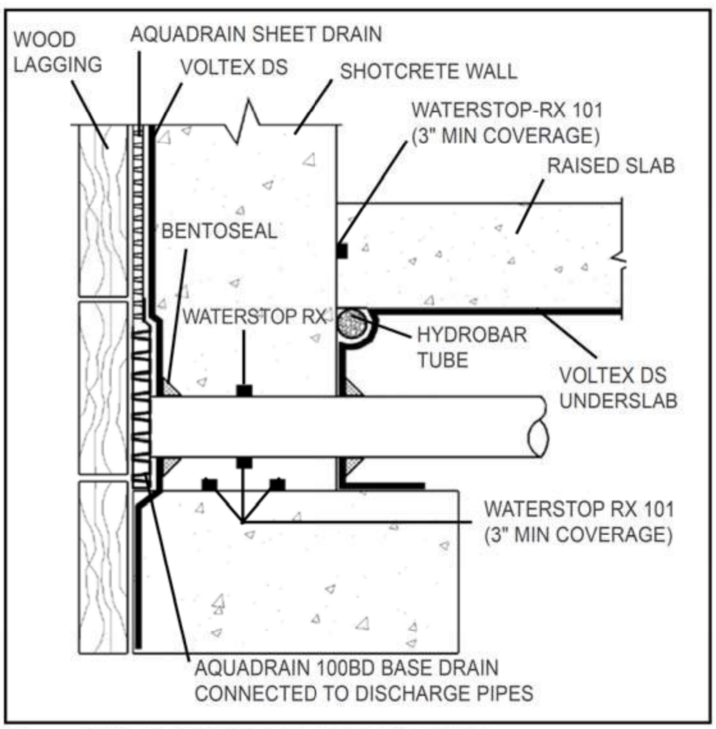


Figure NH3-4: RAISED SLAB CONDITION
Connect Aquadrain 100BD to water discharge pipes using 100BD accessory connectors.

SECTION - NH4
SHORING WALL

For HydroShield Warranty eligibility, CETCO requires that all heavily reinforced columns and pilasters integrated with or attached to the structural shotcrete foundation wall be formed and cast-in-place (Figure NH4-1). It is extremely difficult to place and consolidate shotcrete behind heavy or closely spaced steel reinforcement. Require the cast-in-place columns and pilasters be constructed prior to the connecting shotcrete walls. This will limit the possibility of overspray contamination to the column steel reinforcement and the column sides create a stable form surface to apply the adjacent shotcrete walls.

Membrane Installation: Install Voltex DS sheet to shoring wall (dark gray geotextile side facing installer), over Aquadrain sheet drainage, overlapping top edge of the previously installed Voltex DS corner transition a minimum 6" (150 mm) (Figure NH3-1). Overlap all adjacent sheet edges a minimum 6" (150 mm). Secure edge overlaps with both washer-head fasteners placed maximum 24" (600 mm) on center and pneumatic staples placed 6" (150 mm) on center within 1" (25 mm) of overlap edge. Secure center line of Voltex DS sheets to shoring wall with pneumatic staples or washer-head fasteners as required to hold membrane tight against shoring wall.

After the bottom horizontal course, Voltex DS sheets can be installed either vertically or horizontally oriented. Continue installation up wall until finished grade elevation detail, or as specified, staggering all sheet roll ends of adjacent courses a minimum 12" (300 mm). Do not allow horizontal Voltex DS overlap joint to run at same elevation of the shotcrete lift joint. Plan by chalk lining the location of lift joint lines.

Inspect finished Voltex DS installation and repair any damaged material prior to shotcrete placement. Assure Voltex DS overlap is not separated during shotcrete placement.

Tie-Back Covers: Select appropriate size TB-Boot to fit over tie-back plate and allow proper concrete coverage per project requirements. TB-Boot should fit entirely over tie-back head without the tie-back plate or cables in direct contact with the TB-Boot. Prior to TB-Boot installation, fill voids in retention wall substrate and tie-back head assembly with spray foam (min. 20 PSI) or non-shrink grout. Install and secure Aquadrain drainage composite course per manufacturer's guidelines to soil retention wall prior to installing TB-Boot.

Fill pre-formed of TB-Boot with 2-part urethane spray foam (min. 20 PSI) and place over tie-back head before foam sets up. Secure TB-Boot to soil retention system with washer-head fasteners along the outside edge of the flat base. Apply ¼" (6mm) thick by 3" (75mm) wide continuous ring of Bentoseal onto the flat base just to the outside of the ½" (12mm) raised collar. Install Voltex DS membrane overlapping the entire flat base to the ½" (12mm) raised collar. Secure the Voltex DS sheet edge with washer-head fasteners just outside the ½" (12mm) raised collar so that the fasteners pass through the Bentoseal ring; typical fastener spacing 6" (150mm) on center. Do not install fasteners or puncture TB-Boot inside of the ½" raised collar. Complete detail by applying continuous counter flashing of Bentoseal along Voltex DS field sheet edge.

VOLTEX DS SHOTCRETE NON-HYDROSTATIC

For soil nail rod and plate assemblies, install applicable TB-Boot over assembly and fasten to shoring wall through Aquadrain. Install Voltex DS with Bentoseal detailing per TB-Boot installation guidelines herein.

Penetrations: Install a cut collar of Voltex DS tightly around the penetration; extending Voltex DS around penetration a minimum 12" (300 mm) radius. Apply Bentoseal over Voltex DS collar around penetration; extending Bentoseal a minimum 3" (75 mm) radius at minimum ¼" (6 mm) thickness. Then install main course of Voltex DS membrane tightly around the penetration. Finally, detail around penetration with ¾" (18 mm) thick cant of Bentoseal mastic. With sleeved penetrations, fill the gap between the pipe and the sleeve with Department of Transportation (DOT) non-shrink grout or 2-part polyurethane spray foam (min. 20 psi) and install mechanical seal (by Others) around the pipe (Figure NH4-5A).

Rebar Anchorage: Install Bentoseal ¾" (18 mm) thick around outward end of all rebar anchorage penetrating Voltex DS. Then install a length of Waterstop-RX around the shaft of the rebar anchorage (Figure NH4-6) securing it with plastic zip tie or rebar wire.

Waterstop: As part of Division 3 Shotcrete Work, the shotcrete contractor shall install one strip of Waterstop-RX 101 into each shotcrete lift joint, and two strips of Waterstop-RX 101 at all construction cold joints regardless of whether the joint will encounter hydrostatic or non-hydrostatic conditions. Applicable construction cold joints include the shotcrete wall to slab/footing; vertical formed work edge joints; shotcrete to cast-in-place columns; and daily shotcrete lift joint stops. Refer to Waterstop-RX product literature for installation guidelines.

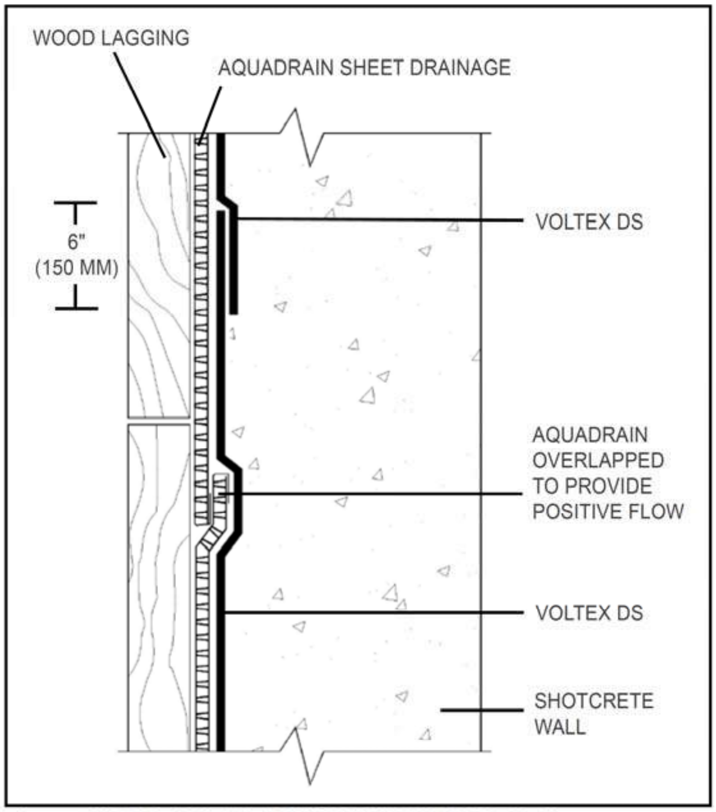


Figure NH4-2: OVERLAP VOLTEX DS EDGES 6"
Overlap all Voltex DS membrane edges a minimum 6" (150 mm). Fasten membrane edges 6" (150 mm) on center with staples within 1" (25 mm) of membrane edge.

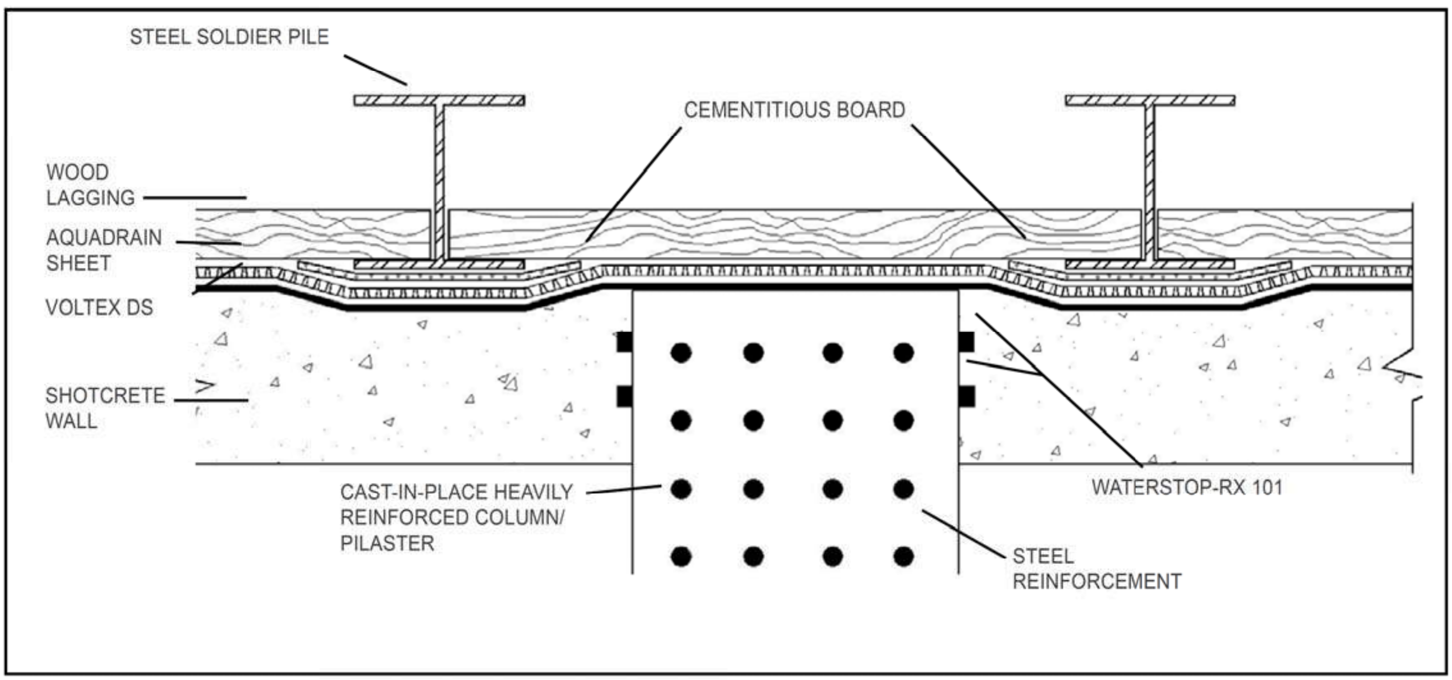


Figure NH4-1: CAST-IN-PLACE STRUCTURAL COLUMNS AND PILASTERS
CETCO HydroShiled eligibility requires that these structural components be cast-in-place (Figure H4-1). Sequence integrated cast columns and pilasters prior to spraying the connecting or adjoining shotcrete walls in order to minimize the impact of overspray and maintain the cleanest surface possible to pour against in these locations.

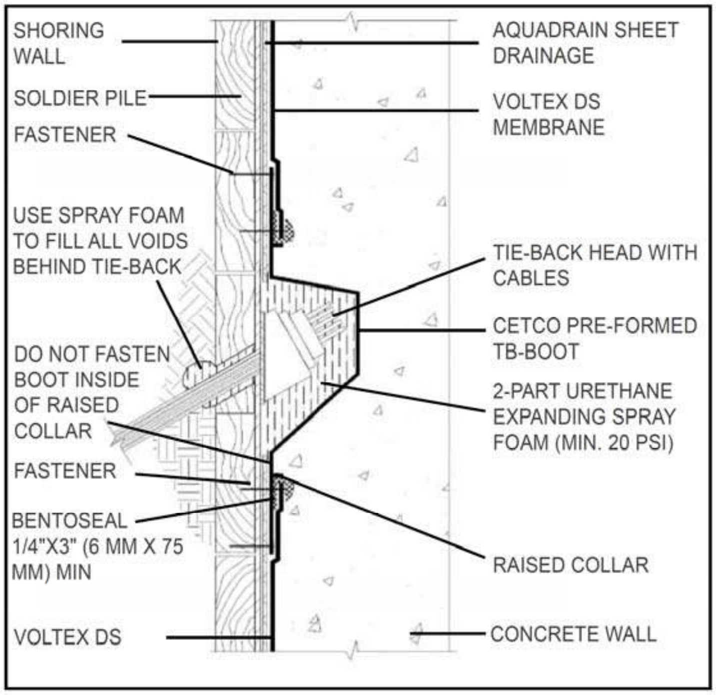


Figure NH4-3: TIE-BACK DETAIL
Install TB-Boot centered over tie-back then install main course of Voltex DS with Bentoseal detailing. Do not fasten boot inside of raised collar around center formed area.

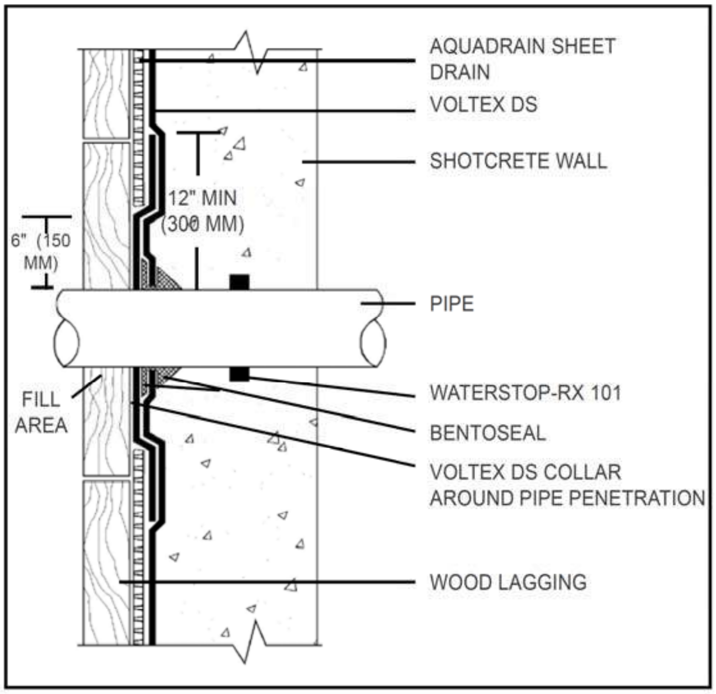


Figure NH4-5: WALL PENETRATION
Cut and secure Voltex DS tightly around penetrations and then apply Bentoseal ¾" (18 mm) ring around penetration and extend over membrane a minimum 6" (150 mm) radius at minimum ¼" (6 mm) thickness.

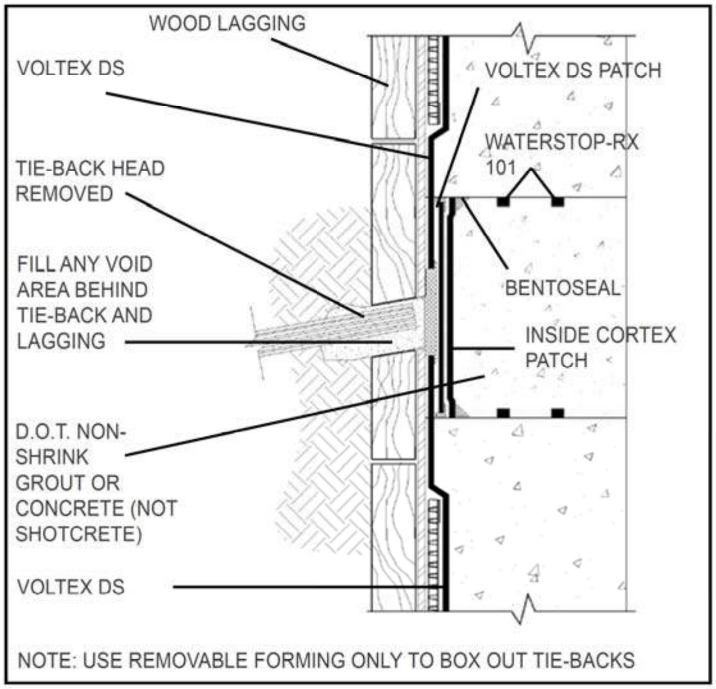


Figure NH4-4: TIE-BACK BOX OUT DETAIL
After tie-back head removal, complete detail by installing Voltex DS patch, Bentoseal, inside Cortex patch and Waterstop-RX. Only use D.O.T. approved non-shrink grout or concrete to fill box out (no shotcrete).

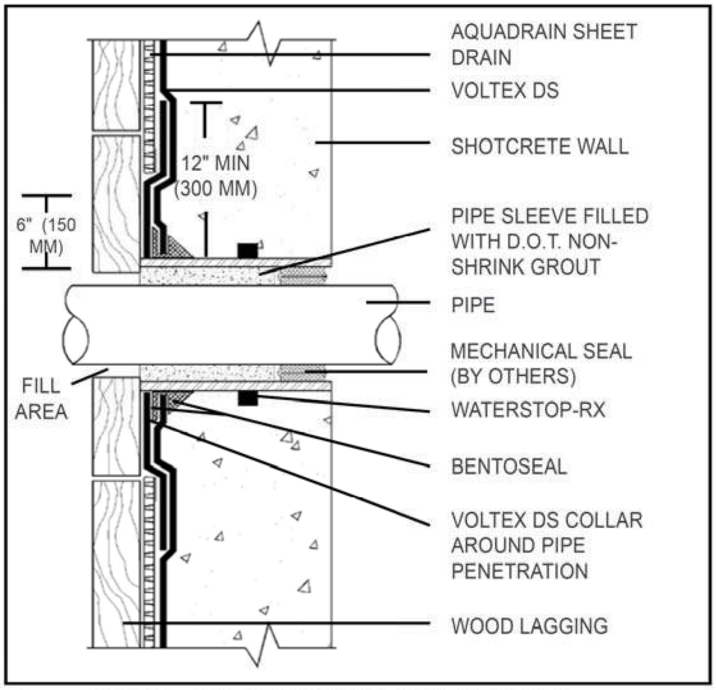


Figure NH4-5A: SLEEVED WALL PENETRATION
Cut and secure Voltex DS tightly around penetrations and then apply Bentoseal ¾" (18 mm) ring around penetration and extend over membrane a minimum 6".

VOLTEX DS SHOTCRETE NON-HYDROSTATIC

CETCO®

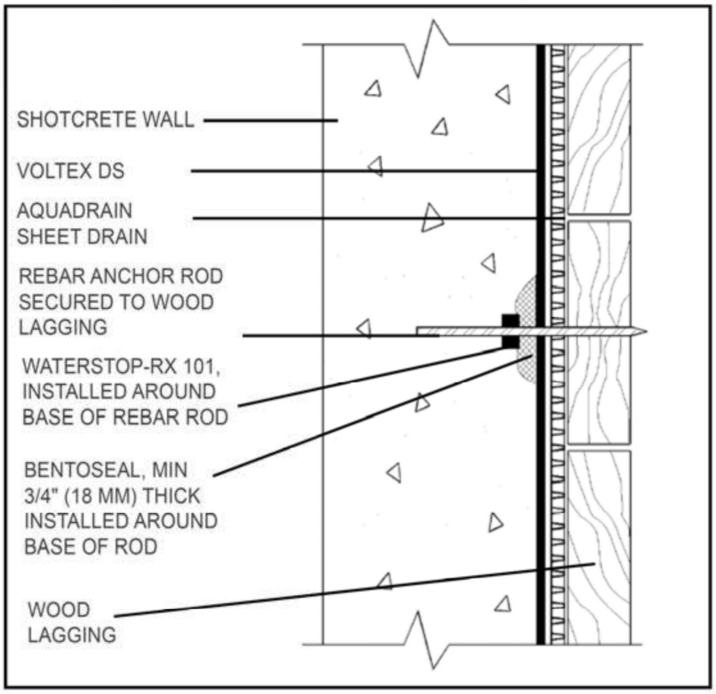


Figure NH4-6: REBAR ANCHORAGE
Install Bentoseal ¾" (18 mm) thick around all rebar anchorage penetrating Voltex DS. Then install a length of Waterstop-RX around the shaft of the rebar anchorage secured with plastic zip tie or rebar wire.

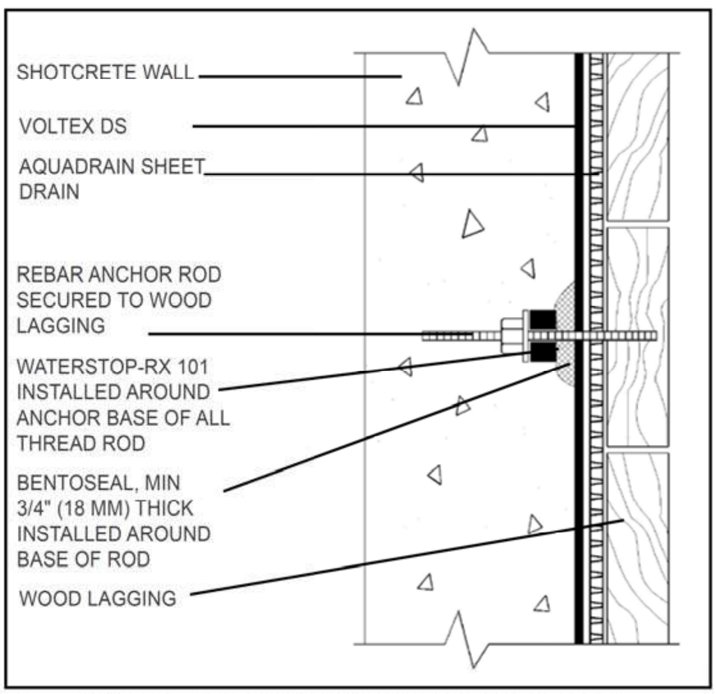


Figure NH4-6A: REBAR ANCHORAGE (ALL THREAD)
Install Bentoseal ¾" (18 mm) thick and Waterstop-RX 101 around all rebar anchorage penetrating Voltex DS. Tighten nut and fender washer down all thread rod until compressing RX-101.

SECTION - NH5 BACKFILLED WALLS

For backfilled walls constructed with shotcrete applied against single-sided removable form, install course of Voltex DS to the exterior surface of the shotcrete wall followed by course of Aquadrain drainage composite with operable Aquadrain 100BD collection and discharge system. Install waterproofing only after inspection and repair of shotcrete wall surface. Shotcrete wall should be sound and without any defects including non-consolidated or irregular surface areas.

Prior to waterproofing, install line of Hydrobar Tubes at the wall/footing corner. Install base course of Voltex DS membrane horizontally oriented (dark gray geotextile side against shotcrete) with the bottom edge extending out onto the footing a minimum 6" (150 mm). As applicable, terminate bottom edge with termination bar and Bentoseal or extend Voltex DS to overlap underslab waterproofing a minimum of 6" (150 mm). Secure Voltex DS with washer-head fasteners and overlap sheet edges minimum 4" (100 mm). Apply CETCO Seamtape to all Voltex DS overlap seams (Figure NH5-1).

After the bottom horizontal course, Voltex DS sheets can be installed either vertically or horizontally oriented. Continue Voltex DS installation up wall to finished grade elevation detail, or as specified, staggering all sheet roll ends of adjacent courses a minimum of 12" (300 mm). Do not allow horizontal Voltex DS overlap joints to run at same elevation as the shotcrete lift joint lines. Install Aquadrain drainage composite course directly over Voltex DS as specified.

Refer to Section - NH6 for applicable grade termination detailing and backfill operation guidelines.

Inspect completed waterproofing installation and repair any damaged material prior to backfilled placement.

NOTE: Reinforced shotcrete walls shall conform to ACI 506 Core Grade 1 or 2. Do not use stay-in-place concrete forming; use removable forming products only.

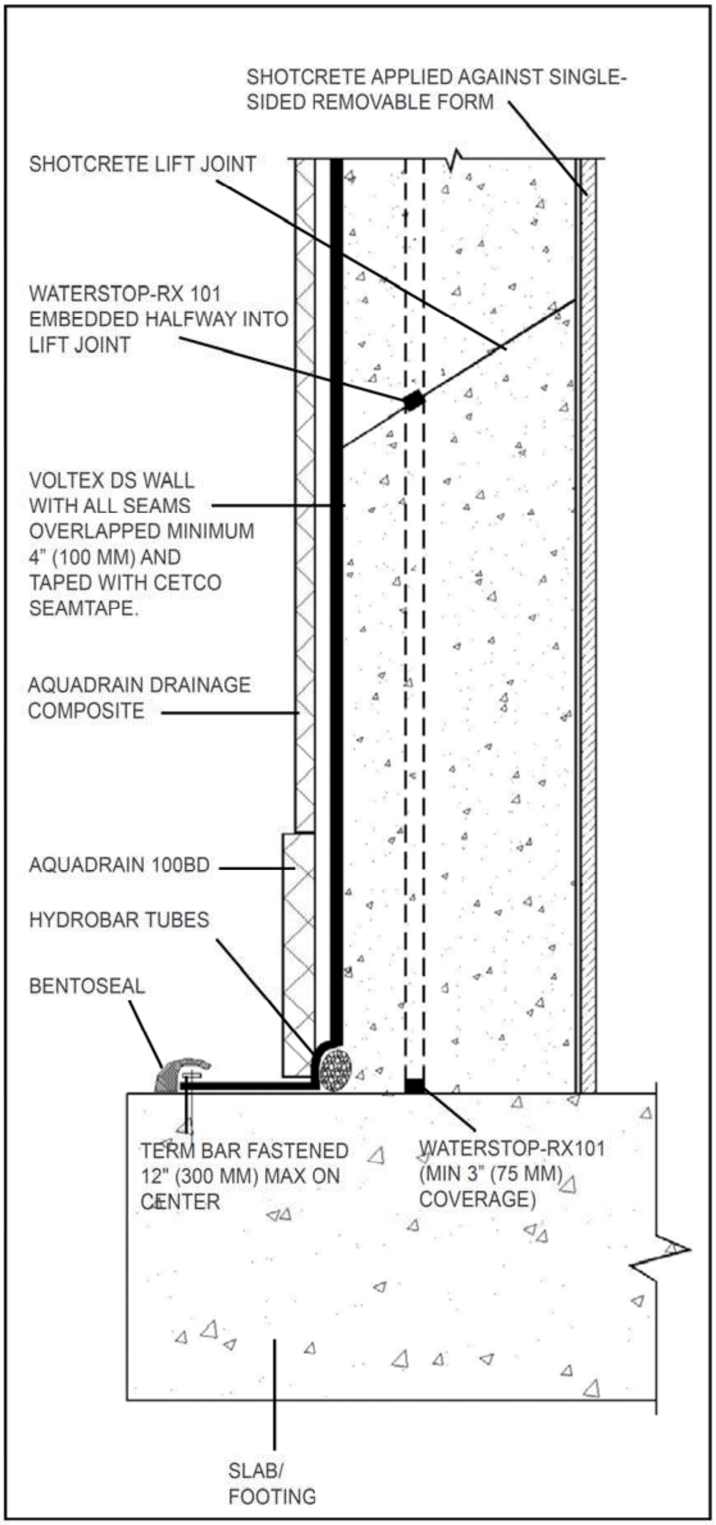


Figure NH5-1: NON- HYDROSTATIC BACKFILLED SHOTCRETE WALL
After form removal and surface prep, install course of Voltex DS to exterior surface of shotcrete wall with all seams overlapped minimum 4" (100 mm) and taped with CETCO Seamtape.

SECTION - NH6 EXCAVATION, BACKFILL & GRADE TERMINATION

Coordinate with excavation and backfill operations conducted under Division 31 Work to remove the top few wood lagging members and top of the steel soldier piles per local building code or as specified. Identify and repair any waterproofing and drainage sheet damaged by excavation and removal of soldier pile heads and lagging.

Terminate Voltex DS at grade elevation detail with metal termination bar fastened 12" (300 mm) on center to exterior of shotcrete wall. Install 1/2" (12 mm) thick, continuous bead of CETSEAL on top edge of Voltex DS system. Secure drainage sheet to shotcrete wall. Fully adhere 18" (450 mm) wide GF-40SA grade flashing strip to concrete wall with bottom edge overlapping top of Voltex DS termination minimum 4" (100 mm) (Figure NH6-1). Complete detail with 3/8" (10 mm) bead of CETSEAL along top edge and overlap seams of GF-40SA grade flashing strip.

Care should be used during backfill operation to avoid damage to the drainage and waterproofing system. Division 31 backfill Work should follow generally accepted practices for backfilling and compaction of soil. Backfilled soils should be added in 6" to 12" (150 - 300 mm) lifts and compacted to a minimum 85% Modified Proctor density. Compacted aggregate backfill should be limited to 3/4" (18 mm) or less in size; non-washed gravel with fines included.

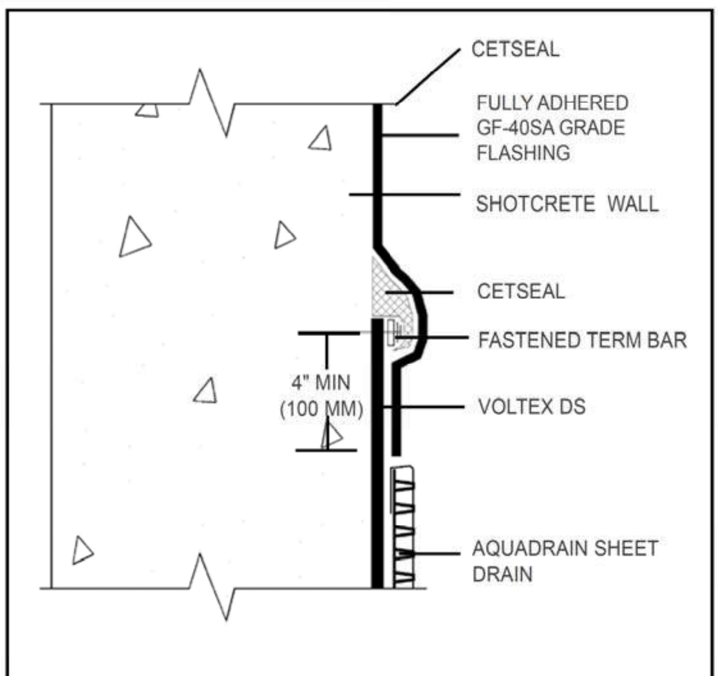


Figure NH6-1: FULL ADHERED GRADE FLASHING
Fully adhere 18" (450 mm) wide GF-40SA grade flashing strip to concrete wall with bottom 4" (100 mm) overlapping top of Voltex DS installation.

VOLTEX DS SHOTCRETE NON-HYDROSTATIC

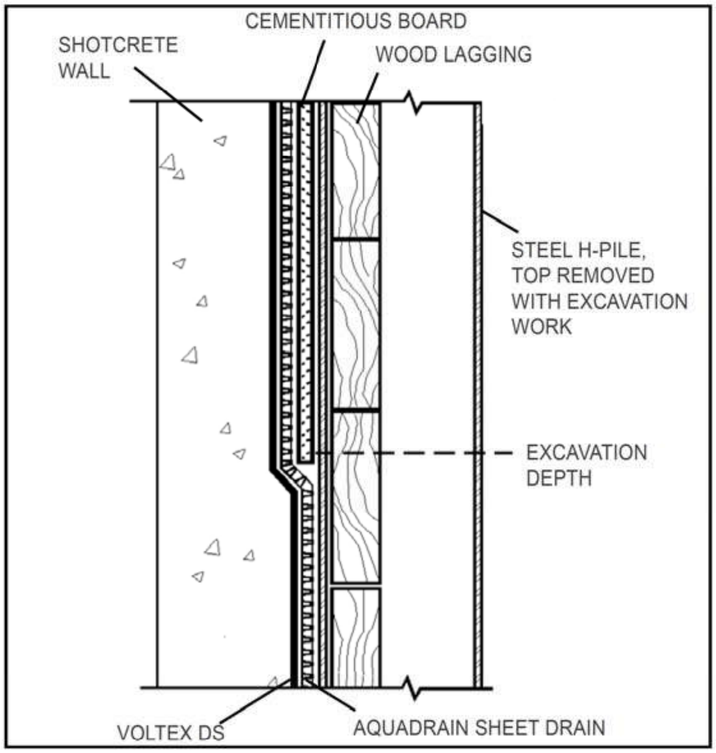


Figure NH6-2: WALL EXCAVATION AT GRADE
Cementitious board protects waterproofing during excavation and removal of steel pile top and wood lagging

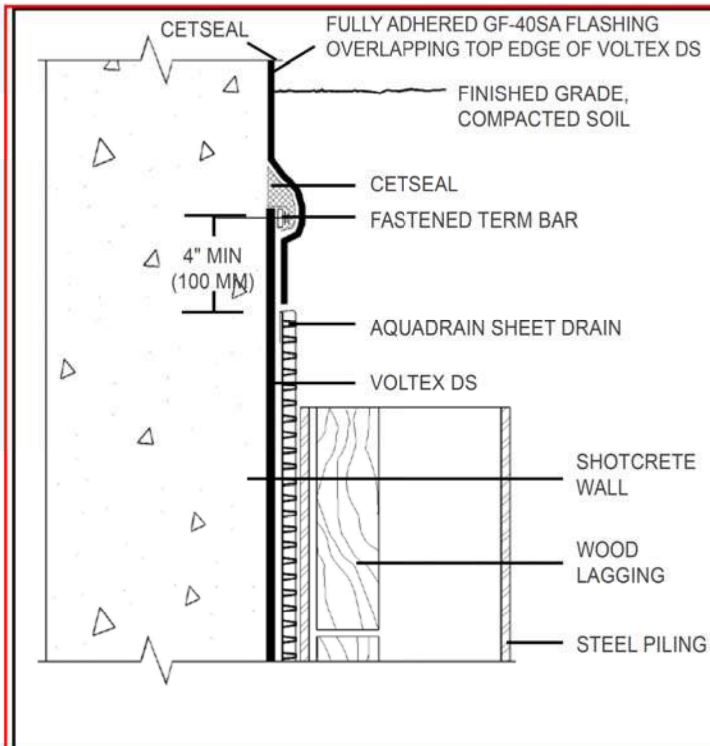


Figure NH6-3: GRADE TERMINATION
Terminate Voltex DS at grade detail with metal termination bar fastened 12" (300 mm) on center and apply CETSEAL centered on top edge system then install GF-40SA grade flashing detail .

SHOTCRETE PHOTO LIBRARY

PHOTO 1: KICKER FORM AT BASE OF SHOTCRETE WALL
Photo illustrates shotcrete placed in 24" (600 mm) kicker form without air velocity being consolidated by vibration.

SHOTCRETE PHOTO LIBRARY

PHOTO 2: SHOTCRETE FOUNDATION WALL
Photo illustrates shotcrete structural foundation wall being applied against Voltex DS/Cortex waterproofing system installed on wood lagging shoring retention wall above 24" (600 mm) kicker formed wall base.

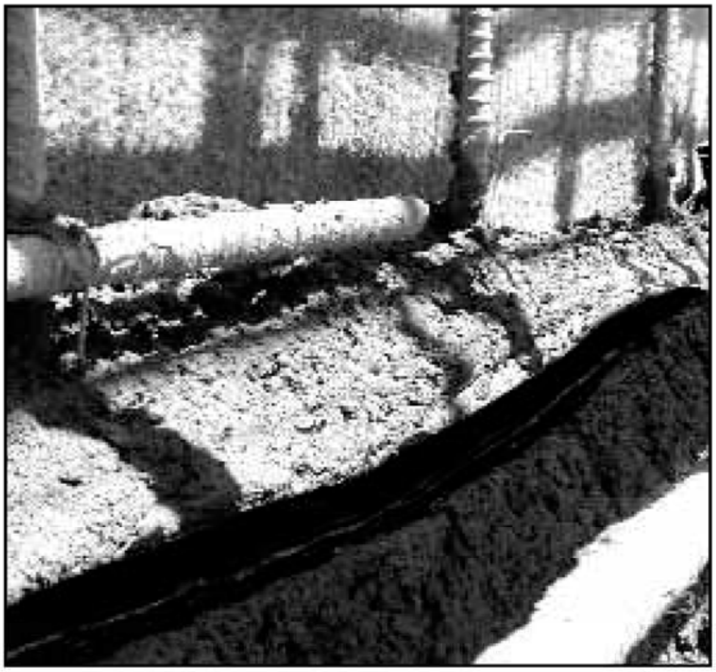


PHOTO 3: SHOTCRETE LIFT JOINT
Strip of Waterstop-RX 101 embedded in shotcrete lift joint by the shotcrete crew during the short work stoppage that allows the vertical work to continue without sloughing. Note how protective sheeting (removed) kept shotcrete overspray from contaminating Voltex DS for next lift work.

PHOTO 4: REBAR ANCHORAGE DETAILING
Trowel Bentoseal 3/4" (18 mm) thick, by 3" (75 mm) radius over Voltex DS at base of rebar anchor rod and then install strip of Waterstop-RX 101 around rod.

PHOTO 5: TB-BOOT OVER (HYDROSTATIC)
For Hydrostatic conditions, install TB-Boot over tie-back then install main course of Voltex DS with Bentoseal detailing.

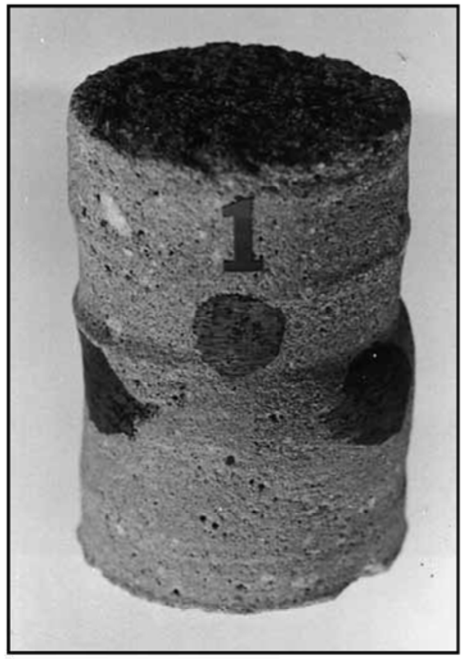


PHOTO 7: SHOTCRETE CORE GRADE 1
Shotcrete specimens are solid; there are no laminations, sandy areas or voids. Small air voids with a maximum diameter of 1/8" and a maximum length of 1/4" are normal and acceptable. Sand pockets, or voids, behind continuous reinforcement steel are unacceptable. The surface against the form of bond plane shall be sound, without a sandy texture or voids.

PHOTO 6: TB-BOOT (NON-HYDROSTATIC)
For non-hydrostatic conditions, install TB-Boot over tie-back after Aquadrain drainage composite course; then install Voltex DS with Bentoseal detailing.

PHOTO 8: SHOTCRETE CORE GRADE 2
Shotcrete specimens shall have no more than two (2) laminations or sand areas with dimensions not to exceed 1/8" thick by 1" long. The height, width and depth of voids shall not exceed 3/8". Porous areas behind reinforcement steel shall not exceed 1/2" in any direction except along the length of the reinforcing steel. The surface against the form or bond plane shall be sound, without a sandy texture or voids.

IMPORTANT NOTICE
REGARDING SHOTCRETE DESIGN AND INSTALLATION

The performance of the Voltex DS waterproofing system described herein is dependent upon the shotcrete structural foundation wall being properly designed and constructed. It is important that all the elements which will aid in providing a well designed and correctly executed shotcrete job be written into the project specifications. Shotcrete Specification Section should explicitly detail type of shotcrete process, contractor and nozzleman experience and qualifications, materials, design mix, admixtures, acceptance/rejection criteria, quality and method of substrate preparation, steel reinforcing details and anchorage, construction joints, finish, curing procedure as well as the quality assurance methods and requirements that will be employed.

Require that shotcrete walls be placed in strict accordance with ACI 506.2-95 Core Grade 1 or 2, which includes, but not limited to, gunning all walls from the bottom up to their full design thickness in a single application, maintaining the lift height to a maximum of 4 feet (1.2 m), dampening absorptive substrates before placing shotcrete, removing rebound and sand pockets, properly encasing steel reinforcement, and properly curing the shotcrete installation. Stay-in-place forming products should not be used; use only conventional removable concrete forms.

As a condition of HydroShield warranty eligibility on shotcrete foundation wall applications, CETCO requires independent, third party inspection services hired by owner; to monitor shotcrete placement and to ensure that the shotcrete grades and minimum design requirements specified are met through a well executed, independently administrated quality assurance process. Also, specify the shotcrete subcontractor to have a minimum of three (3) years successful experience in structural shotcrete work of similar scope, mix type, and project size. Require shotcrete subcontractor to submit written evidence giving qualifications and experience of foreman, delivery equipment operator, nozzle helper (rebound cleaner), and nozzleman certifying that each has experience with the specified shotcrete mix type and the application technique required for the Work. Require that the nozzleman be ACI 506.3R-91 certified or other equivalent certification.

CONTACT CETCO FOR A COMPLETE LIST OF SHOTCRETE SPECIFICATION REQUIREMENTS TO COMPLY WITH ELIGIBILITY OF HYDROSHIELD WARRANTY. CONSULT WITH CETCO FOR APPLICATIONS NOT COVERED HEREIN.

CETCO®
phone: 800·527·9948 ■ 847·851·1800
www.cetco.com

LIMITED WARRANTY

The information and data contained herein is believed to be accurate and reliable. Specifications and other information contained herein supersede all previously printed material and are subject to change without notice.

Manufacturer's warranty of installed system is available. Contact seller for terms and sample documents including all limitations.

All goods sold by seller are warranted to be free from defects in material and workmanship.

The foregoing warranty is in lieu of and excludes all other warranties not expressly set forth herein, whether expressed or implied by operation of law or otherwise including but not limited to any implied warranties of merchantability or fitness.

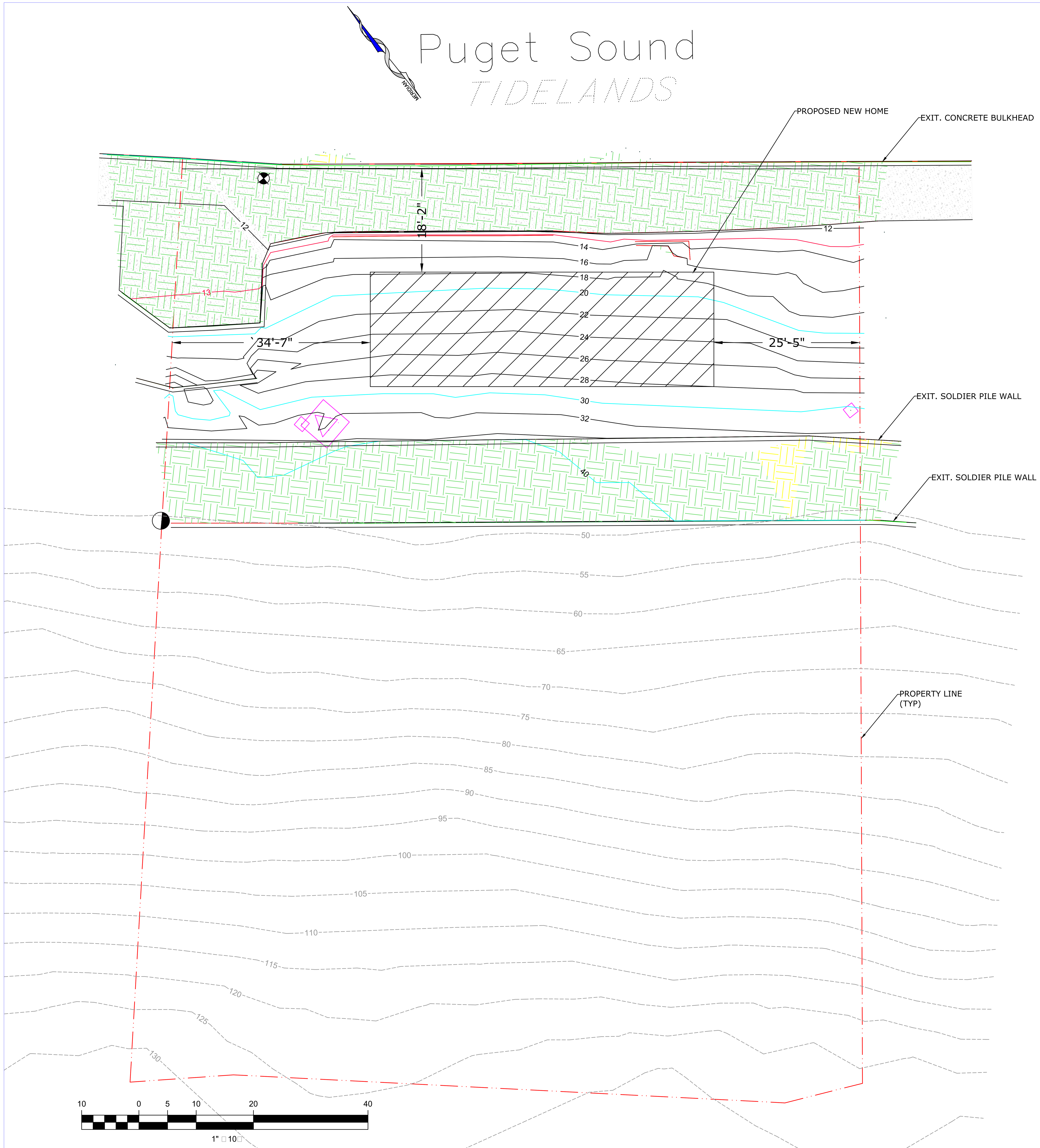
Seller shall not be liable for incidental or consequential losses, damages or expenses, directly or indirectly arising from the sale, handling or use of the goods, or from any other cause relating thereto, and seller's liability hereunder in any case is expressly limited to the replacement (in the form originally shipped) of goods not complying with this agreement or at seller's election, to the repayment of, or crediting buyer with, an amount equal to the purchase price of such goods, whether such claims are for breach of warranty or negligence.

Any claim by buyer with reference to the goods sold hereunder for any cause shall be deemed waived by buyer unless submitted to seller in writing within thirty (30) days from the date buyer discovered or should of discovered, any claimed breach.

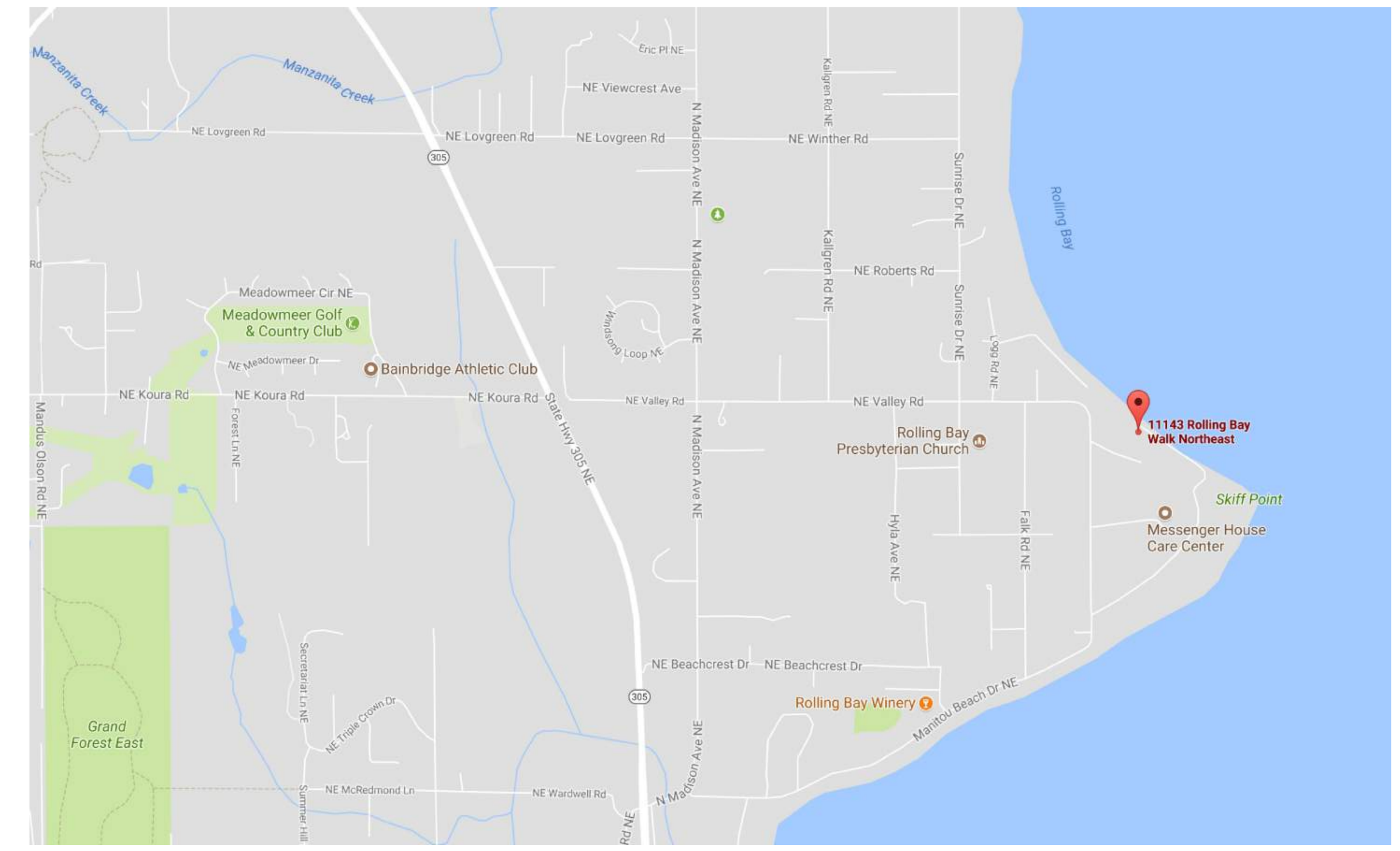
Materials should be inspected and tested by purchaser prior to their use if product quality is subject to verification after shipment. Performance guarantees are normally supplied by the applicator.

Note: Voltex DS waterproofing system is not an expansion joint material. Expansion joints shall be the responsibility of Others.

MARCH 2010
(SUPERSEDES ALL PREVIOUS VERSIONS)



SITE PLAN



VICINITY

OWNER INFORMATION:

DUFRENSE MARGARET A
3866 NE State Highway 104
Poulsbo, Washington 98370
(206)491-3917

SITE INFORMATION:

11143 Rolling Bay Walk NE
Bainbridge, Island, Washington 98110

Parcel No.: 4156-001-004-1006

GEOTECHNICAL ENGINEER:

Aspect Consulting, LLC
350 Madison Avenue N.
Bainbridge Island, Washington 98110
(206) 780-9370

ENGINEER:

Lucia Engineering, Inc.
7307 12th Avenue NE
Seattle, Washington 98115
(206)790-8039

DUFRENSE RESIDENCE
11143 Rolling Bay Walk NE
Bainbridge Island, WA

Soldier Pile & Timber Lagging
Retaining Wall

LUCIA ENGINEERING, INC.
7307 12th Avenue N.E.
Seattle, Washington 98115
PHONE: (206) 790-8039
E-MAIL: joe@luciaeng.com

JOSEPH M. LUCIA
STATE OF WASHINGTON
23314
REGISTERED PROFESSIONAL ENGINEER
11-15-17

Number	Date	By	Description
2	11-15-17		Bainbridge Island, WA

SHEET
SH-1.0

SOLDIER PILE & TIMBER LAGGING - SHORING NOTES:

CAST-IN-PLACE CONCRETE:

REFERENCE STANDARDS:
(1) ACI 301-10 "STANDARD SPECIFICATIONS FOR STRUCTURAL CONCRETE"
(2) 2015 INTERNATIONAL BUILDING CODE

CONCRETE MIXTURES: CONFORM TO:
(1) ACI 301 SECTION 4 "CONCRETE MIXTURES"

MATERIALS: CONFORM TO:
(1) ACI 301 SECTION 4.2.1 "MATERIALS" FOR REQUIREMENTS FOR CEMENTITIOUS MATERIALS, AGGREGATES, MIXING WATER AND ADMIXTURES.

MIX DESIGN REQUIREMENTS:

PILE CONCRETE:

ABOVE EXCAVATION LINE (DREDGE LINE):	LEAN MIX:	500 PSI
BELOW EXCAVATION LINE (DREDGE LINE):	STRUCTURAL MIX:	2,500 PSI

MIX DESIGN NOTES:

LEAN MIX SHALL HAVE A MINIMUM OF 1-1/2 SACKS (141 POUNDS) OF CEMENT PER CUBIC YARD OF LEAN MIX CONCRETE.

STRUCTURAL MIX SHALL HAVE A MINIMUM OF 3 SACKS (282 POUNDS) OF CEMENT PER CUBIC YARD OF STRUCTURAL MIX CONCRETE.

PORTLAND CEMENT SHALL BE TYPE I, II, OR III CONFORMING TO ASTM C150 / AASHTO M85
FLY ASH, IF USED, SHALL BE TYPE F CONFORMING TO ASTM C618

FINE AGGREGATES SHALL CONFORM TO ASTM C88 / AASHTO M6
COARSE AGGREGATES SHALL CONFORM TO AASHTO M80. CLASS B

SLUMP FOR LEAN -MIX CONCRETE SHALL NOT BE LESS THAN 5 INCHES AND NOT MORE THAN 9 INCHES.

ADMIXTURES SHALL CONFORM TO ASTM C494 / AASHTO M194

MIX DESIGNS ARE TO BE SUBMITTED TO THE SHORING DESIGN ENGINEER FOR APPROVAL PRIOR TO USE

STRUCTURAL STEEL:

REFERENCED STANDARDS:
(1) AISC "MANUAL OF STEEL CONSTRUCTION - ALLOWABLE STRESS DESIGN"
(2) AISC "CODE OF STANDARD PRACTICE FOR STEEL BUILDINGS & BRIDGES"
(3) AWS D1.1 "STRUCTURAL WELDING CODE - STEEL"

MATERIALS: CONFORM TO:
STRUCTURAL WF SHAPES - ASTM A992-GR50
HEADED STUDS SHALL CONFORM TO ASTM A108

TIMBER:

TIMBER:

MATERIALS:
(1) TIMBER LAGGING SHALL BE 4x12
HEM FIR No. 2 OR BETTER

PRESERVATIVE TREATMENT:
NONE REQUIRED

PAINT:

NONE REQUIRED

TIMBER LAGGING & EXCAVATION:
Each lagging lift shall be 4-foot-high maximum, and that no excavation for the immediate lower lift is allowed until voids behind the lagging of the ongoing lift are filled with approved materials. All voids behind lagging shall be backfilled prior to project main structure construction.

WELDING:

WELDING AND REPAIR WELDING FOR ALL STEEL FABRICATION SHALL COMPLY WITH THE AWS D1.1/D1.1M, LATEST EDITION, STRUCTURAL WELDING CODE. THE REQUIREMENTS DESCRIBED IN THE REMAINDER OF THIS SECTION SHALL PREVAIL WHENEVER THEY DIFFER FROM EITHER OF THE ABOVE WELDING CODES.

THE CONTRACTOR SHALL WELD STRUCTURAL STEEL ONLY TO THE EXTENT SHOWN IN THE PLANS.
NO WELDING, INCLUDING TACK AND TEMPORARY WELDS SHALL BE DONE IN THE SHOP OR FIELD UNLESS THE LOCATION OF THE WELDS IS SHOWN ON THE APPROVED SHOP DRAWINGS OR APPROVED BY THE ENGINEER IN WRITING.
WELDING PROCEDURES SHALL BE SUBMITTED FOR APPROVAL WITH SHOP DRAWINGS . THE PROCEDURES SHALL SPECIFY THE TYPE OF EQUIPMENT TO BE USED, ELECTRODE SELECTION, PREHEAT REQUIREMENTS, BASE MATERIALS, AND JOINT DETAILS. WHEN THE PROCEDURES ARE NOT PREQUALIFIED BY AWS OR AASHTO, EVIDENCE OF QUALIFICATION TESTS SHALL BE SUBMITTED.

WELDING SHALL NOT BEGIN UNTIL AFTER THE CONTRACTOR HAS RECEIVED THE ENGINEER'S APPROVAL OF SHOP PLANS.
THESE PLANS SHALL INCLUDE PROCEDURES FOR WELDING, ASSEMBLY, AND ANY HEAT-STRAIGHTENING OR HEAT-CURVING.

IN SHIELDED METAL-ARC WELDING, THE CONTRACTOR SHALL USE LOW-HYDROGEN ELECTRODES.
IN SUBMERGED-ARC WELDING, FLUX SHALL BE OVEN-DRIED AT 550°F FOR AT LEAST 2-HOURS, THEN STORED IN OVENS HELD AT 250°F OR MORE. IF NOT USED WITHIN 4-HOURS AFTER REMOVAL FROM A DRYING OR STORAGE OVEN, FLUX SHALL BE REDRIED BEFORE USE.
PREHEAT AND INTERPASS TEMPERATURES SHALL CONFORM TO THE APPLICABLE WELDING CODE AS SPECIFIED IN THIS SECTION. REFER TO APPROVED WELDING PROCEDURES WHEN WELDING MAIN TO STEEL MEMBERS. IF GROOVE WELDS (WEB-TO-WEB OR FLANGE-TO-FLANGE) HAVE BEEN REJECTED, THEY MAY BE REPAIRED NO MORE THAN TWICE. IF A THIRD FAILURE OCCURS, THE CONTRACTOR SHALL:

1. TRIM THE MEMBERS, IF THE ENGINEER APPROVES, AT LEAST ½-INCH ON EACH SIDE OF THE WELD;
2. REPLACE THE MEMBERS AT NO EXPENSE TO THE CONTRACTING AGENCY.

BY USING EXTENSION BARS AND RUNOFF PLATES, THE CONTRACTOR SHALL TERMINATE GROOVE WELDS IN A WAY THAT ENSURES THE SOUNDNESS OF EACH WELD TO ITS ENDS. THE BARS AND PLATES SHALL BE REMOVED AFTER THE WELD IS FINISHED AND COOLED. THE WELD ENDS SHALL THEN BE GROUND SMOOTH AND FLUSH WITH THE EDGES OF ABUTTING PARTS.

THE CONTRACTOR SHALL NOT:

1. WELD WITH ELECTROGAS OR ELECTROSLAG METHODS,
2. WELD NOR FLAME CUT WHEN THE AMBIENT TEMPERATURE IS BELOW 20°F,
3. USE COPED HOLES IN THE WEB FOR WELDING BUTT SPLICES IN THE FLANGES UNLESS THE PLANS SHOW THEM.

UTILITIES & INTERFERENCES:

ALL EXISTING UTILITIES AND OTHER OBJECTS WHICH MAY INTERFERE WITH THE INSTALLATION OF THE SHORING SYSTEM ARE TO BE LOCATED PRIOR TO BEGINNING CONSTRUCTION.

POSSIBLE INTERFERENCES BETWEEN THE SHORING AND ANY UTILITY OR OTHER OBJECT(S) IS TO BE PROVIDED TO THE SHORING DESIGNER PRIOR TO THE START OF WORK.

GEOTECHNICAL REFERENCE:

GEOTECHNICAL REPORT BY: ASPECT CONSULTING
REPORT DATED: JULY 20, 2017

UTILITIES & INTERFERENCES:

ALL EXISTING UTILITIES AND OTHER OBJECTS WHICH MAY INTERFERE WITH THE INSTALLATION OF THE SHORING SYSTEM ARE TO BE LOCATED PRIOR TO BEGINNING CONSTRUCTION.

POSSIBLE INTERFERENCES BETWEEN THE SHORING AND ANY UTILITY OR OTHER OBJECT(S) IS TO BE PROVIDED TO THE SHORING DESIGNER PRIOR TO THE START OF WORK.

SHORING INSTALLATION REVIEW:

A REPRESENTATIVE OF THE SHORING DESIGNER IS TO BE PRESENT AT ALL TIMES DURING THE INSTALLATION OF THE SHORING SYSTEM.

SEE THE GEOTECHNICAL REPORT FOR REQUIRED GEOTECHNICAL INSPECTIONS & REVIEW

MANUFACTURE OF STEEL PILES

STEEL PILES SHALL BE MADE OF ROLLED STEEL H OR W-PILE SECTIONS, STEEL PIPE PILES, OR OF OTHER STRUCTURAL STEEL SECTIONS DESCRIBED IN THE CONTRACT.
A FULL PENETRATION GROOVE WELD WITH A AXIMUM 1⁄16-INCH OFFSET BETWEEN WELDED EDGES IS REQUIRED FOR SPLICED PILE.

SPLICING STEEL CASINGS AND STEEL PILES

THE ENGINEER WILL NORMALLY PERMIT STEEL PILES TO BE SPLICED. BUT IN EACH CASE, THE CONTRACTOR
MUST OBTAIN APPROVAL ON THE NEED AND THE METHOD FOR SPLICING. WELDED SPLICES SHALL BE SPACED
AT A MINIMUM DISTANCE OF 10-FEET. ONLY FULL PENETRATION WELDED SPLICES WILL BE PERMITTED.
SPLICE WELDS SHALL COMPLY WITH SECTION 6-03.3(25) AND AWS D1.1 STRUCTURAL
WELDING CODE. SPLICING OF STEEL PILES SHALL BE PERFORMED IN ACCORDANCE WITH AN APPROVED
WELD PROCEDURE. THE CONTRACTOR SHALL SUBMIT A WELD PROCEDURE TO THE ENGINEER FOR
APPROVAL PRIOR TO WELDING. ALL JOINTS SHALL BE WELDED BY A WASHINGTON STATE CERTIFIED WELDER.

GEOTECHNICAL REFERENCE:

GeoEngineers report dated 11/23/2016

TIEBACK ANCHOR TESTING:

PERFORMANCE TESTS AND PROOF TESTING SHALL BE PERFORMED AT THE LOCATIONS SPECIFIED BY THE SHORING DESIGNER. IF THE SHORING DESIGNER HAS NOT SPECIFIED THE TEST LOCATIONS, THE CONTRACTOR SHALL SELECT LOCATIONS AND SUBMIT TO THE SHORING DESIGNER FOR APPROVAL.

PERFORMANCES TESTS SHALL BE COMPLETED AT LEAST ONE ANCHOR IN EACH WALL. ALL ANCHORS SHALL BE PROOF TESTED.

NO TESTING SHALL TAKE PLACE UNTIL THE ANCHOR GROUT HAS ATTAINED 100% OF ITS SPECIFIED 28-DAY COMPRESSIVE STRENGTH.

PERFORMANCE TESTING:

PERFORMANCE TEST SHALL BE MADE BY INCREMENTALLY LOADING THE ANCHOR IN ACCORDANCE WITH THE LOAD SCHEDULE BELOW. AT EACH INCREMENT OF MOVEMENT THE TENDON SHALL BE RECORDED TO THE NEAREST 0.001 INCHES WITH RESPECT TO AN INDEPENDENT FIXED REFERENCE POINT.

THE JACK LOAD SHALL BE MONITORED WITH A PRESSURE GAUGE CALIBRATED WITH THE JACK AND ACCURATE ENOUGH TO READ 100 PSI CHANGES IN PRESSURE. THE PUMP SHALL BE CAPABLE OF APPLYING EACH LOAD INCREMENT IN LESS THAN 60 SECONDS.

THE INCREMENT OF LAD SHALL BE ASS FOLLOWS:

LOAD	HOLD TIME
AL	
0.25 p	
0.50 P	
0.75 P	
1.00 P	
1.25 P	
1.50 P	
1.75 P	
2.00 P	10 MINUTES

P = DESIGN LOAD, AL = ALIGNMENT LOAD

THE LOAD USED TO ALIGN THE TESTING EQUIPMENT SHALL NOT EXCEED 0.05 TL

THE LOAD SHALL BE HELD AT EACH INCREMENT JUST LONG ENOUGH TO OBTAIN MOVEMENT READING. EXCEPT FOR THE READING OF THE RESIDUAL MOVEMENT AT AL, NO MOVEMENT READING NEEDS TO BE TAKEN DURING UNLOADING OF THE ANCHOR. THE TEST LOAD SHALL BE HELD FOR 10 MINUTES. TOTAL MOVEMENT WITH RESPECT TO A FIXED REFERENCE POINT SHALL BE RECORDED AT 1 MINUTE, 2, 3, 4, 5, 6, & 10 MINUTES. IF THE TOTAL MOVEMENT BETWEEN 1 AND 10 MINUTES EXCEEDS 0.04 INCHES THE TEST LOAD SHALL BE HELD FOR AN ADDITIONAL 50 MINUTES. TOTAL MOVEMENT SHALL BE RECORDED AT 15 MINUTES, 20, 25, 30, 45 AND 60 MINUTES. THE TEST LOAD TIME SHALL START WHEN THE THE TEST LOAD IS REACHED.

PROOF TESTING OF PRODUCTION ANCHORS:

PROOF TESTING SHALL BE PREFORMED ON ALL OF THE PRODUCTION ANCHORS.

PROOF TEST SHALL BE INCREMENTALLY LOADED IN ACCORDANCE WITH THE FOLLOWING SCHEDULE. AT EACH INCREMENT THE MOVEMENT OF THE TENDON SHALL BE RECORDED TOT HE NEAREST 0.001 INCHES WITH RESPECT TO AN INDEPENDENT FIXED REFERENCE POINT. THE JACK LOAD SHALL BE MONITORED WITH A GAUGE OR LOAD CELL.

THE INCREMENT OF LAD SHALL BE ASS FOLLOWS:

LOAD	HOLD TIME
AL	
0.25P	
0.50P	
0.75P	
1.00P	
1.20P	
1.33P	10 MINUTES

ADJUST TO LOCK-OFF LOAD: (LOCK-OFF LOAD IS 80% P U.O.N.)

P = DESIGN LOAD, AL = ALIGNMENT LOAD

THE LOAD USED TO ALIGN THE TESTING EQUIPMENT SHOULD NOT EXCEED 0.050 TL. DIAL GAUGES ARE TO BE ZEROED AFTER THE ALIGNMENT LOAD IS APPLIED.

THE LOAD SHALL BE HELD AT EACH INCREMENT JUST LONG ENOUGH TO OBTAIN MOVEMENT READING BUT NOT MORE THAN 1 MINUTE. THE TEST LOAD SHALL BE HELD FOR 10 MINUTES. TOTAL MOVEMENT WITH RESPECT TO A FIXED REFERENCE POINT SHALL BE RECORDED AT 1 MINUTE, 2, 3, 4, 5, 6, & 10 MINUTES. IF THE TOTAL MOVEMENT BETWEEN 1 AND 10 MINUTES EXCEEDS 0.04 INCHES THE TEST LOAD SHALL BE HELD FOR AN ADDITIONAL 50 MINUTES. TOTAL MOVEMENT SHALL BE RECORDED AT 15 MINUTES, 20, 25, 30, 45 AND 60 MINUTES. THE TEST LOAD TIME SHALL START WHEN THE PUMP BEGINS TO LOAD THE ANCHOR FROM 1.20P LOAD TO THE TEST LOAD.

SHORING MONITORING PLAN

A MONITORING PROGRAM IS TO BE IMPLEMENTED TO VERIFY THE PERFORMANCE OF THE SHORING SYSTEM AND POSSIBLE EXCAVATION EFFECTS ON NEIGHBORING PROPERTIES.

THE FIRST STEP IN THIS PROGRAM SHOULD CONSIST OF SURVEYING THE SHORING WALL LINE & ELEVATIONS AND DOCUMENTING THE CONDITION/LOCATION OF THE SHORING AS INSTALLED.
THIS DOCUMENTATION SHOULD INCLUDE A PHOTOGRAPHIC RECORD, WITH MONITORING POINTS ESTABLISHED AS FOLLOWS:
ELEVATION AND LOCATION POINTS AT APPROXIMATELY 25 FOOT INTERVALS ON THE TOP OF THE SHORING WALL - THE 3 FOOT OFFSETS CAN BE USED

MONITORING OF THE SHORING SYSTEM AND SURROUNDING AREAS SHOULD OCCUR TWO TIMES PER WEEK AS THE EXCAVATION PROCEEDS AND THEN ONCE EVERY TWO WEEKS ONCE THE EXCAVATION IS COMPLETED.

A REGISTERED LAND SURVEYOR SHOULD BE RETAINED TO ESTABLISH THE BASELINE DATA AND AT LEAST ONE OF THE WEEKLY READINGS AND OBTAIN THE BI-WEEKLY READINGS. THE SECOND WEEKLY READING MAY BE OBTAINED BY THE PROJECT CONTRACTOR.

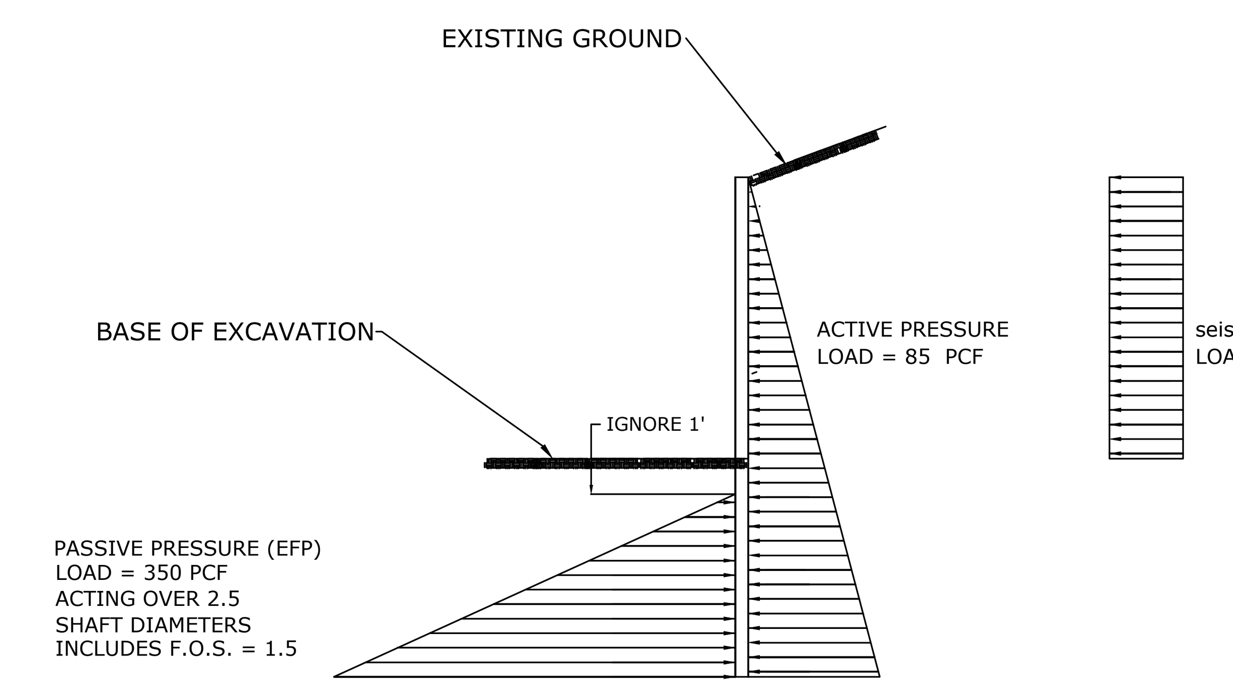
MONITORING SHOULD CONTINUE UNTIL THE PERMANENT SITE WORK IS COMPLETED.

THE DATA SHOULD INCLUDE SURVEYING THE VERTICAL AND HORIZONTAL ALIGNMENT OF EVERY MONITORING POINT. THE POINT ON THE SHORING SHOULD BE ESTABLISHED WITHIN 24 HOURS OF THE PILE BEING INSTALLED AND PRIOR TO EXCAVATION NEXT TO THE PILE. THE POINT SHOULD BE LOCATED ON THE FACE OF THE PILE NEAR THE GROUND SURFACE ELEVATION BEHIND THE PILE.

THE PROJECT'S SHORING DESIGNER AND GEOTECHNICAL ENGINEER SHOULD REVIEW THE MONITORING DATA WEEKLY. SURVEY FREQUENCY CAN BE DECREASED AFTER THE SHORING SYSTEM HAS BEEN INSTALLED AND THE EXCAVATION IS COMPLETE IF THE DATA INDICATES LITTLE OR NO ADDITIONAL MOVEMENT OUTSIDE WHAT IS EXPECTED BY THE SHORING DESIGNER.

SURVEYING SHOULD CONTINUE UNTIL THE PERMANENT STRUCTURE IS COMPLETE UP TO FINAL AND STREET GRADES. THE SURVEY FREQUENCY WILL BE DETERMINED BY THE SHORING DESIGN ENGINEER.

IMMEDIATELY NOTIFY THE GEOTECHNICAL AND SHORING DESIGNER ENGINEERS, IF 0.5 INCHES OF MOVEMENT OCCURS BETWEEN TWO CONSECUTIVE READINGS AND WHEN TOTAL MOVEMENTS REACH 1.0 INCHES. THE ENGINEERS AND SHORING DESIGNER SHALL DETERMINE THE CAUSE OF ANY ADVERSE DISPLACEMENT AND DEVELOP REMEDIAL MEASURES, IF REQUIRED.

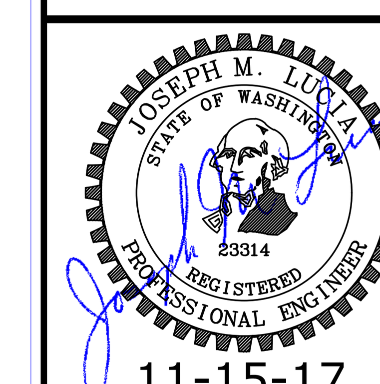


DESIGN LOADING DIAGRAM - SOLDIER PILE SHORING WALL

DUFRENSE RESIDENCE
11143 Rolling Bay Walk NE
Bainbridge Island, WA

Soldier Pile & Timber Lagging
Retaining Wall

LUCIA ENGINEERING, INC.
7307 12th Avenue N.E.
Seattle, Washington 98115
PHONE: (206) 790-8039
E-MAIL: joe@luciaeng.com

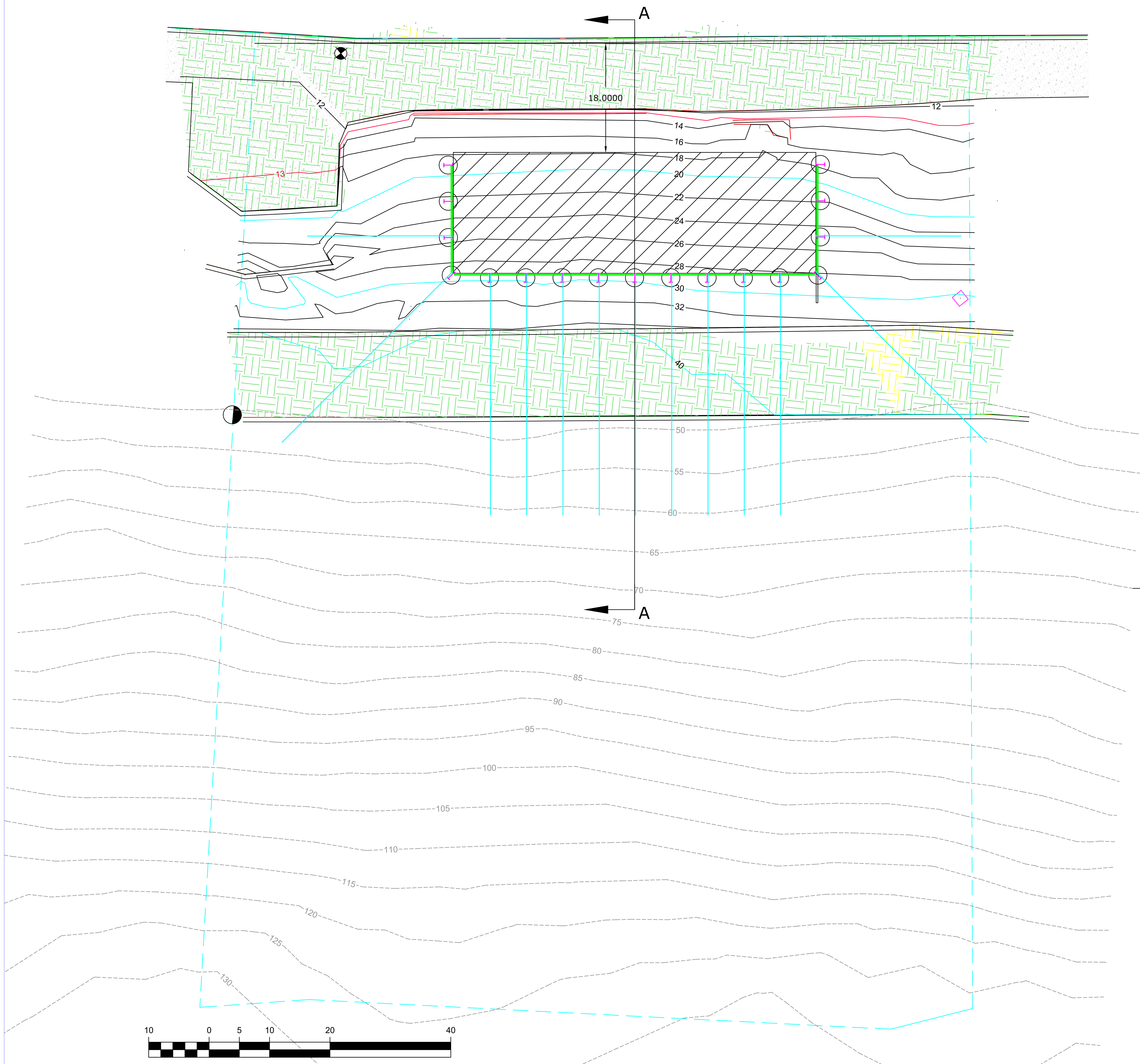


11-15-17

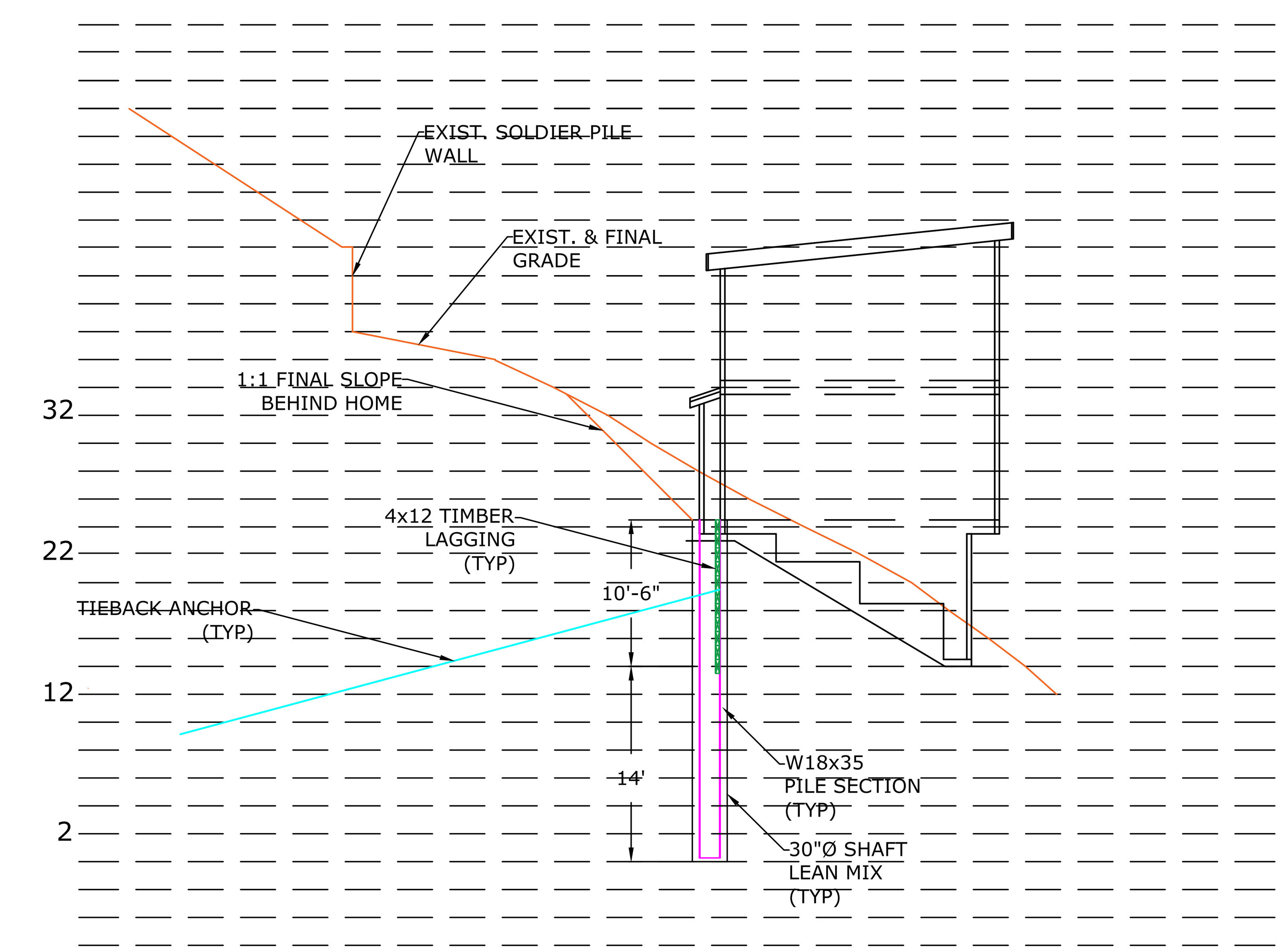
Number	Date	By	Description
2	11-15-17		Bainbridge Island, WA

SHEET
SH-2.0

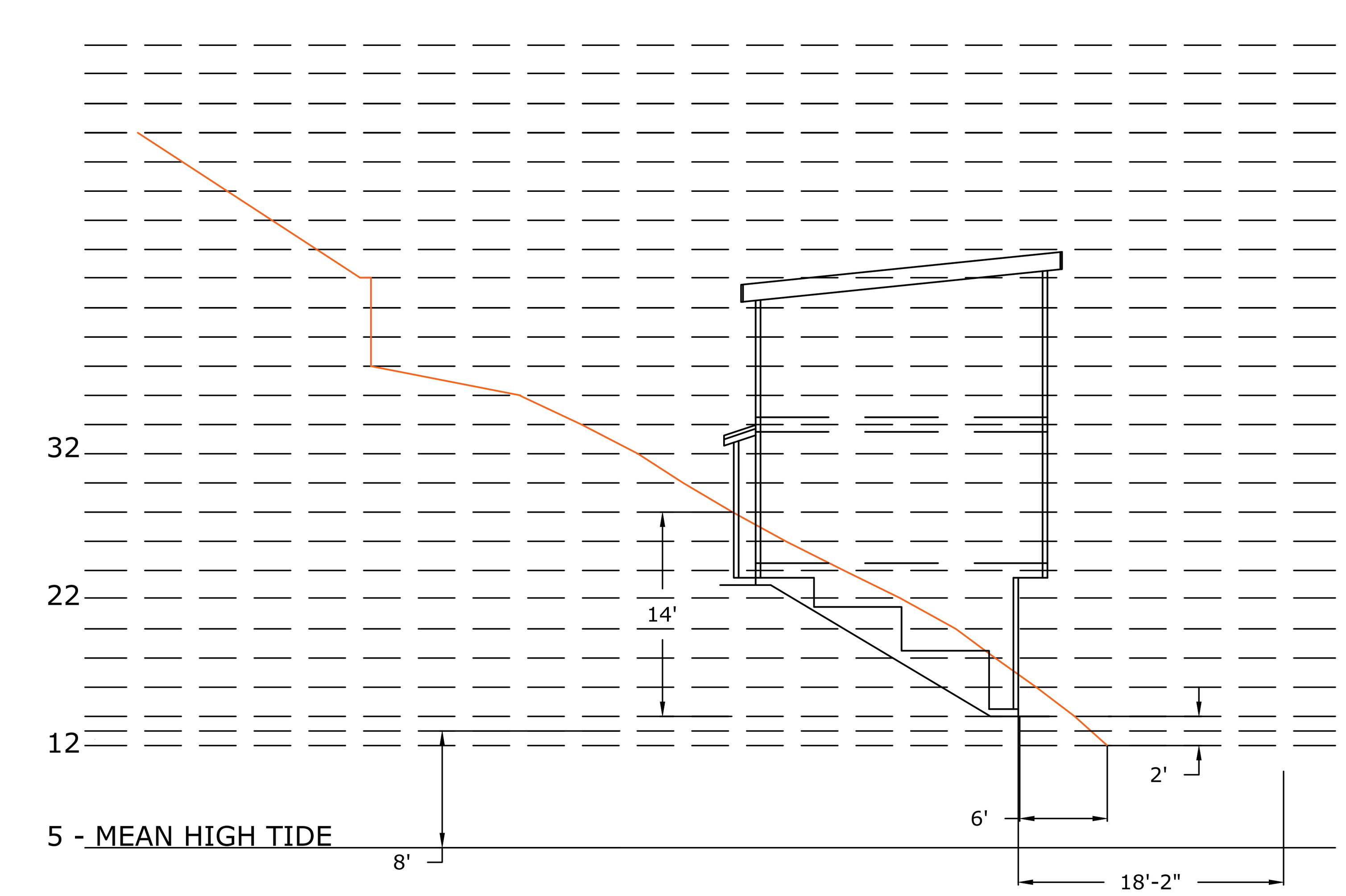
Puget Sound TIDELANDS



SITE PLAN



TYPICAL SECTION - SECTION A-A



TYPICAL SECTION - SECTION A-A

DUFRENSE RESIDENCE
11143 Rolling Bay Walk NE
Bainbridge Island, WA

Soldier Pile & Timber Lagging Retaining Wall

LUCIA ENGINEERING, INC.
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Seattle, Washington 98115
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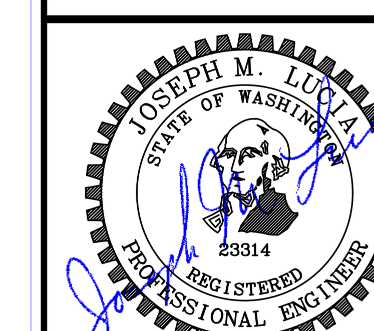
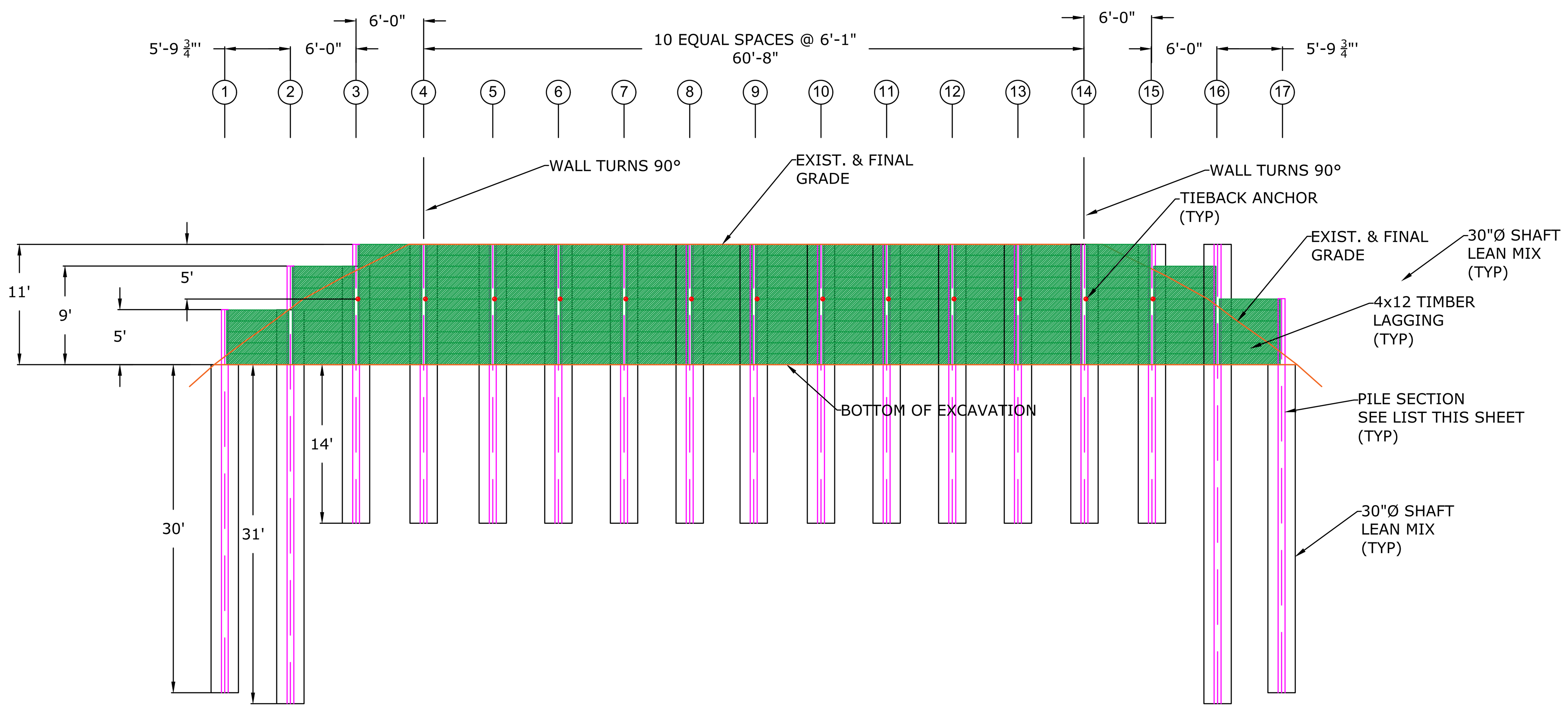


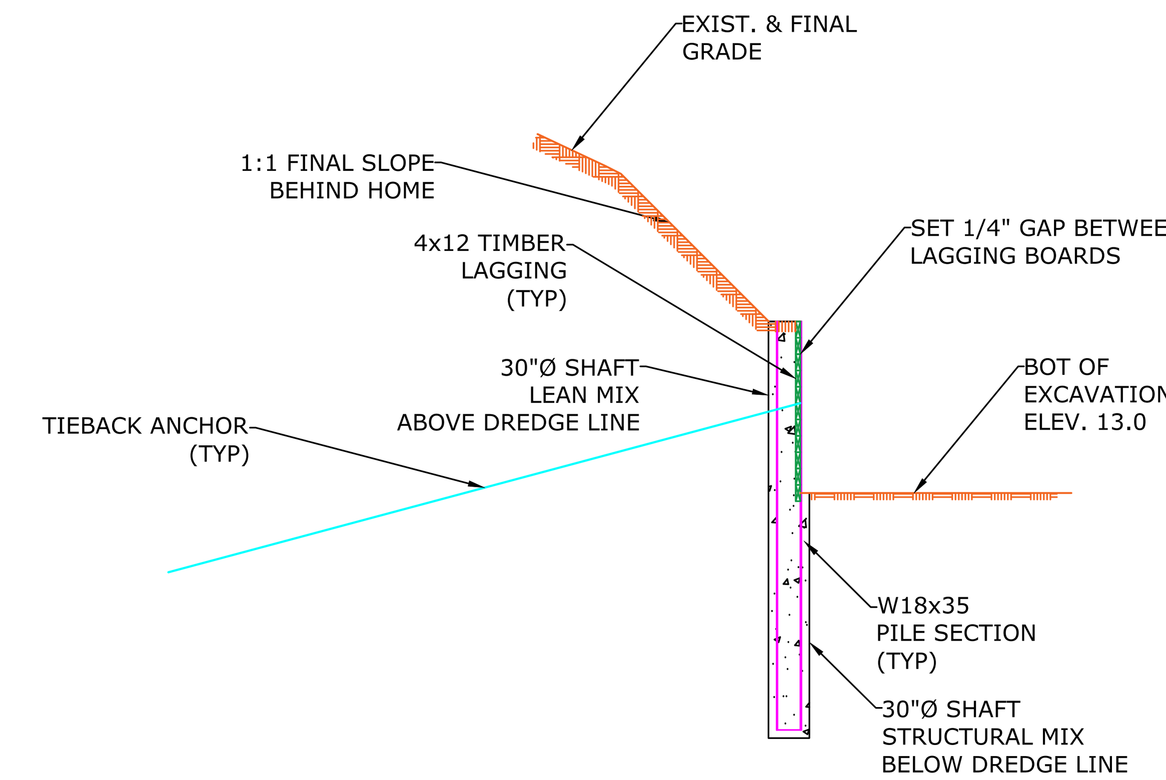
11-15-17

Number	Date	By	Description
2	11-15-17		Bainbridge Island, WA

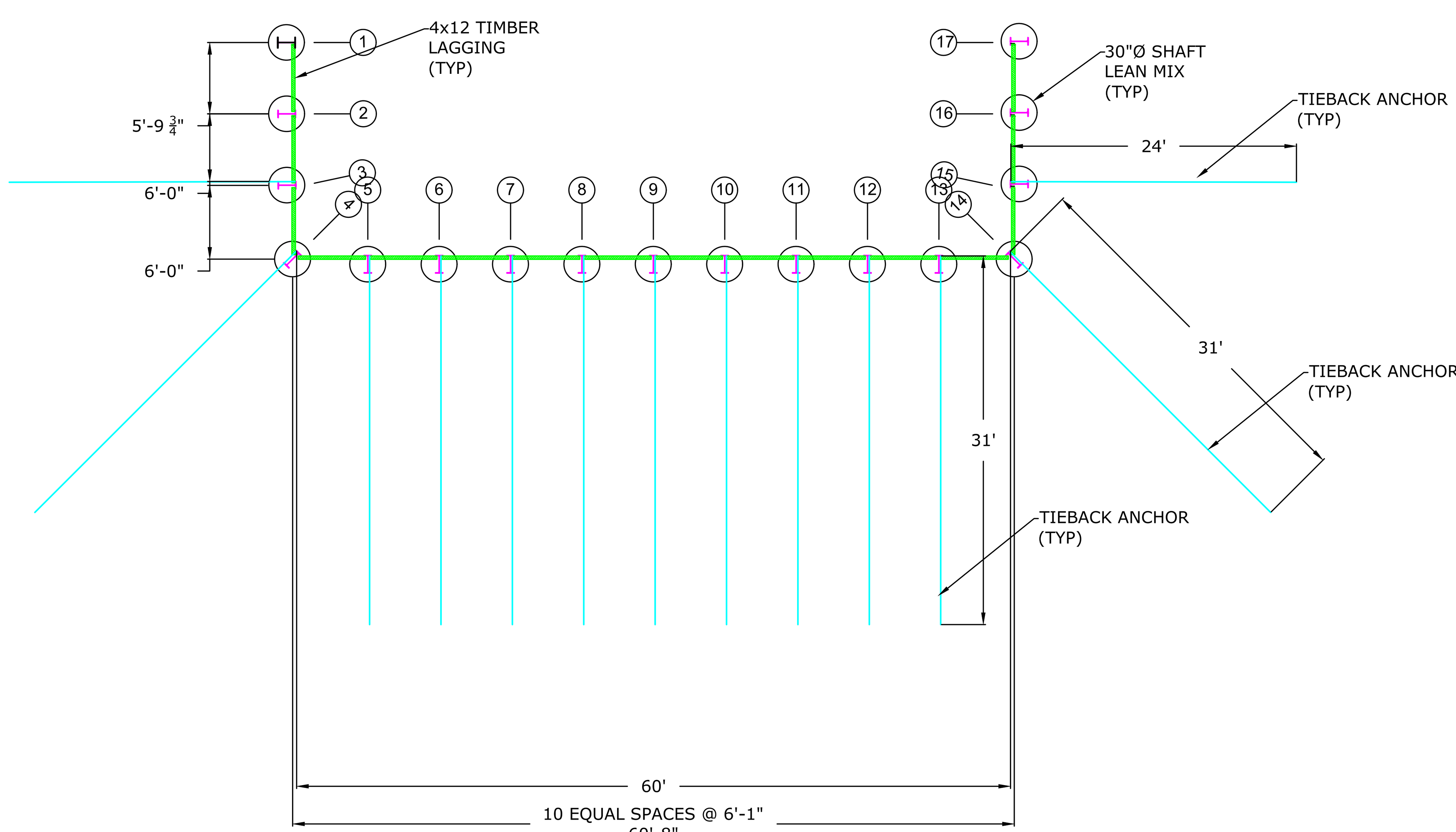
SHEET
SH-3.0



SOLDIER PILE WALL ELEVATION



TYPICAL SECTION - THROUGH PILE



PLAN VIEW - PILE LAYOUT

PILES 1 & 17
W18 x 76 x 35'

PILES 2 & 16
W18 x 130 x 40'

PILES 3 & 15
W18 x 35 x 25'
TIEBACK ANCHOR:
NO-LOAD ZONE = 9'
GROUTED ZONE = 15'
LOADING = 34 KIPS

PILES 4 through 14
W18 x 35 x 25'
TIEBACK ANCHOR:
NO-LOAD ZONE = 9'
GROUTED ZONE = 22'
LOADING = 50 KIPS

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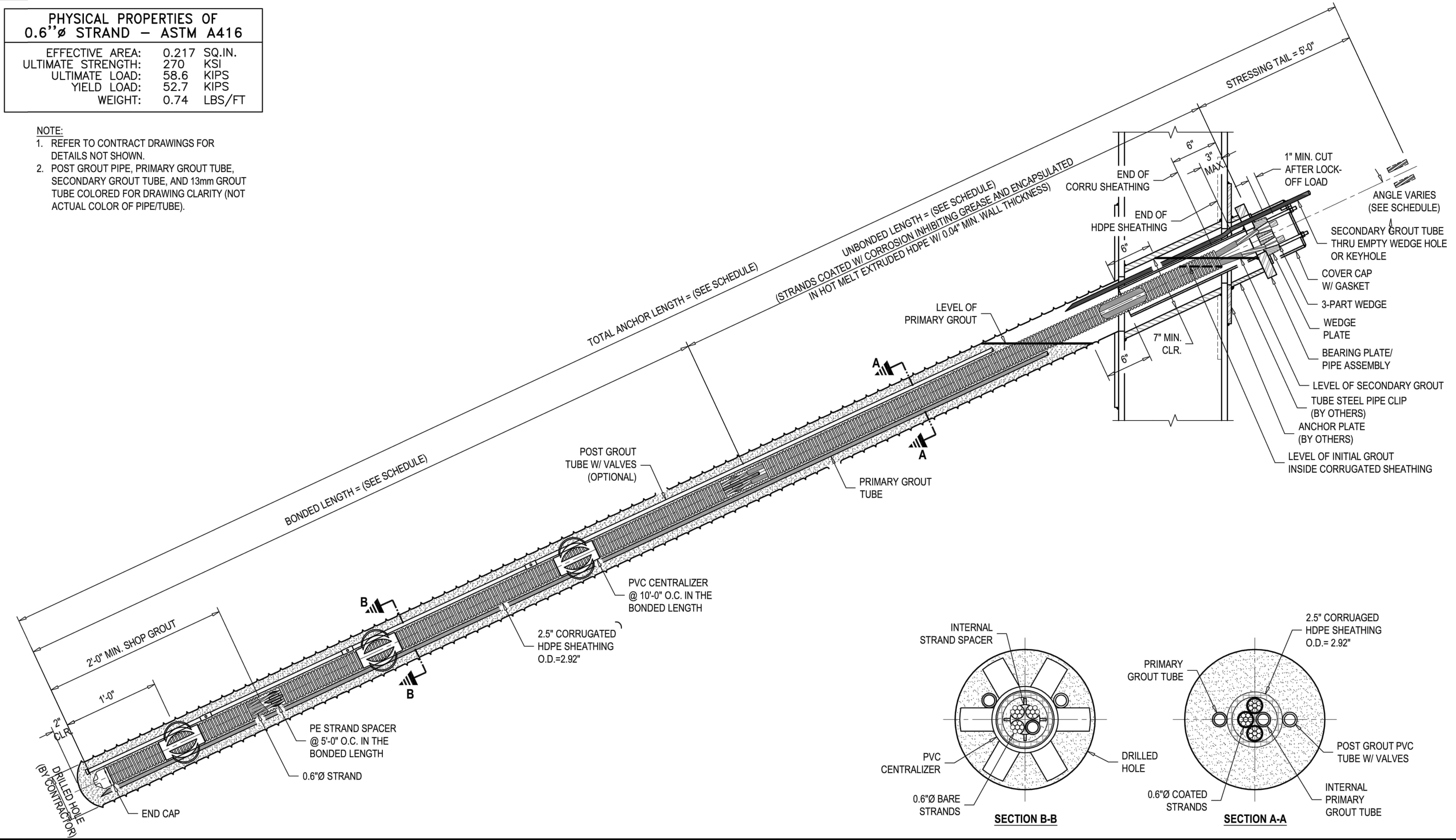
JOSEPH M. LUCIA
STATE OF WASHINGTON
23314
REGISTERED PROFESSIONAL ENGINEER
11-15-17

Number	Date	By	Description
2	11-15-17		Bainbridge Island, WA

SHEET
SH-4.0

PHYSICAL PROPERTIES OF 0.6"ø STRAND – ASTM A416		
EFFECTIVE AREA:	0.217	SQ.IN.
ULTIMATE STRENGTH:	270	KSI
ULTIMATE LOAD:	58.6	KIPS
YIELD LOAD:	52.7	KIPS
WEIGHT:	0.74	LBS/FT

NOTE:
1. REFER TO CONTRACT DRAWINGS FOR DETAILS NOT SHOWN.
2. POST GROUT PIPE, PRIMARY GROUT TUBE, SECONDARY GROUT TUBE, AND 13mm GROUT TUBE COLORED FOR DRAWING CLARITY (NOT ACTUAL COLOR OF PIPE/TUBE).



1 **TYPICAL LONGITUDINAL SECTION**

WARNING! CAST IRON COMPONENTS (CAST WEDGE PLATES, CAST HEX NUTS, CAST COUPLERS, ETC.) SHOULD NOT BE USED AS PART OF A HYDRAULIC JACK TENSIONING SYSTEM BECAUSE THEY CAN REACT EXPLOSIVELY. USE ONLY COMPONENTS THAT HAVE BEEN DESIGNED SPECIFICALLY FOR USE WITH THE TENSIONING SYSTEM. FAILURE TO HEED THIS WARNING CAN RESULT IN SEVERE INJURY OR DEATH.

DSI USA INC. DOES NOT ASSUME ANY LIABLITY FOR THE DESIGN OF THIS STRUCTURE. THIS SHOP DRAWING IS INTENDED TO PROVIDE ANCHOR DETAILS ONLY.

REV.	DATE	ISSUE DESCRIPTION	NAME	CHKD.
⚠	10.05.17	REVISION AS SHOWN	L.D.B.	A.C.

DWG TITLE: **PERMANENT STRAND TIEBACK DETAILS**

DRAWN L.BRION 08.16.17 SCALE: N.T.S.
CHKD. A.CHEN

PROJECT:
LOCATION:
CONTRACTOR: DYWIDAG Systems International, USA, Inc.

DSI

JOB NO. J118213
DWG NO.

DUFRENSE RESIDENCE
11143 Rolling Bay Walk NE
Bainbridge Island, WA

Soldier Pile & Timber Lagging Retaining Wall

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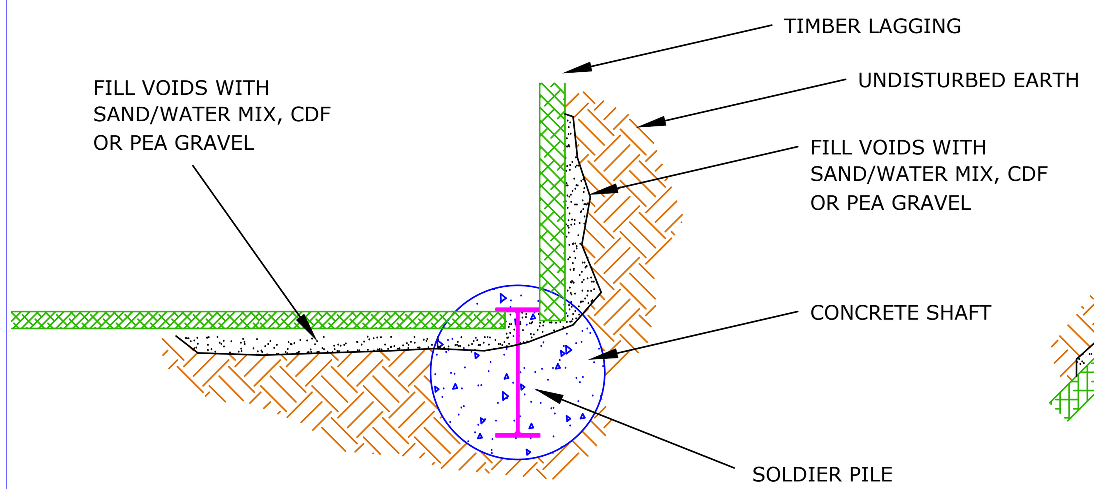
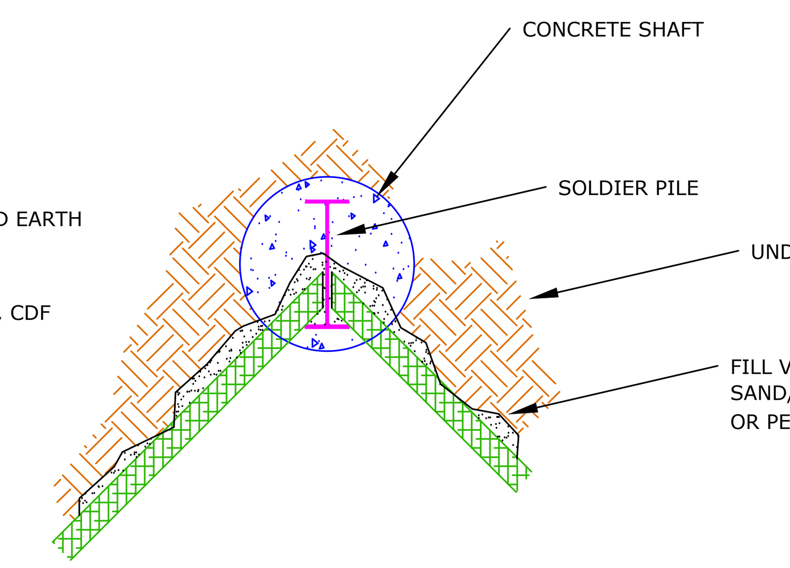
JOSEPH M. LUCIA
STATE OF WASHINGTON
23314
REGISTERED PROFESSIONAL ENGINEER
11-15-17

Number	Date	By	Description
2	11-15-17		Bainbridge Island, WA

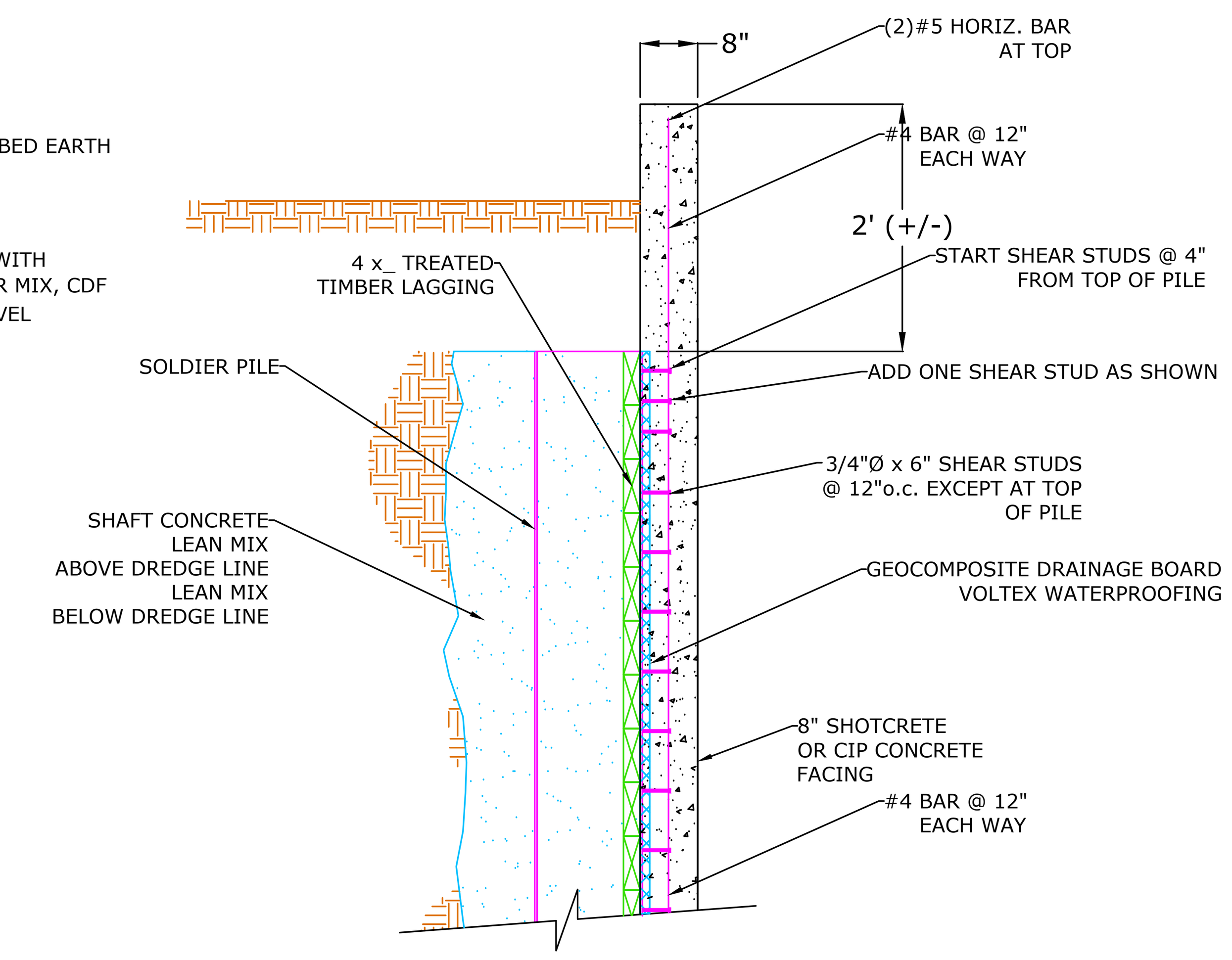
SHEET
SH-5.0



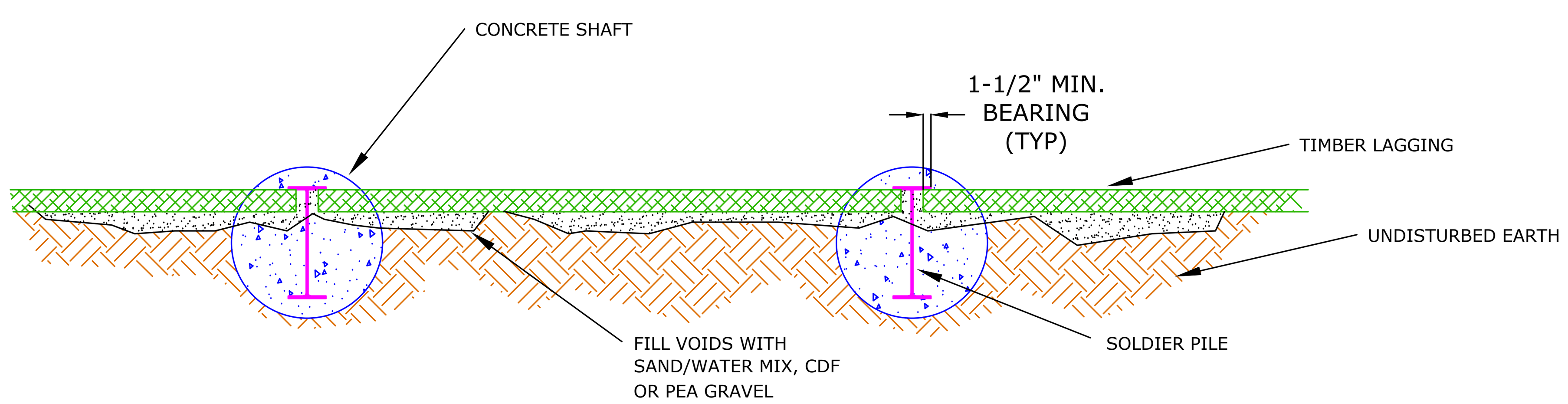
INSIDE 90° CORNER TYPE 1



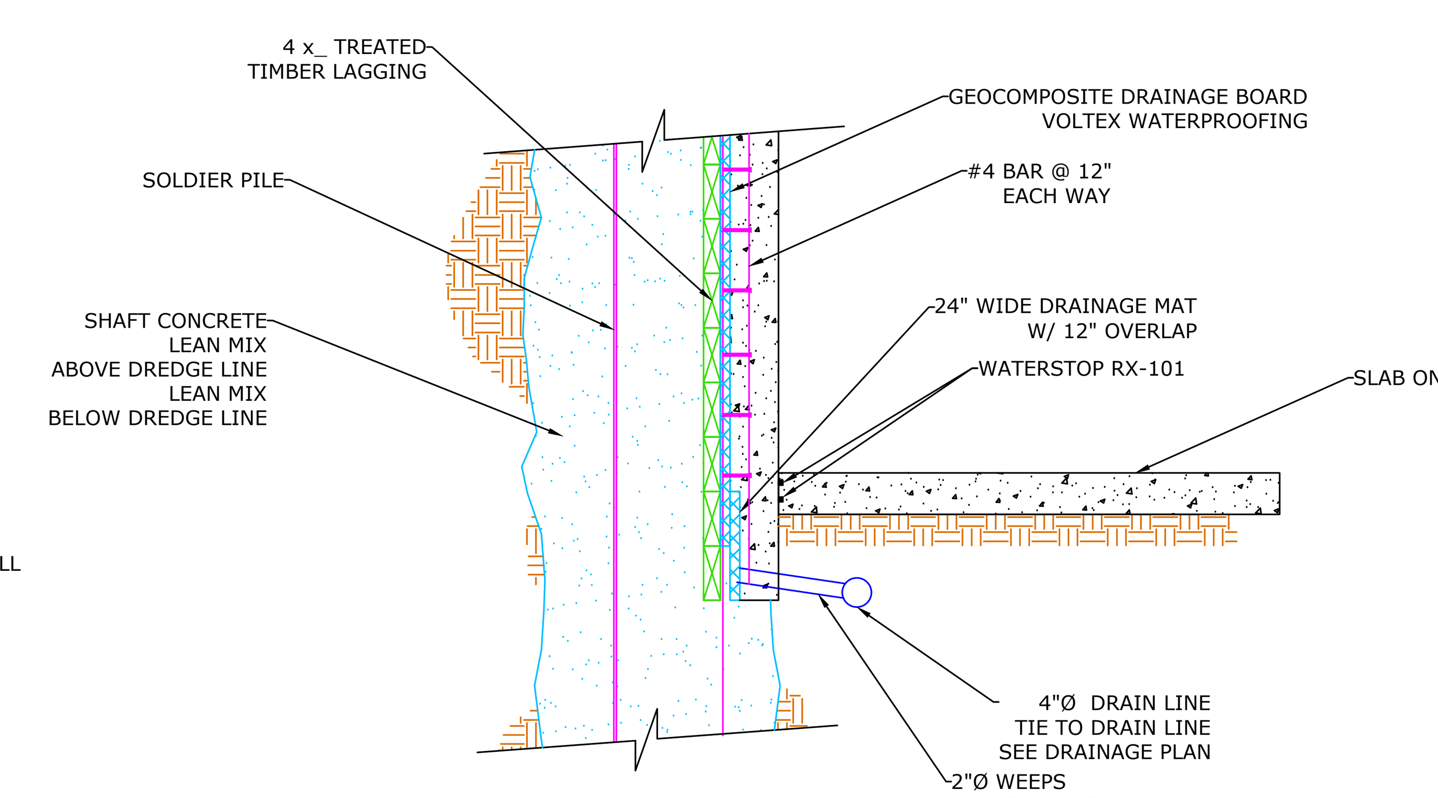
INSIDE 90° CORNER TYPE 2



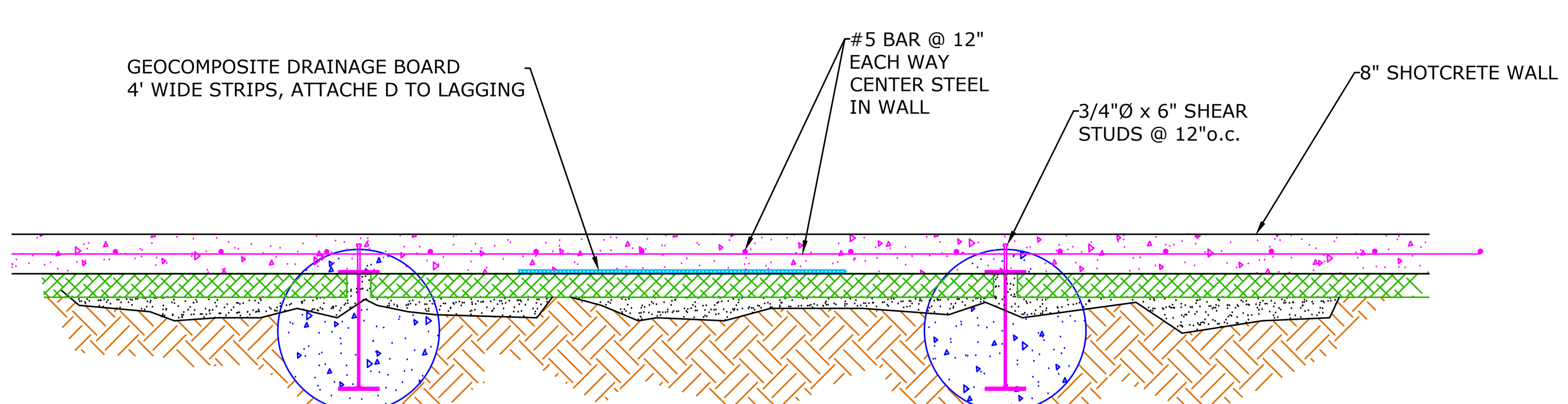
WALL DETAIL - TOP OF WALL



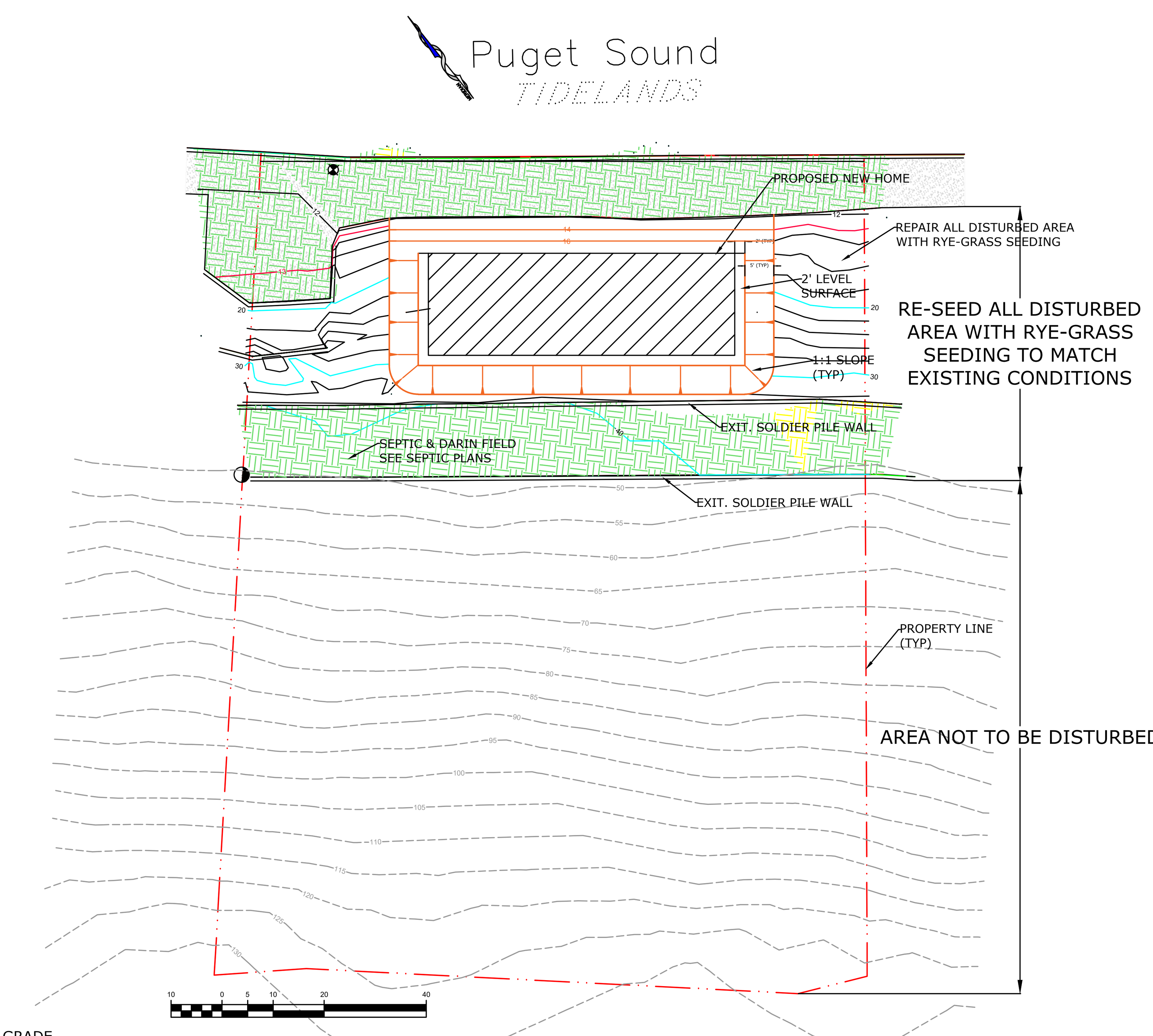
TYPICAL WALL SECTION - TEMPORARY CONDITION



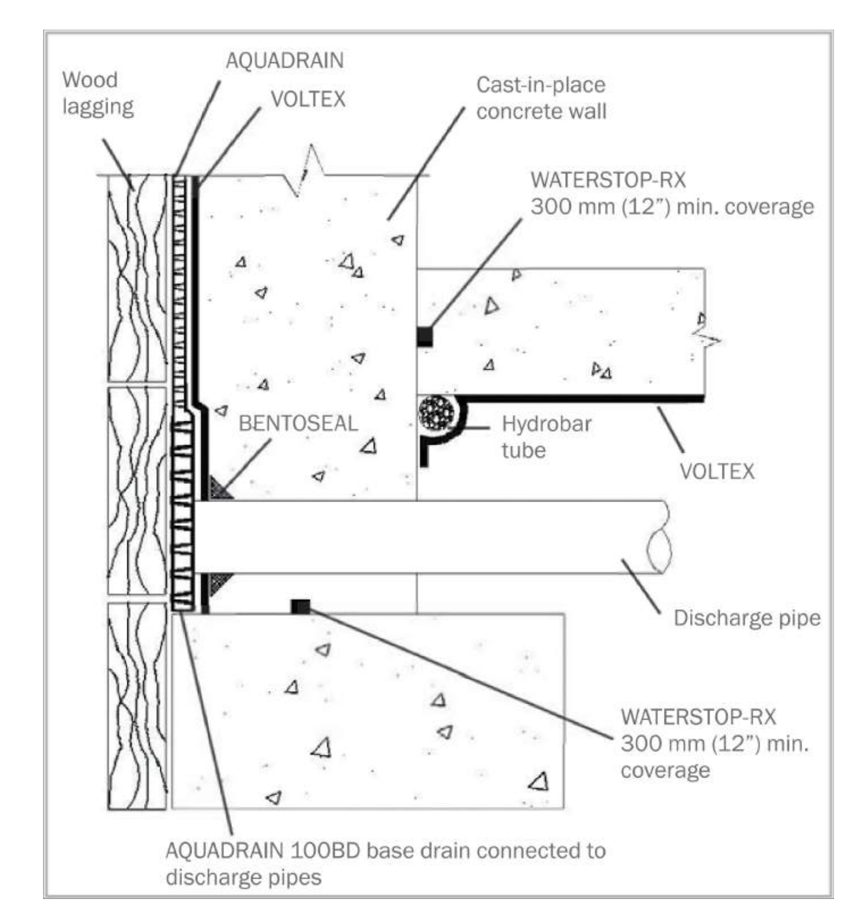
WALL DETAIL - BOTTOM OF WALL



TYPICAL WALL SECTION - FINAL CONDITION



SITE PLAN - FINAL GRADING & FINAL LANDSCAPING



WALL DRAINAGE DETAIL - BOTTOM OF WALL

DUFRENSE RESIDENCE
11143 Rolling Bay Walk NE
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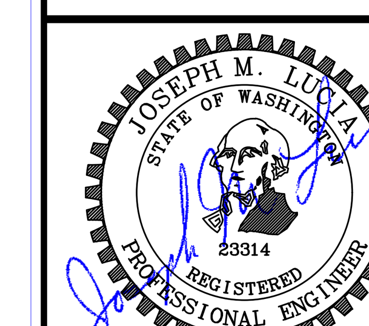
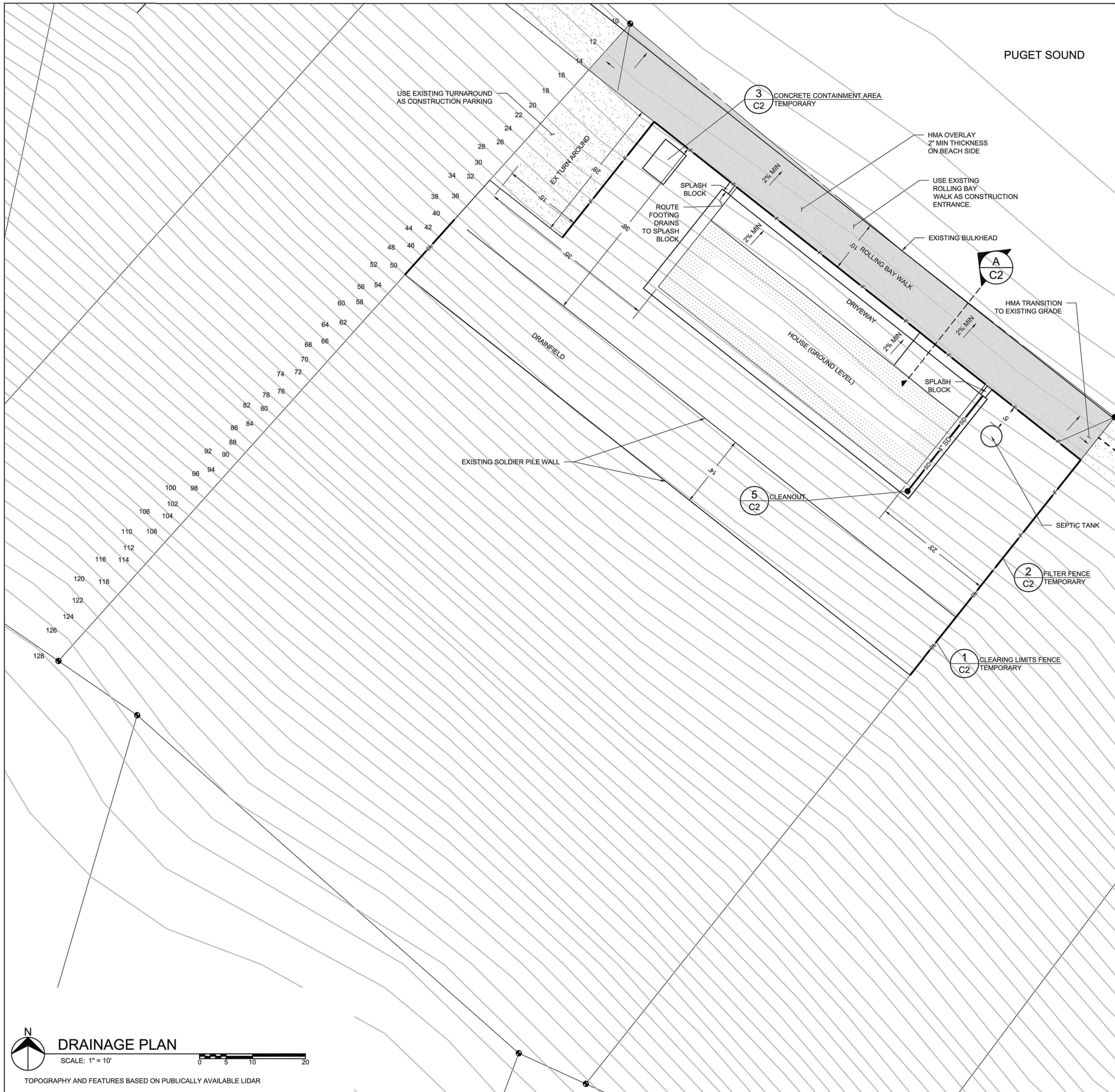


11-15-17

Number	Date	By	Description
2	11-15-17		Bainbridge Island, WA

SHEET
SH-6.0



DRAINAGE PLAN

SCALE: 1" = 10'

TOPOGRAPHY AND FEATURES BASED ON PUBLICALLY AVAILABLE LIDAR

GENERAL NOTES

1. PROPERTY LINES AND EASEMENTS SHOWN ARE BASED ON INFORMATION PROVIDED AND OTHER EASEMENTS NOT SHOWN MAY EXIST. TOPOGRAPHY AND LOCATION OF FEATURES IS APPROXIMATE AND NOT BASED ON DETAILED SURVEY.

2. UNLESS OTHERWISE SPECIFIED, STORM DRAIN PIPE SHALL BE 4" DIAMETER, EITHER RIGID PVC (ASTM D3034) WITH THREE FEET OF COVER, OR SMOOTH INTER CORRUGATED POLYETHYLENE (ADS N12 OR EQUIVALENT) WITH ONE FOOT OF COVER. OTHER PIPE MATERIALS SPECIFIED MAY NOT BE SUBSTITUTED. PIPE SHA BE LAID AT A MINIMUM SLOPE OF 2% (1/4" PER FOOT UNLESS OTHERWISE NOTED). CARE SHALL BE TAKEN TO MAINTAIN MINIMUM SLOPE AND AVOID LOW POINTS ALONG THE PIPE.

3. ROOF DOWNSPOUTS SHALL BE CONNECTED TO ROOF DRAIN LINES SHOWN ACCORDING TO ARCHITECTURAL DETAILS.

4. CURTAIN AND FOUNDATION DRAINS MAY BE CONNECTED TO THE STORM DRAIN SYSTEM AT A POINT AT LEAST 1 FOOT BELOW THE LOWEST FOOTING.

5. GRADE DRIVEWAY AND PAVED AREAS TO DRAIN AWAY FROM BUILDINGS AND TOWARD CATCH BASINS AS SHOWN ON THE PLANS.

6. UNLESS OTHERWISE SPECIFIED, MATERIALS AND WORKMANSHIP SHALL BE IN ACCORDANCE WITH THE REQUIREMENTS OF THE CURRENTLY ADOPTED EDITION OF "STANDARD SPECIFICATIONS FOR ROAD, BRIDGE AND MUNICIPAL CONSTRUCTION", APWA/WSDOT EXCEPT AS MODIFIED BY THE CITY OF BAINBRIDGE ISLAND CURRENT EDITION OF "DESIGN AND CONSTRUCTION STANDARDS AND SPECIFICATIONS".

7. CONTRACTOR SHALL OBTAIN AN UNDERGROUND UTILITIES LOCATE PRIOR TO BEGINNING CONSTRUCTION (UNDERGROUND UTILITIES LOCATION SERVICE, 811).THE LOCATIONS OF EXISTING UNDERGROUND UTILITIES AND OTHER FEATURES ON THE PLAN ARE APPROXIMATE AND MAY NOT BE COMPLETE. ACTUAL LOCATION SHALL BE FIELD VERIFIED BY THE CONTRACTOR AS REQUIRED.

8. CONTRACTOR SHALL BE SOLELY RESPONSIBLE FOR WORKER SAFETY. ALL TRENCHING AND OTHER ACTIVITIES SHALL BE IN ACCORDANCE WITH STATE AND LOCAL SAFETY REGULATIONS AND REQUIREMENTS.

EROSION CONTROL NOTES

1. THE CONTRACTOR SHALL APPLY ALL MEASURES NECESSARY TO PREVENT THE DISCHARGE OF SEDIMENT-LADEN WATER OFF THE PROJECT SITE. FACILITIES SHOWN ON THE PLANS ARE THE MINIMUM REQUIREMENTS FOR ANTICIPATED SITE CONDITIONS.

2. THE CONTRACTOR SHALL INSPECT AND MAINTAIN ALL EROSION CONTROL FACILITIES REGULARLY, PARTICULARLY DURING AND FOLLOWING LARGE STORMS.

3. ALL STREETS ADJACENT TO THIS PROJECT SHALL BE KEPT CLEAN OF ALL MATERIAL DEPOSITS RESULTING FROM CONSTRUCTION.

4. SITE WORK SHALL BE SCHEDULED TO MINIMIZE THE EXPOSURE OF DISTURBED SOILS. ALL DISTURBED AREAS SHALL BE STABILIZED (CLEAR PLASTIC, MULCHING (SEE 4/C2) , ETC.) AS QUICKLY AS POSSIBLE AFTER COMPLETION OF WORK IN THE AREA. FROM OCTOBER 1 THROUGH APRIL 30, NO SOILS SHALL REMAIN EXPOSED FOR MOR THAN 2 DAYS. FROM MAY 1 THROUGH SEPTEMBER 30, NO SOILS SHALL REMAIN EXPOSED FOR MORE THAN 7 DAYS.

5. CARE SHALL BE TAKEN TO PREVENT ANY DISCHARGE OF SEDIMENT-LADEN WATER INTO THE STORMWATER SYSTEMS. ANY INLETS WHICH RECEIVE RUNOFF SHOULD PROTECTED WITH SEDIMENT FILTERS.

6. PETROLEUM PRODUCTS AND OTHER POTENTIAL POLLUTANTS SHALL BE PROTECTED TO PREVENT THEIR INTRODUCTION INTO SITE RUNOFF OR THE STORM DRAINAC SYSTEM.

7. THE CONTRACTOR SHALL DEVELOP A DEWATERING PL.AN TO PREVENT SEDIMENT LADEN WATER FROM DISCHARGING TO THE BEACH.

8. CLEARING LIMITS, IF SHOWN, SHALL BE CLEARLY FLAGGED PRIOR TO ANY CLEARING OR CONSTRUCTION ON THE SITE. DURING CONSTRUCTION, NO DISTURBANCE BEYOND THE FLAGGED CLEARING LIMITS SHALL BE PERMITTED. THE FLAGGING SHALL BE MAINTAINED BY THE CONTRACTOR UNTIL ALL CONSTRUCTION IS APPROVED.

9. ALL TEMPORARY EROSION CONTROL FACILITIES, INCLUDING PERIMETER CONTROLS, SHALL REMAIN IN PLACE UNTIL FINAL SITE CONSTRUCTION IS COMPLETE AND APPROVAL HAS BEEN RECEIVED FROM THE CITY.

SOIL AMENDMENT NOTES

AREAS TO BE DISTURBED SHALL HAVE THE TOPSOIL STRIPPED AND STOCKPILED. ALL DISTURBED AREAS THAT ARE TO BE LANDSCAPED SHALL HAVE THE STOCKPILED SOIL REPLACED OR SHALL BE AMENDED WITH COMPOST.

AREAS WHERE THE TOPSOIL WILL BE REPLACED SHALL HAVE THE SUBSOILS SCARIFIED TO 4 INCHES AND A MINIMUM DEPTH OF 8 INCHES OF TOPSOIL SHALL BE PLACED ON TOP OF SUBSOILS. 1-INCH OF COMPOSTED MATERIAL SHALL BE TILLED INTO THE TOP 4 INCHES OF TOPSOIL.

AMENDING THE SUBSOIL SHALL CONSIST OF SCARIFYING THE TOP 8-INCHES OF SOIL AND TILLING IN THE AMOUNT OF COMPOST DESCRIBED BELOW.

LANDSCAPED AREAS (10% ORGANIC CONTENT): PLACE AND TILL 3-INCHES OF COMPOSTED MATERIAL INTO TOP 5-INCHES OF SOIL. RAKE BEDS SMOOTH, REMOV ROCKS LARGER THAN 2-INCHES IN DIAMETER AND MULCH AREAS WITH 2-INCHES OF ORGANIC MULCH.

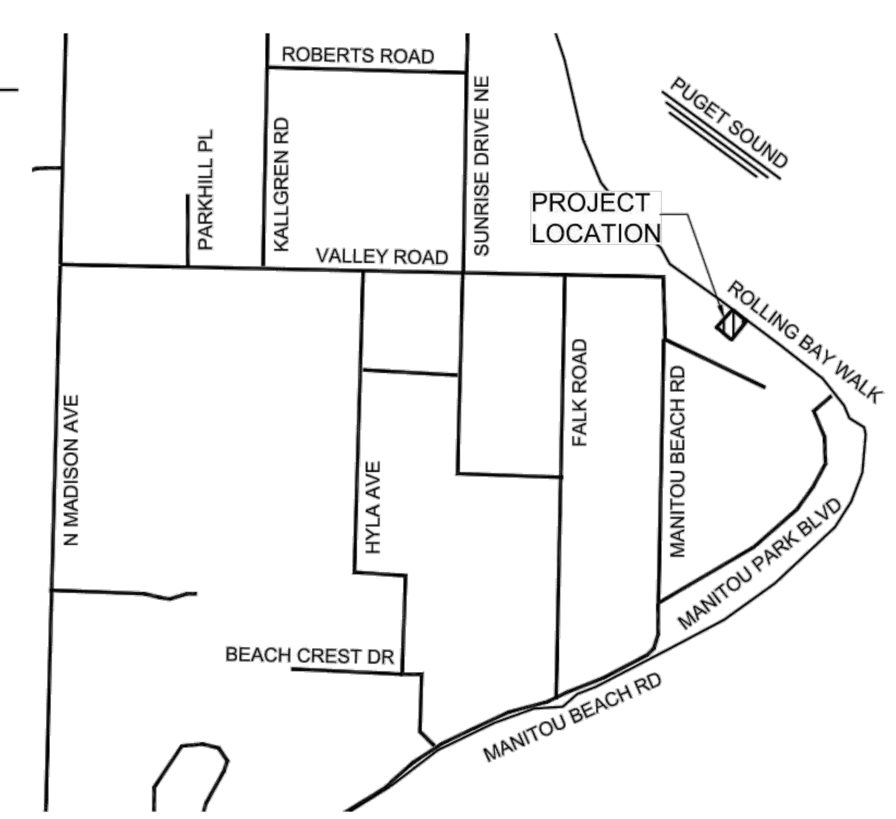
TURF AREAS (5% ORGANIC CONTENT) : PLACE AND TILL 1.75-INCHES OF COMPOSTED MATERIAL INTO 6.25-INCHES OF SOIL . WATER OR ROLL TO COMPACT SOIL T 85% OF MAXIMUM. RAKE TO LEVEL, AND REMOVE SURFACE WOODY DEBRIS AND ROCKS LARGER THAN 1-INCH IN DIAMETER.

DO NOT SCARIFY SOIL WITHIN THE DRIP-LINE OF EXISTING TREES TO BE RETAINED. WITHIN 3-FEET OF THE TREE DRIP-LINE, AMENDMENT SHOULD BE INCORPORATED NO DEEPER THAN 3 TO 4-INCHES TO REDUCE DAMAGE TO ROOTS.

MATURE COMPOST SHALL BE GRADE A COMPOST MEETING WSDOT STD SPEC 9-14.4(8).

LEGEND

- EXISTING HOUSE/DRIVEWAY
- EXISTING MAJOR CONTOUR
- EXISTING MINOR CONTOUR
- PROPERTY LINE
- PROPERTY CORNER/STAKE
- CL — PROPOSED CLEARING LIMITS FENCE
- F — PROPOSED SILT FENCE
- PROPOSED CLEANOUT
- SD — PROPOSED STORM DRAIN
- PROPOSED DITCH
- PROPOSED BUILDING



VICINITY MAP

SCALE: 1" = 1000'

ADAM E. WHEELER STATE OF WASHINGTON 40430 REGISTERED PROFESSIONAL ENGINEER 2-9-2017

DRAINAGE PLAN
DUFRESNE RESIDENCE
BUILDING PERMIT SUBMITTAL

BROWNE • WHEELER ENGINEERS, INC
241 ERICKSEN AVENUE NE
BAINBRIDGE ISLAND, WA 98110
P 206.842.0605 INFO@BrowneWheeler.COM

MARGARET DUFRESNE
3912 NE STATE HIGHWAY 104
POULBSO WA 98370

DATE 2/9/2017
DESIGNED AEW
DRAWN NDW
CHECKED AEW
PROJECT # DU05-001

DUFRENSE RESIDENCE
11143 Rolling Bay Walk NE
Bainbridge Island, WA

Soldier Pile & Timber Lagging Retaining Wall

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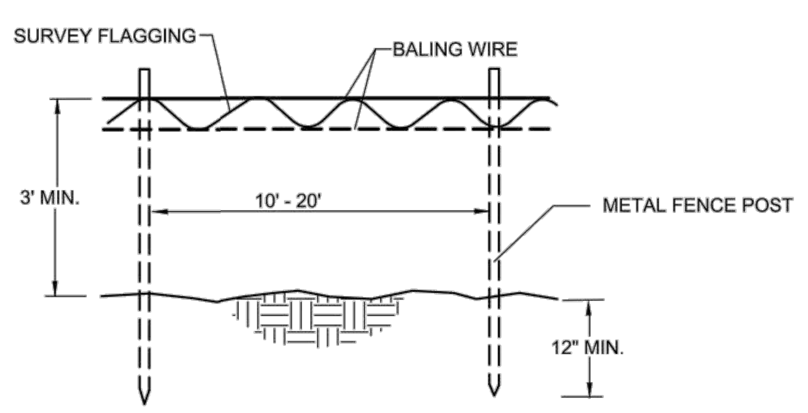
Seattle, Washington 98115
PHONE: (206) 790-8039
E-MAIL: joe@luciaeng.com

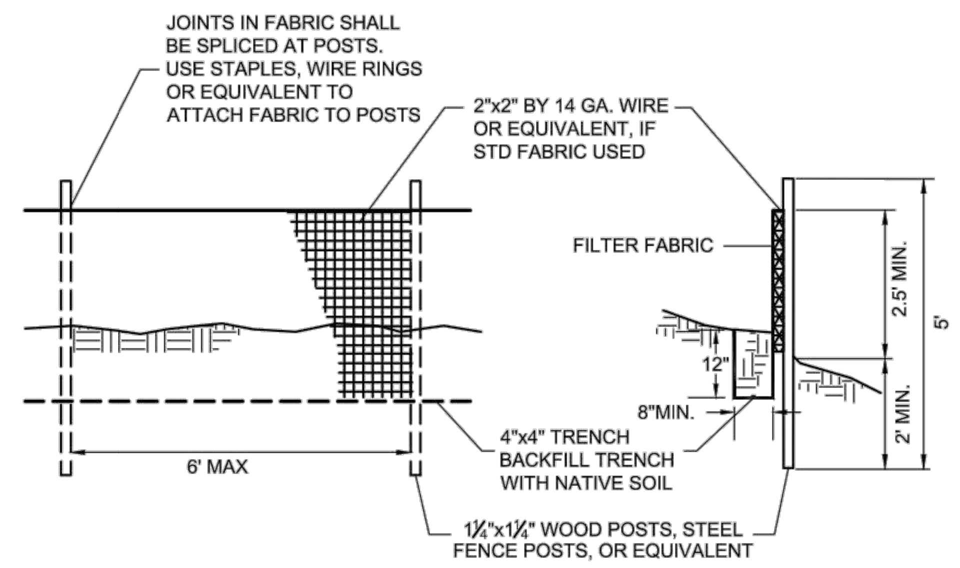
JOSEPH M. LUCIA STATE OF WASHINGTON 23314 REGISTERED PROFESSIONAL ENGINEER
11-15-17

Number	Date	By	Description
2	11-15-17		Bainbridge Island, WA

SHEET
SH-7.0

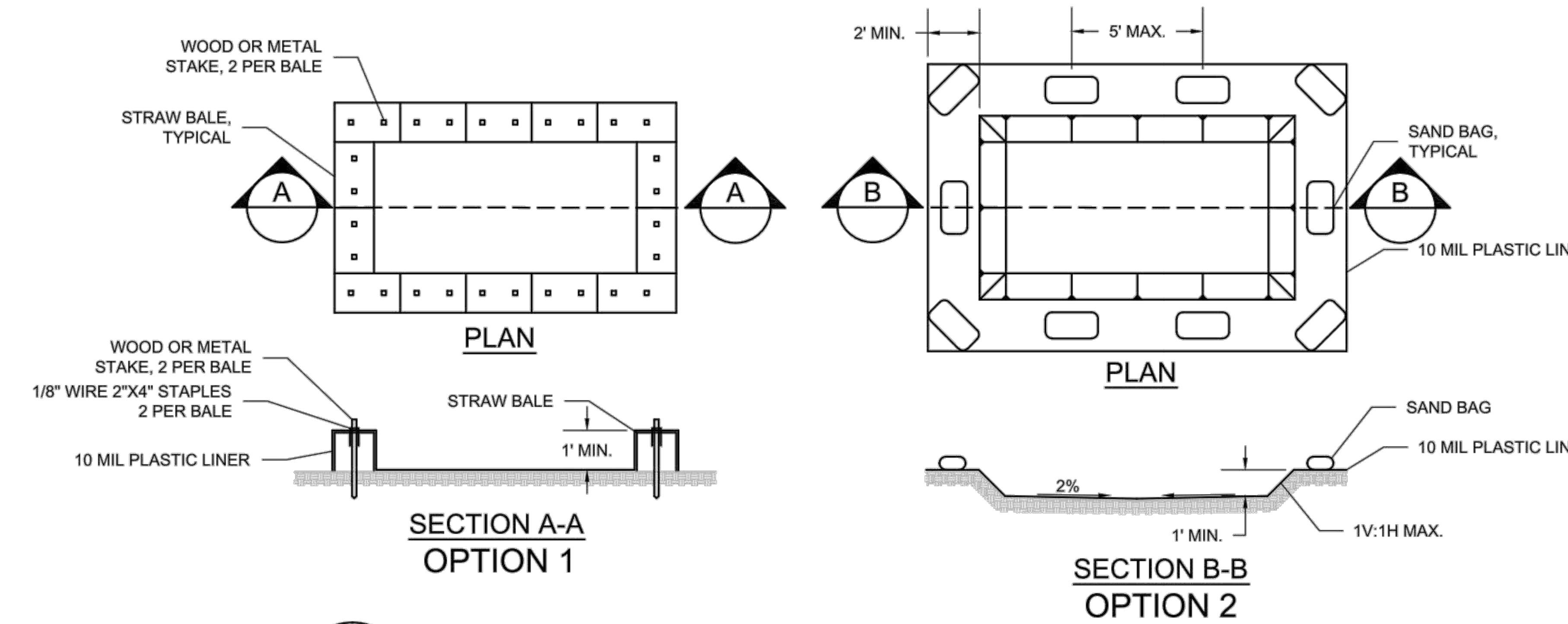


1 / C2 CLEARING LIMITS FENCE
NO SCALE

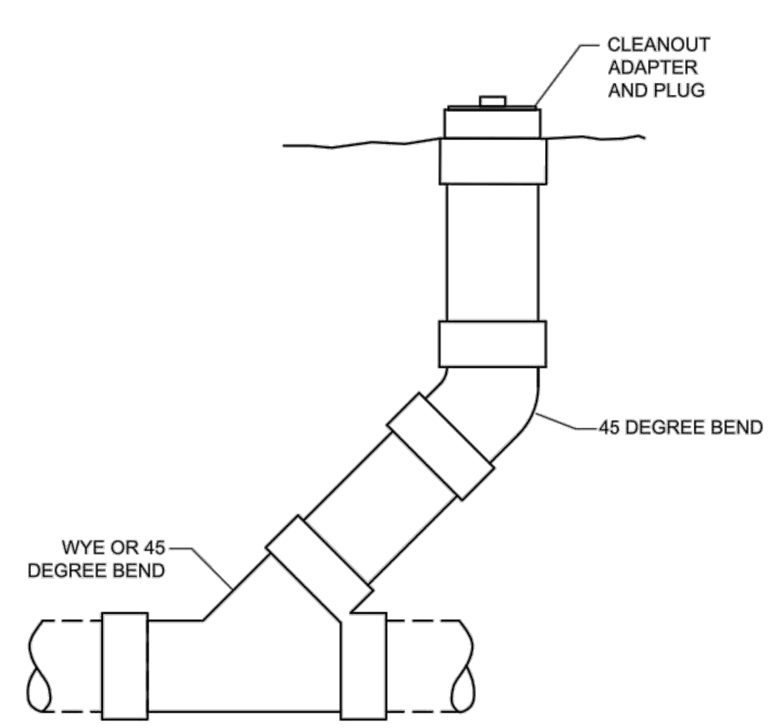


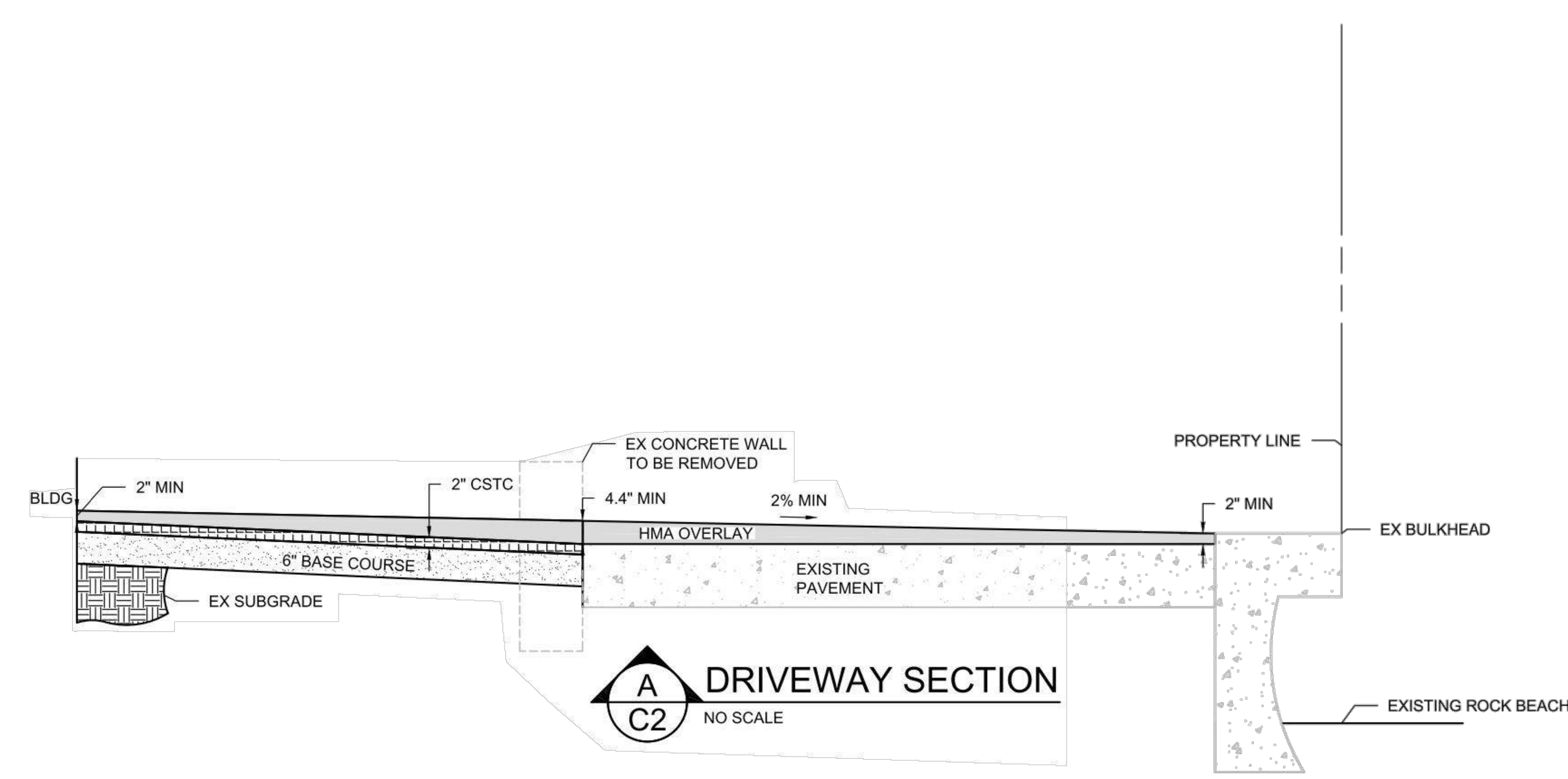
NOTES:
1. MAXIMIZE DETENTION OF STORMWATER BY PLACING FENCE AS FAR AWAY FROM TOE OF SLOPE AS POSSIBLE WITHOUT ENCROACHING ON SENSITIVE AREAS OR OUTSIDE CLEARING BOUNDARIES.
2. INSTALL SILT FENCING ALONG CONTOURS.
3. INSTALL THE ENDS OF THE SILT FENCE TO POINT SLIGHTLY UP-SLOPE TO PREVENT SEDIMENT FROM FLOWING AROUND THE ENDS OF THE FENCE.
4. PERFORM MAINTENANCE IN ACCORDANCE WITH STANDARD SPECIFICATIONS 8.01.3(9)A AND 8.01.3(15).
5. FILTER FABRIC SHALL MEET WSDOT 9-33.2(1) TABLE 6.

2 / C2 SILT FENCE
NO SCALE



3 / C2 CONCRETE CONTAINMENT AREA
NO SCALE





A / C2 DRIVEWAY SECTION
NO SCALE

MULCH REQUIREMENTS		
MULCH MATERIAL	QUALITY STANDARDS	APPLICATION RATES
STRAW	AIR-DRIED; FREE FROM UNDESIRABLE SEED AND COARSE MATERIAL.	2"-3" THICK; 5 BALES PER 1000 SF OR 2-3 TONS PER ACRE
HYDROMULCH	NO GROWTH INHIBITING FACTORS.	APPROX. 25-30 LBS PER 1000 SF OR 1500-2000 LBS PER ACRE
COMPOSTED MULCH AND COMPOST	NO VISIBLE WATER OR DUST DURING HANDLING. MUST BE PURCHASED FROM SUPPLIER WITH SOLID WASTE HANDLING PERMIT (UNLESS EXEMPT).	2" THICK MIN.; APPROX. 100 TONS PER ACRE (APPROX 800 LBS PER YARD)
CHIPPED SITE VEGETATION	AVERAGE SIZE SHALL BE SEVERAL INCHES. GRADATIONS FROM FINES TO 6 INCHES IN LENGTH FOR TEXTURE, VARIATION, AND INTERLOCKING PROPERTIES.	2" MINIMUM THICKNESS
WOOD-BASED MULCH	NO VISIBLE WATER OR DUST DURING HANDLING. MUST BE PURCHASED FROM A SUPPLIER WITH A SOLID WASTE HANDLING PERMIT OR ONE EXEMPT FROM SOLID WASTE REGULATIONS.	2" THICK; APPROX. 100 TONS PER ACRE (APPROX. 800 LBS PER CUBIC YARD)

4 / C2 MULCHING
NO SCALE

5 / C2 CLEANOUT
NO SCALE

ADAM E. WHEELER STATE OF WASHINGTON 40430 REGISTERED PROFESSIONAL ENGINEER 2-9-2017

DETAILS
DUFRESNE RESIDENCE
BUILDING PERMIT APPLICATION

BROWNE • WHEELER ENGINEERS, INC
241 ERICKSEN AVENUE NE
BAINBRIDGE ISLAND, WA 98110
P 206.842.0605 INFO@BrowneWheeler.COM

MARGARET DUFRESNE
3912 NE STATE HIGHWAY 104
POULBSO WA 98370

DATE 2-9-2017
DESIGNED AEW
DRAWN NDW
CHECKED AEW
PROJECT # DU05-001

DUFRENSE RESIDENCE
11143 Rolling Bay Walk NE
Bainbridge Island, WA

Soldier Pile & Timber Lagging Retaining Wall

LUCIA ENGINEERING, INC.
7307 12th Avenue N.E.
Seattle, Washington 98115
PHONE: (206) 790-8039
E-MAIL: joe@luciaeng.com

JOSEPH M. LUCIA STATE OF WASHINGTON 23314 REGISTERED PROFESSIONAL ENGINEER
11-15-17

Number	Date	By	Description
2	11-15-17		Bainbridge Island, WA

SHEET
SH-8.0

GENERAL NOTES

1. ALL CONSTRUCTION MATERIALS AND WORKMANSHIP SHALL CONFORM TO THE REQUIREMENTS OF THE DRAWINGS, SPECIFICATIONS, AND THE CODES, RULES AND REGULATIONS OF INTERNATIONAL BUILDING CODE, 2015 EDITION.
2. THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS PRIOR TO CONSTRUCTION. THE ARCHITECT SHALL BE NOTIFIED OF ANY DISCREPANCIES OR INCONSISTENCIES.
3. IF ANY ERRORS OR OMISSIONS APPEAR IN THESE DRAWINGS, SPECIFICATIONS, OR OTHER DOCUMENTS THE CONTRACTOR SHALL NOTIFY THE STRUCTURAL ENGINEER OR ARCHITECT IN WRITING OF SUCH OMISSION OR ERROR BEFORE PROCEEDING WITH THE WORK.
4. MANUFACTURED MATERIALS SHALL BE APPROVED BY THE CHECKING AGENCY PRIOR TO THEIR USE. ALL REQUIREMENTS OF THOSE APPROVALS SHALL BE FOLLOWED.
5. ALL STRUCTURAL SYSTEMS THAT ARE TO BE COMPOSED OF MANUFACTURED COMPONENTS TO BE FIELD ERECTED SHALL BE APPROVED BY THE CHECKING AGENCY PRIOR TO THEIR USE AND SHALL BE SUPERVISED BY THE SUPPLIER DURING MANUFACTURING, DELIVERY, HANDLING, STORAGE, AND ERECTION IN ACCORDANCE WITH INSTRUCTIONS PREPARED BY THE SUPPLIER
6. FRAMING MEMBERS THAT ARE NOT DIMENSIONED SHALL BE EQUALLY SPACED BETWEEN DIMENSIONED POINT OR MEMBERS.
7. SEE ARCHITECTURAL DRAWINGS AND PROJECT SPECIFICATIONS FOR THE FOLLOWING:
   - SIZE AND LOCATION OF ALL DOOR AND WINDOW OPENINGS AND THRESHOLD REQUIREMENTS.
   - SIZE AND LOCATION OF ALL NON-BEARING PARTITIONS.
   - SIZE AND LOCATION OF ROOF, FLOOR AND WALL OPENINGS.
   - SIZE AND LOCATION OF DEPRESSED AREAS, CHANGES IN ELEVATION, FLOOR AND ROOF DRAINS,
   - SLOPES, CONCRETE CURBS, LEDGES, PADS AND ISLANDS, CHAMFERS, GROOVES, INSERTS, ETC.
   - DIMENSIONS NOT SHOWN ON THE STRUCTURAL DRAWINGS, SIZE, WEIGHT AND LOCATION OF MACHINES AND EQUIPMENT BASES.
8. THE CONTRACT DOCUMENTS REPRESENT THE FINISHED STRUCTURE, THEY DO NOT INDICATE THE METHOD OF CONSTRUCTION. THE CONTRACTOR SHALL PROVIDE ALL MEASURES NECESSARY TO PROTECT THE STRUCTURE DURING CONSTRUCTION. SUCH MEASURES SHALL INCLUDE, BUT NOT BE LIMITED TO, BRACING, SHORING FOR LOADS DUE TO CONSTRUCTION EQUIPMENT, ETC. OBSERVATION VISITS TO THE SITE BY THE STRUCTURAL ENGINEER SHALL NOT INCLUDE INSPECTION OF THE ABOVE ITEMS.
9. OPENINGS, POCKETS, ETC. SHALL NOT BE PLACED IN STRUCTURAL MEMBERS UNLESS SPECIFICALLY DETAILED ON THE STRUCTURAL DRAWINGS. NOTIFY THE STRUCTURAL ENGINEER WHEN DRAWINGS BY OTHERS SHOW OPENINGS, POCKETS, ETC., LARGER THAN 6 INCHES NOT SHOWN ON THE STRUCTURAL DRAWINGS, BUT WHICH ARE LOCATED IN STRUCTURAL MEMBERS.
10. SPECIFICATIONS, CODES, AND STANDARDS NOTED IN THE CONTRACT DOCUMENTS SHALL BE OF THE LATEST APPROVED ISSUE, INCLUDING SUPPLEMENTS, UNLESS OTHERWISE NOTED. MATERIAL SPECIFICATIONS ARE ASTM LATEST EDITION.
11. CONTRACTOR SHALL PROVIDE TEMPORARY BRACING FOR THE STRUCTURE AND STRUCTURAL COMPONENTS UNTIL ALL FINAL CONNECTIONS HAVE BEEN COMPLETED IN ACCORDANCE WITH THE PLANS.

DESIGN CRITERIA

LIVE LOADS

ROOF SNOW LOAD	25.0 PSF BASIC
FLOOR	
RESIDENTIAL	40.0 PSF (REDUCIBLE)
EXTERIOR TERRACES	60.0 PSF

DEAD LOADS

SUPERIMPOSED ROOF DEAD LOAD	
FRAMING, CEILING, ETC.	9.5 PSF
SUPERIMPOSED FLOOR DEAD LOAD	
FRAMING, CEILING, ETC.	10 PSF
PLANTER -LIGHT WEIGHT TOP SOIL	90 PSF

WIND DESIGN (PER 1615 -1622)

BASIC WIND SPEED	110 MPH
EXPOSURE	D
IMPORTANCE FACTOR	1.0
TOPOGRAPHIC FACTOR	1.00

SEISMIC DESIGN (PER 1615 - 1633)
SEISMIC CATEGORY II
IMPORTANCE FACTOR 1.0
MAPPED SPECTRAL RESPONSE ACCELERATION PARAMETERS:
S_S 1.352
S_1 0.531 SITE CLASS D
S_{DS} 0.901 S_{D1} 0.531
SEISMIC RISK CATEGORY D
BASIC SEISMIC FORCE-RESISTING SYSTEMS:
STORIES 1 THRU 3: LIGHT FRAMED WALLS SHEATHED WITH WOOD STRUCTURAL PANELS RATED FOR SHEAR RESISTANCE.
MARTIN STREET & HARVARD AVE:
SPECIAL REINFORCED CONCRETE SHEAR WALLS
DESIGN BASE SHEAR: 35.11 KIPS
R 6.5 - Wood Framed
R 5.0 - Concrete
ANALYSIS METHODS USED:
WIND METHOD 2 - ANALYTICAL PROCEDURE
SEISMIC METHOD 2 - EQUIVALENT LATERAL FORCE

MAPPED SPECTRAL RESPONSE ACCELERATIONS OBTAINED FROM THE USGS - SEISMIC HAZARD MAPS & DATA

FOUNDATIONS

1. ALL FOUNDATIONS SHALL BE FOUNDED A MINIMUM OF 18" BELOW LOWEST ADJACENT FINAL FINISH FLOOR OR GRADE. EXPOSED SOIL SHALL BE INSPECTED FOR COMPLIANCE BY THE ENGINEER OR HIS REPRESENTATIVE PRIOR TO CONSTRUCTING CONCRETE FORMS AND/OR PLACING REINFORCING STEEL. ANY EXCESS OR NON-COMPLYING MATERIAL AS DETERMINED BY THE ENGINEER OR HIS REPRESENTATIVE SHALL BE REMOVED AND REPLACED AS DIRECTED.
2. THE ALLOWABLE SOIL BEARING LOAD IS PER THE GEOTECHNICAL REPORT.

REINFORCING STEEL

1. REINFORCING STEEL SHALL BE DETAILED, INCLUDING HOOKS AND BENDS, AND PLACED IN ACCORDANCE WITH ACI 315 AND ACI 318.
2. REINFORCING STEEL SHALL CONFORM TO ASTM A-615 OR A-706, GRADE 40 OR BETTER,
3. WELDED WIRE FABRIC SHALL CONFORM TO ASTM A-185,
4. ALL REINFORCING BAR BENDS SHALL BE MADE COLD,
5. REINFORCING SPLICES SHALL BE MADE AS INDICATED ON THE DRAWINGS,
6. DOWELS BETWEEN FOOTINGS AND WALLS OR COLUMNS SHALL BE THE SAME GRADE, SIZE AND SPACING AS THE VERTICAL REINFORCING, RESPECTIVELY, UON,
7. NO BARS PARTIALLY EMBEDDED IN HARDENED CONCRETE SHALL BE FIELD BENT UNLESS SPECIFICALLY SO DETAILED AND REVIEWED BY THE STRUCTURAL ENGINEER
8. WELDING OF REINFORCEMENT SHALL BE WITH LOW HYDROGEN ELECTRODES IN CONFORMANCE WITH ACI 318-95 AND THE RECOMMENDATIONS OF THE AMERICAN WELDING SOCIETY, AWS DI,4 AND WITH THE REVIEW OF THE STRUCTURAL ENGINEER

CONCRETE

1. ALL CONCRETE CONSTRUCTION SHALL CONFORM TO THE BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE' ACI 318 AND ACI 301, WITH MODIFICATIONS AS NOTED IN THE CONTRACT DOCUMENTS,
2. PORTLAND CEMENT SHALL CONFORM TO ASTM C-150 TYPE 1 OR TYPE II,
3. COARSE AND FINE AGGREGATE FOR NORMAL WEIGHT CONCRETE SHALL CONFORM TO ASTM C-33,
4. WATER SHALL BE CLEAR AND SHALL CONFORM TO ASTM C-94,
5. CONCRETE MIXING OPERATION SHALL CONFORM TO ASTM C-94,
6. ADD TO ALL CONCRETE EXPOSED TO WEATHER MICROAIR OR MBVR AIR ENTRAINING AGENT TO ATTAIN 5 PERCENT +1- 1 PERCENT ENTRAINED AIR, BY VOLUME, CONFORMING TO ASTM C-260, ALL REFERENCE DATA USED FOR PAST PERFORMANCE DESIGN SHALL HAVE CONTAINED THE SAME ADMIXTURE BRAND AS THAT USED IN THE MIX SUBMITTED,
7. CONCRETE STRENGTHS SHALL BE VERIFIED BY 28-DAY CYLINDER TESTS, UNLESS OTHERWISE APPROVED, CONCRETE SHALL BE AS FOLLOWS:

ELEMENT TYPE	STRENGTH PSI CONCRETE
FOOTINGS, GRADE BEAMS	2,500 NORMAL WT
SLAB ON GRADE	2,500 NORMAL WT
ELEVATED PT SLAB	4,000 NORMAL WT

A MINIMUM 5 SACK MIX SHALL BE USED TO ACHIEVE THE DESIGN STRENGTHS LISTED ABOVE.

8. CONTRACTOR MAY USE AN ADMIXTURE SYSTEM TO PRODUCE FLOWABLE CONCRETE, MAXIMUM SLUMP SHALL NOT EXCEED 10 INCHES MEASURED AT THE PUMP, THE WATER/CEMENTIOUS MATERIAL RATIO OF THE APPROVED MIXES SHALL BE MAINTAINED OR LOWERED WHEN FLOWABLE CONCRETE IS USED,
9. THE FOLLOWING MINIMUM CONCRETE COVER SHALL BE PROVIDED FOR REINFORCEMENT PLACED IN CAST-IN-PLACE CONCRETE:

	CONCRETE COVER (MINIMUM)
A . CONCRETE CAST AGAINST AND PERMANENTLY EXPOSED TO EARTH	3"
B. CONCRETE EXPOSED TO EARTH OR WEATHER:	
#6 THROUGH #18 BARS	2"
#5 BAR, W31 OR D31 WIRE, A1413 SMALLER	1 1/2"
C. CONCRETE NOT EXPOSED TO WEATHER OR IN CONTACT WITH GROUND:	
SLABS, WALLS, JOISTS	
#14 AND #18 BARS	1 1/2"
#11 BARS AND SMALLER	3/4"
BEAMS, COLUMNS:	
PRIMARY REINFORCEMENT, TIES, STIRRUPS, SPIRALS	1 1/2"

10. PLACEMENT OF CONCRETE SHALL CONFORM TO ACI 304 AND THE CONTRACT DOCUMENTS. SANDBLAST ALL CONCRETE SURFACES AGAINST WHICH CONCRETE IS TO BE PLACED,
11. ALL REINFORCING BARS, ANCHOR BOLTS AND OTHER CONCRETE INSERTS SHALL BE WELL SECURED IN POSITION PRIOR TO PLACING CONCRETE,
12. PROVIDE SLEEVES FOR PLUMBING AND ELECTRICAL OPENINGS IN CONCRETE BEFORE PLACING, REINFORCING SHALL NOT BE CUT, CORING OF CONCRETE IS NOT PERMITTED EXCEPT AS INDICATED,
13. CURING COMPOUNDS USED ON CONCRETE TO RECEIVE A FINISH SHALL BE APPROVED BY THE FINISH APPLICATOR BEFORE USE,

WOOD

1. FRAMING LUMBER SHALL BE GRADED AND MARKED IN CONFORMANCE WITH WCLB STANDARD GRADING AND DRESSING RULES FOR WEST COAST LUMBER NO, 16, LATEST EDITION. UNLESS OTHERWISE NOTED ON THE DRAWINGS, LUMBER GRADES SHALL BE AS FOLLOWS:
   A. JOISTS: 2" AND 3" THICKNESS, HEM FIR NO, 1,
   B. BEAMS AND STRINGERS: DOUGLAS FIR NO, 1,
   C. POST AND TIMBERS: DOUGLAS FIR NO, 1,
   D. PLATES AND MISCELLANEOUS LIGHT FRAMING: HEM FIR STANDARD,
   E. STUDS: HEM FIR STUD.
   F. ALL BOLTED CONNECTIONS TO BE 3/4"Ø A302 BOLTS

2. MINIMUM NAILING REQUIREMENTS:

   UNLESS OTHERWISE NOTED, MINIMUM NAILING SHALL CONFORM TO THE GOVERNING CODE AND AS FOLLOWS:
   A. JOISTS OR RAFTERS TO SIDES OF STUDS 8-INCH OR LESS 3-16DB
   B. FOR EACH ADDITIONAL 4-INCH IN DEPTH OF JOISTS 1-16DC
   C. JOISTS OR RAFTERS AT ALL BEARINGS - TOENAILS EACH SIDE 2-10DD
   D. STUDS TO BEARING - TOENAILS EACH SIDE 2-10DE
   E. BLOCKING BETWEEN JOISTS OR RAFTERS
      TO JOIST OR RAFTERS - TOENAILS EACH SIDE EACH END 2-10D
      TO JOIST OR RAFTER BEARINGS - TOENAILS EACH SIDE 2-10D
   F. CROSS-BRIDGING BETWEEN JOISTS OR RAFTERS TOE NAILS EACH END 2-8D
   G. BLOCKING BETWEEN STUDS - TOENAILS EACH END 2-10D
   H. DOUBLE TOP PLATES - LOWER PLATE TO TOP OF STUD 2-16D
   J. UPPER TO LOWER PLATE - STAGGERED 16D @ 16" O,C,
   K. MULTIPLE JOISTS - STAGGERED 16D @ 12" O,C,
   L. MULTIPLE JOISTS STAGGER FOR WIDTHS MORE THAN 4 INCHES 16D @ 12" O,C.

3. INDIVIDUAL MEMBERS OF BUILT-UP POSTS AND BEAMS SHALL EACH BE ATTACHED WITH 16D SPIKES AT 12" O,C. STAGGERED, MIN,
4. ALL NAILS SHALL BE COMMON WIRE NAILS, WHENEVER POSSIBLE, NAILS DRIVEN PERPENDICULAR TO THE GRAIN SHALL BE USED, THERE SHALL BE A MINIMUM OF 2 NAILS AT ALL WOOD CONTACTS AND JOINTS USING 8D NAILS FOR 1-INCH THICK MATERIAL, 16D NAILS FOR 2-INCH THICK MATERIAL, AND 40D NAILS FOR 3-INCH THICK MATERIAL, ALL CONTINUOUS CONTACTS PROVIDE MINIMUM NAILS AT 12" O,C, WITH NAIL SIZES AS CALLED ABOVE,
5. NOTATIONS ON DRAWINGS RELATING TO FRAMING CLIPS, JOIST HANGERS, AND OTHER CONNECTING DEVICES REFER TO CATALOG NUMBERS OF STRONG-TIE CONNECTORS MANUFACTURED BY THE SIMPSON COMPANY, EQUIVALENT DEVICES BY OTHER MANUFACTURERS MAY BE SUBSTITUTED PROVIDED THAT THEY HAVE ICBO APPROVAL FOR EQUAL OR GREATER LOAD CAPACITIES AND ARE REVIEWED BY THE STRUCTURAL ENGINEER,
6. AT SAWN TIMBER JOISTS WITH THICKNESS-TO-DEPTH RATIO OF 1:6 AND GREATER, PROVIDE CROSS-BRIDGING AT 8' 0" O,C, AND SOLID BLOCKING AT BEARING POINTS,
7. ALL WOOD FRAMING DETAILS NOT SHOWN OTHERWISE SHALL BE CONSTRUCTED TO THE MINIMUM STANDARDS OF THE GOVERNING CODE,
8. ALL BEARING AND EXTERIOR STUD WALLS SHALL BE 2X6 @6"O.C. BELOW SECOND FLOOR AND 2X4 @ 16" O.C. ELSEWHERE, UNLESS OTHERWISE NOTED.
9. PROVIDE CONTINUOUS SOLID BLOCKING AT MID-HEIGHTS AND AT INTERVALS NOT TO EXCEED 8 FEET OF ALL STUD-BEARING WALLS OVER 8 FEET IN HEIGHT,
10. SEE ARCHITECTURAL DRAWINGS FOR LOCATIONS OF INTERIOR NONBEARING STUD PARTITIONS FOR LOCATION AND SIZE OF OPENINGS IN STUD WALLS, AND FOR ALL WALL FINISH DETAILS,
11. ALL CANTS AND CRICKETS SHALL BE PLACED OVER BASIC ROOF SHEATHING. SEE ARCHITECTURAL DRAWINGS FOR DETAILS AND LOCATIONS,
12. ALL WOOD STUD WALL SILL PLATES SHALL BE ATTACHED TO CONCRETE OR MASONRY WITH 1/2-INCH DIAMETER ANCHOR BOLTS AT 48" O.C,, UNLESS OTHERWISE NOTED ,
13. ALL WOOD STUD WALLS SHALL HAVE LOWER WOOD PLATE ATTACHED TO WOOD FRAMING BELOW WITH 16D NAILS AT 6" 0.C, STAGGERED UNLESS SHOWN OTHERWISE.
14. FASTEN ALL POSTS TO CONCRETE WITH "CB" COLUMN BASE OR EQUAL,
15. ALL WOOD PLATES AND BLOCKING IN DIRECT CONTACT WITH CONCRETE OR MASONRY SHALL BE PRESSURE TREATED WITH AN APPROVED PRESERVATIVE IN ACCORDANCE WITH AWPS-FDN, AND BEAR THAT QUALITY MARK,
16. PROVIDE STANDARD CUT WASHERS UNDER ALL BOLTS HEADS AND NUTS IN CONTACT WITH WOOD.
17. ATTACH TIMBER JOISTS TO FLUSH HEADERS AND BEAMS WITH "U" SERIES METAL JOIST HANGERS TO SUIT THE JOIST SIZE.
18. ALL PLYWOOD SHALL BE HEM FIR, STRUCTURAL 2 OR BETTER AND SHALL CONFORM TO APA C-D INTERIOR GRADE WITH EXTERIOR GLUE, WITH UBC STANDARD 23-2 AND WITH PRODUCT STANDARD PSI, WOOD-BASED STRUCTURAL-USE PANELS SHALL CONFORM WITH UBC STANDARD 23-3 AND WITH PRODUCT STANDARD PS2, TYPE AND THICKNESS SHALL BE AS SPECIFIED ON THE PLANS.
19. PLYWOOD NAILING, USE UNLESS OTHERWISE NOTED:

A. ROOF:	8D @ 6" O.C. AT SHEET EDGES
	8D @ 12" O.C. AT INTERMEDIATE BEARING POINTS
B. FLOOR:	10D @ 6" O.C. AT SHEET EDGES
	10D @ 10" O.C. AT INTERMEDIATE BEARING POINTS
C. WALLS:	8D @ 6" O.C. AT EDGES
	8D @ 12" O.C. AT INTERMEDIATE BEARING POINTS

PLYWOOD AND WOOD-BASED STRUCTURAL-USE PANELS USED FOR WALL SHEATHING SHALL HAVE SOLID BLOCKING AT ALL EDGES.

20. MACHINE APPLIED NAILING IS SUBJECT TO A SATISFACTORY DEMONSTRATION AND THE APPROVAL OF THE CHECKING AGENCY AND THE ARCHITECT, NAIL HEADS SHALL NOT PENETRATE THE OUTER PLY MORE THAN WOULD BE NORMAL FOR A HAND HAMMER. EDGE DISTANCES SHALL BE MAINTAINED, SHINERS SHALL BE REMOVED AND REPLACED, THE APPROVAL IS SUBJECT TO CONTINUED SATISFACTORY PERFORMANCE, MACHINE APPLIED NAILING ONLY ON PLYWOOD GREATER THAN 5/16".

STRUCTURAL STEEL, MISC. METAL

1. STRUCTURAL STEEL DETAILING, FABRICATION AND ERECTION SHALL BE BASED ON THE LATEST EDITION AND SUPPLEMENTS OF THE AISC "SPECIFICATION FOR STRUCTURAL STEEL FOR BUILDINGS - ALLOWABLE STRESS DESIGN AND PLASTIC DESIGN", STRUCTURAL STEEL SHALL CONFORM TO THE FOLLOWING REQUIREMENTS,

TYPE OF MEMBER	ASTM SPECIFICATION	FY
WIDE FLANGE SHAPES	A572 OR A992	50 KSI
PLATES, SHAPES, ANGLES, AND RODS	A36	36 KSI
HOLLOW STRUCTURAL SECTION (ROUND)	A53 (GRADE B)	36 KSI
HOLLOW STRUCTURAL SECTION (SQUARE OR RECTANGLE)	A500 (GRADE B)	46 KSI
ANCHOR RODS (EMBEDDED IN CONCRETE)	A307	

2. ALL WELDS SHALL BE PREQUALIFIED IN CONFORMANCE WITH AISC AND AWS STANDARDS AND SHALL BE PERFORMED BY WELDERS CERTIFIED IN THE JURISDICTION HAVING AUTHORITY OVER THIS PORTION OF THE WORK, USE E70XX ELECTRODES.3, WELD LENGTHS CALLED FOR ON THE PLANS ARE THE NET EFFECTIVE LENGTH REQUIRED, WELD SIZE SHALL BE AISC MINIMUM, UNLESS OTHERWISE NOTED.

ANCHORAGE

1. EXPANSION ANCHORS SHALL BE ZINC PLATED IN ACCORDANCE WITH ASTM B 633, AND CONFORM WITH FS FF-S-325, GROUP II, TYPE 4, CLASS 1,
2. SLEEVE ANCHORS SHALL BE ZINC PLATED IN ACCORDANCE WITH ASTM B 633, AND CONFORM WITH FS FF-S-325, GROUP II, TYPE 3, CLASS 3,
3. FLUSH SHELL ANCHORS SHALL ZINC PLATED IN ACCORDANCE WITH ASTM B 633, AND CONFORM WITH FS FF-S-325, GROUP VIII, TYPE 1,
4. ADHESIVE ANCHORS SHALL CONSIST OF ALL-THREAD ANCHOR ROD, NUT, WASHER AND EPOXY INJECTION GEL OR ADHESIVE CAPSULE SYSTEM, ANCHOR RODS SHALL BE MANUFACTURED FROM A-36 MATERIAL, ZINC PLATED IN ACCORDANCE WITH ASTM B 633,
5. ALL RELATED PRODUCTS, MATERIALS AND INSTALLATION SHALL BE IN ACCORDANCE WITH THE MANUFACTURER'S RECOMMENDATIONS,
6. NOTATIONS ON DRAWINGS RELATING TO EXPANSION, SLEEVE, FLUSH OR ADHESIVE ANCHORS AND OTHER CONNECTING DEVICES REFER TO CONNECTORS MANUFACTURED BY POWERS FASTENING, INC, EQUIVALENT DEVICES BY OTHER MANUFACTURERS MAY BE SUBSTITUTED PROVIDED THAT THEY HAVE ICBO APPROVAL FOR EQUAL OR GREATER LOAD CAPACITIES AND ARE REVIEWED BY THE STRUCTURAL ENGINEER

SPECIAL INSPECTION

1. SPECIAL INSPECTION BY A REGISTERED DEPUTY BUILDING INSPECTOR, APPROVED BY THE ARCHITECT AND THE CHECKING AGENCY SHALL BE REQUIRED FOR THE FOLLOWING TYPES OF WORK. SEE THE PROJECT SPECIFICATIONS FOR FURTHER REQUIREMENTS, SPECIAL INSPECTIONS SHALL NOT BE REQUIRED WHEN THE WORK IS DONE ON THE PREMISES OF A FABRICATOR REGISTERED AND APPROVED BY THE BUILDING OFFICIAL TO PERFORM SUCH WORK WITHOUT SPECIAL INSPECTION,
   - SOIL
     - EXCAVATION
     - SOIL COMPACTION
   - CONCRETE
     - DESIGN STRENGTHS GREATER THAN 2,500 PSI PLACING OF REINFORCING STEEL
   - WELDING
     - STRUCTURAL STEEL
     - REINFORCING STEEL
   - FABRICATED TIMBER JOISTS
   - EXPANSION TYPE ANCHOR BOLTS
   - STRUCTURAL MASONRY CONSTRUCTION
   - PILING, DRILLED OR DRIVEN
   - STRUCTURAL STEEL FABRICATION
2. ALL PREPARED SOIL-BEARING SURFACES SHALL BE INSPECTED BY THE SOILS ENGINEER PRIOR TO PLACEMENT OF REINFORCING STEEL,
3. EXPANSION TYPE ANCHORS SHALL BE APPROVED BY THE CHECKING AGENCY FOR THEIR USE AND SHALL BE INSTALLED ACCORDING TO THE MANUFACTURER'S RECOMMENDATIONS,
4. THE OWNER, ARCHITECT, STRUCTURAL ENGINEER, AND BUILDING OFFICIAL SHALL BE FURNISHED WITH COPIES OF ALL TEST RESULTS.

GEOTECHNICAL REFERENCE:
GEOTECHNICAL REPORT BY: ASPECT CONSULTING
REPORT DATED: JULY 20, 2017

DUFRENSE RESIDENCE
11143 Rolling Bay Walk NE
Bainbridge Island, WA

Soldier Pile & Timber Lagging
Retaining Wall

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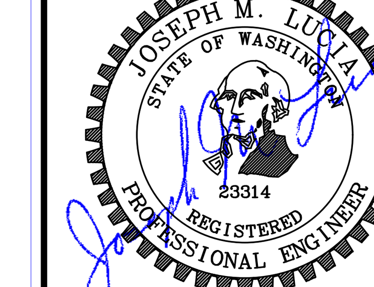
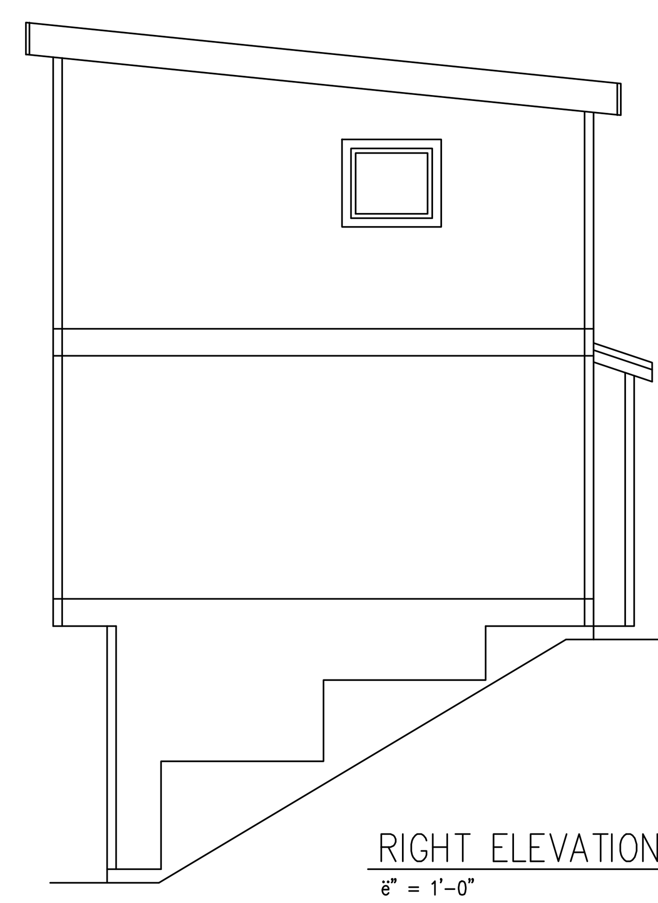


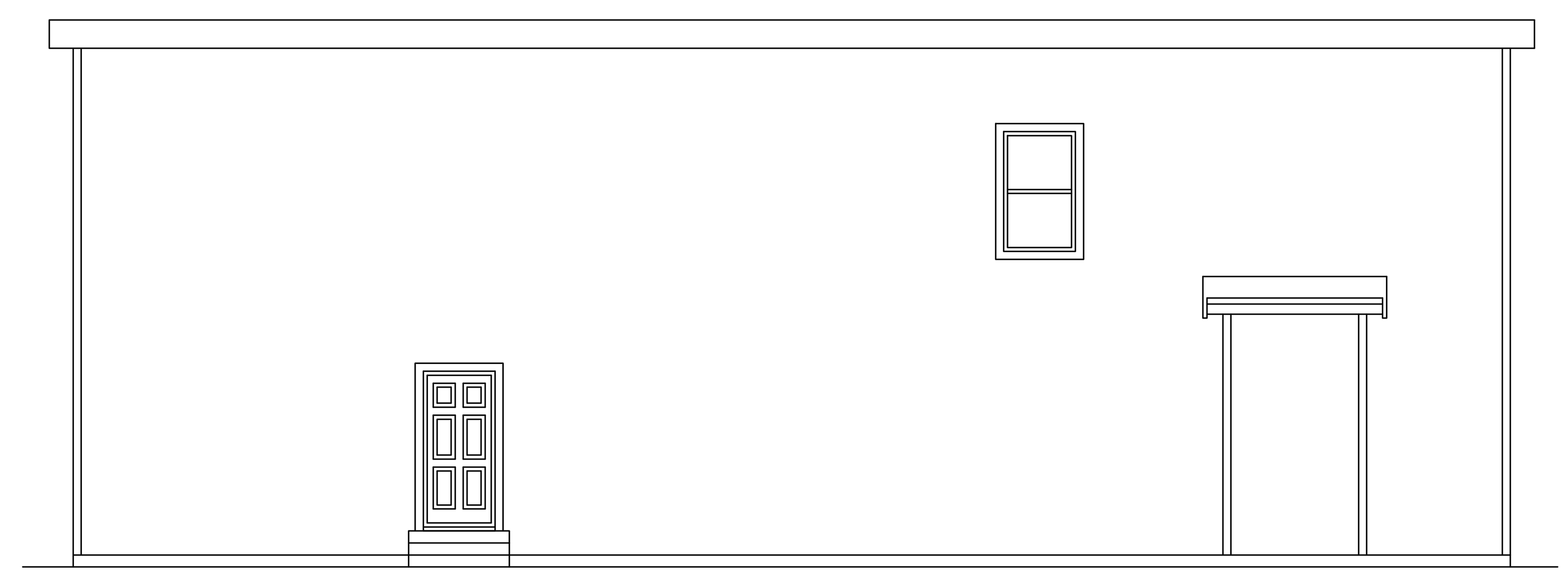
11-01-17

Number	Date	By	Description
1	11-01-17		Bainbridge Island, WA

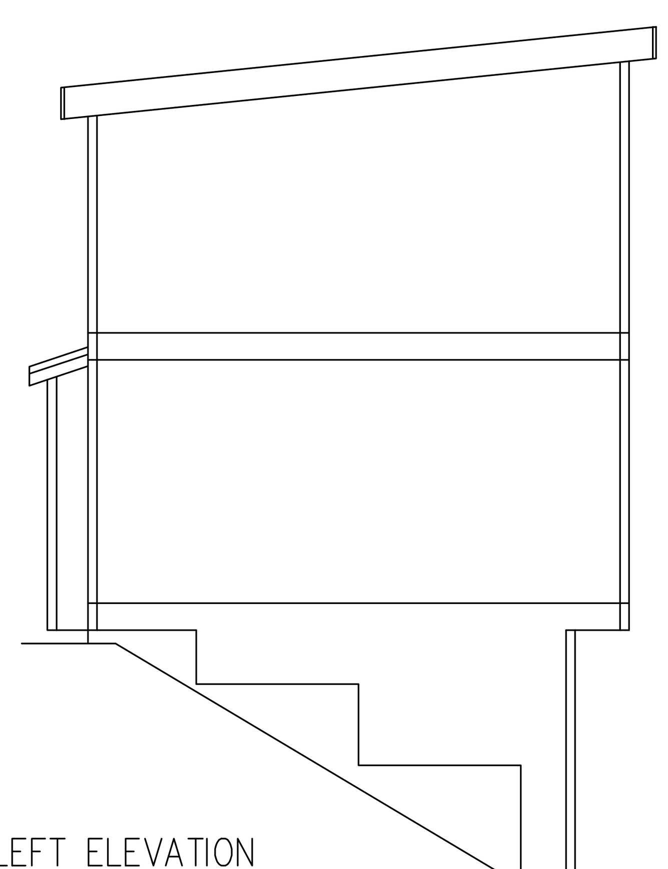
SHEET
SH-9.0



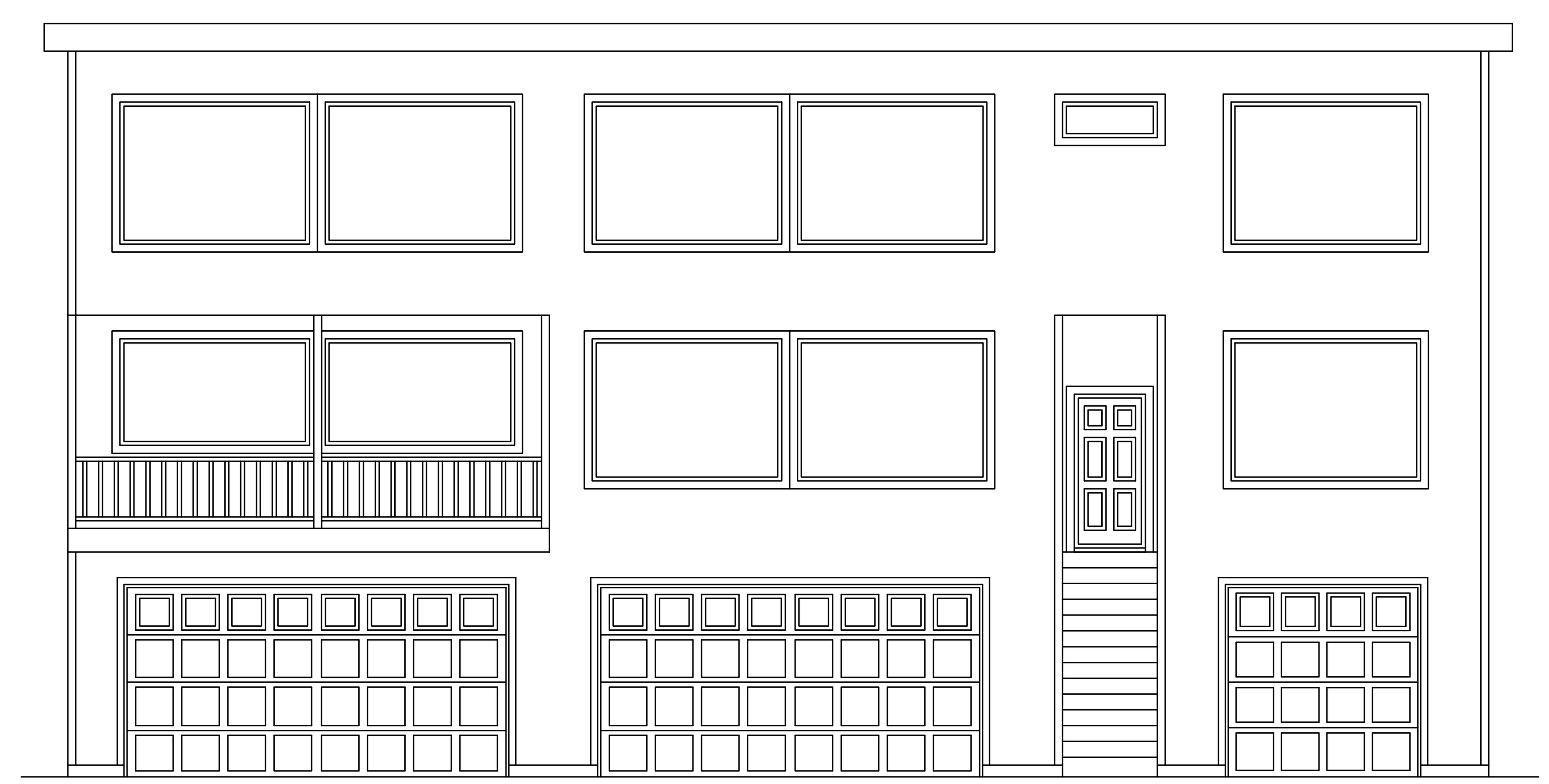
RIGHT ELEVATION
ë" = 1'-0"



REAR ELEVATION
ë" = 1'-0"



LEFT ELEVATION
ë" = 1'-0"



FRONT ELEVATION
ë" = 1'-0"

DUFRENSE RESIDENCE
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Bainbridge Island, WA

Soldier Pile & Timber Lagging Retaining Wall

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STATE OF WASHINGTON
23314
REGISTERED PROFESSIONAL ENGINEER
11-15-17

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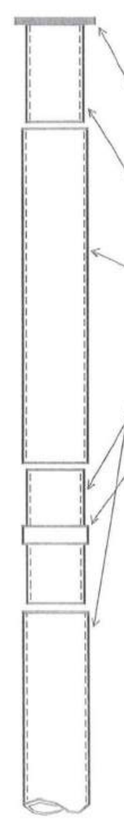
TYPICAL PIPE PILE ASSEMBLY

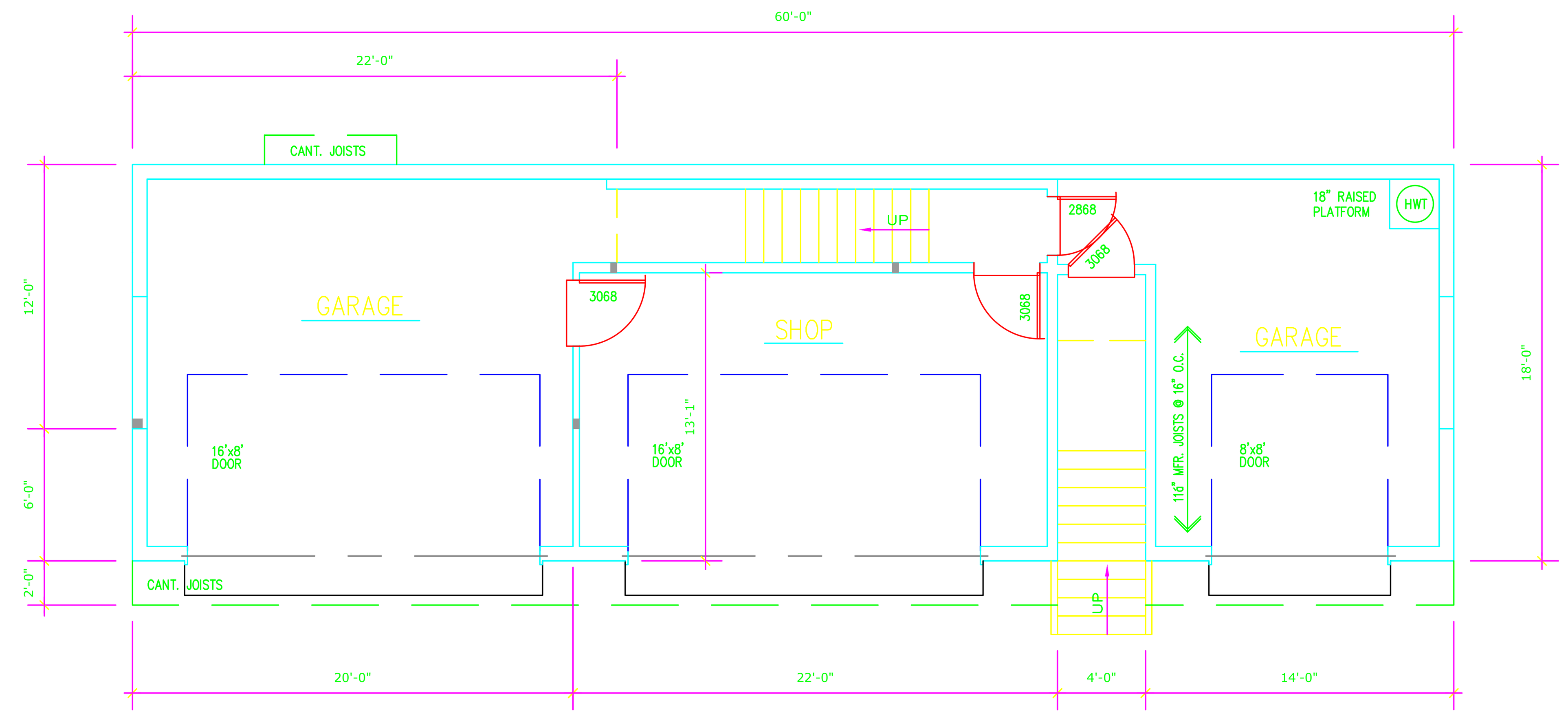


2" Sch 80 Pipe	3" Sch 40 Pipe	4" Sch 40 Pipe	6" Sch 40 Pipe
2.375" O.D	3.5" O.D.	4.5" O.D.	6.625" O.D.
.375" x 4" x 4" or .50" x 6" x 6" (Compression Fit Top Plate)	.375" x 4" x 4" or .50" x 6" x 6" (Compression Fit Top Plate)	.50" x 6" x 6" (Compression Fit Top Plate)	.50 x 8" x 8" (Compression Fit Top Plate)
5" x 1.5" (Sleeve)	5" x 2.5" (Sleeve)	5" x 3.5" (Sleeve)	5" x 5.5" (Sleeve)
2.375" Sch. 80 (.218) (typical 10.5' lengths)	3.5" Sch. 40 (.216) or Sch.80 (.300) (typical 10.5' - 21' lengths)	4.5" Sch. 40 (.237) (typical 21' lengths)	6.625" Sch. 40 (.280) (typical 21' lengths)
10" x 1.5" Sch. 80 (.200) (Sleeve)	12" x 2.5" Sch. 40 (.203) (Sleeve)	14" X 3.5" Sch. 40 (.226) (Sleeve)	16" x 6" O.D. (.250) A513/5 dom. tubing (Sleeve)
2.375" x 1.25" (Coupling Ring)	3.5" x 1.5" (Coupling Ring)	4.5" X 1.75" (Coupling Ring)	6.625" O.D. Sch. 80 x 2" (Coupling Ring)

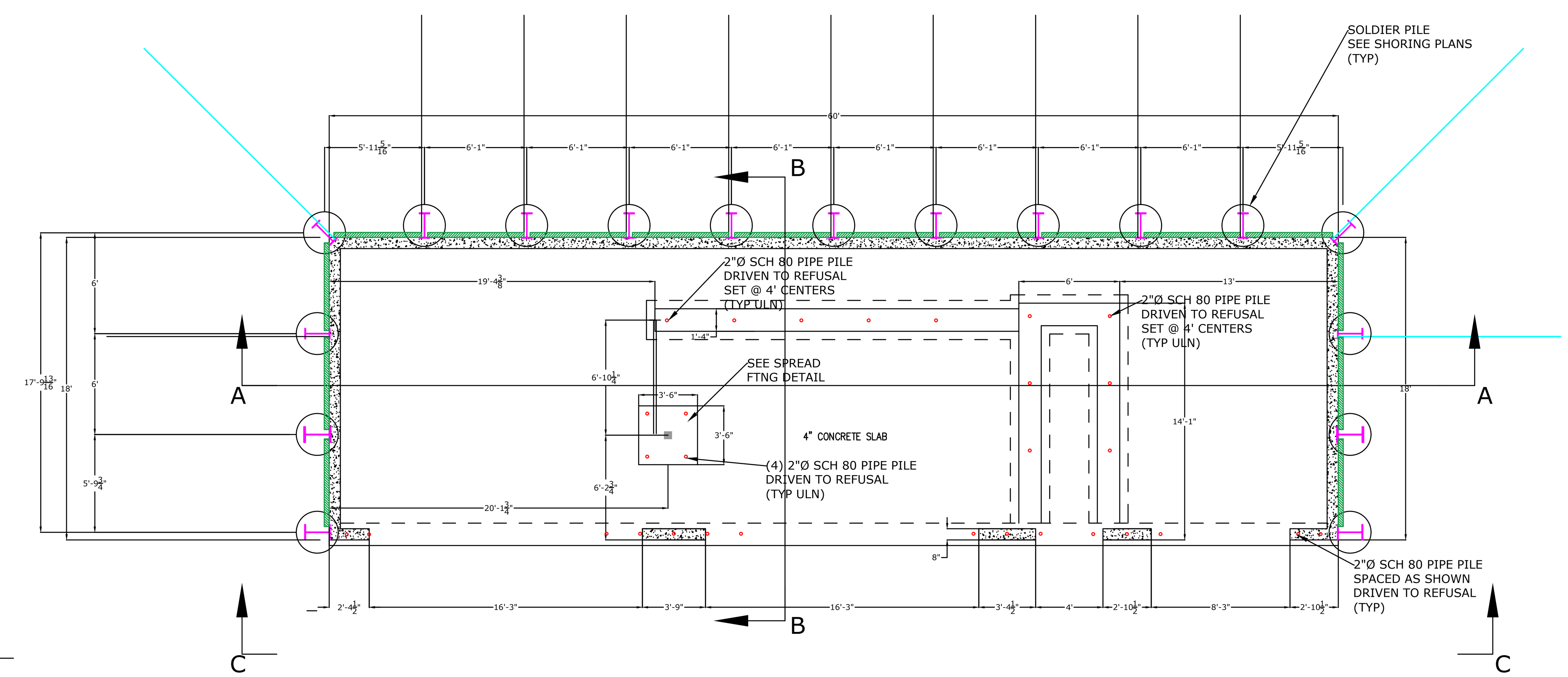
HAMMER	WEIGHTS (Actual)	WEIGHTS (Class)	Blows per Minute @ 80 % Throttle	SECONDS PER INCH 2" PIPE	3" PIPE	4" PIPE	6" PIPE
Pneumatic: Jackhammer	90 Lbs.			60			
Pneumatic: Rhino	140 Lbs.			60			
Hydraulic:							
TB-100	135 Lbs.	125 Lbs.	1200	40			
TB-125	400 Lbs.	350 Lbs.	1000	14			
TB-225	650 Lbs.	550 Lbs.	1000	8	12		
TB-325	850 Lbs.	850 Lbs.	900	6	10	16	
TB-425	1100 Lbs.	1100 Lbs.	900		6	10	20
TB-725	2700 Lbs.	2000 Lbs.	600			4-5	10 or 12
TB-830	3000 Lbs.	3000 Lbs.	500				6

PIPE SIZE	O.D.	WALL	HAMMER SIZE	TYPICAL CAPACITY (assumes 2x safety factor)
2"	2.375"	.218" sch. 80	90, 140, 400 Lbs.	4 – 6 KPS
3"	3.500"	.216" sch. 40	650 or 850 Lbs.	12 KPS
4"	4.500"	.237" sch. 40	850 or 1100 Lbs.	20 KPS
6"	6.625"	.280" sch. 40	2000 or 3000 Lbs.	30 KPS

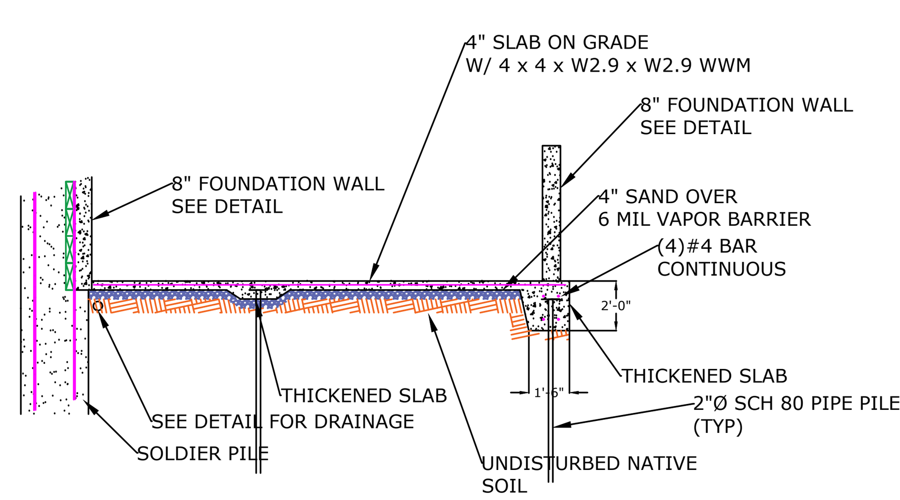
PIPE PILE INFO



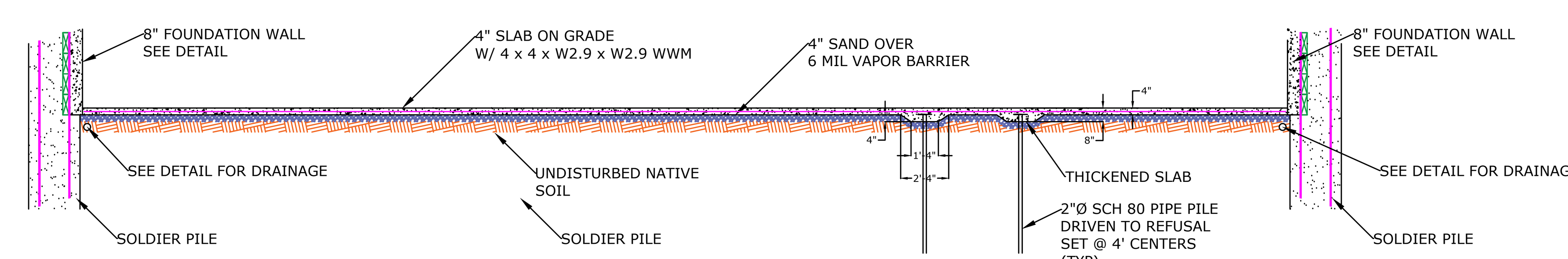
BASEMENT PLAN



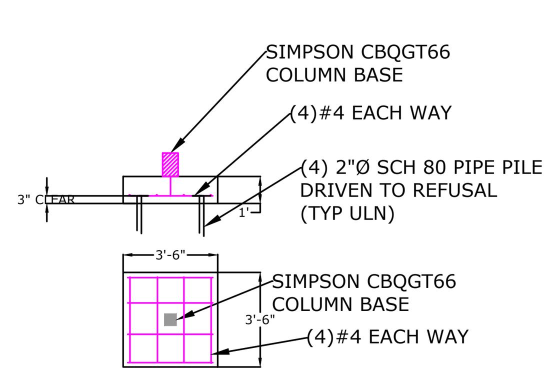
FOUNDATION PLAN VIEW



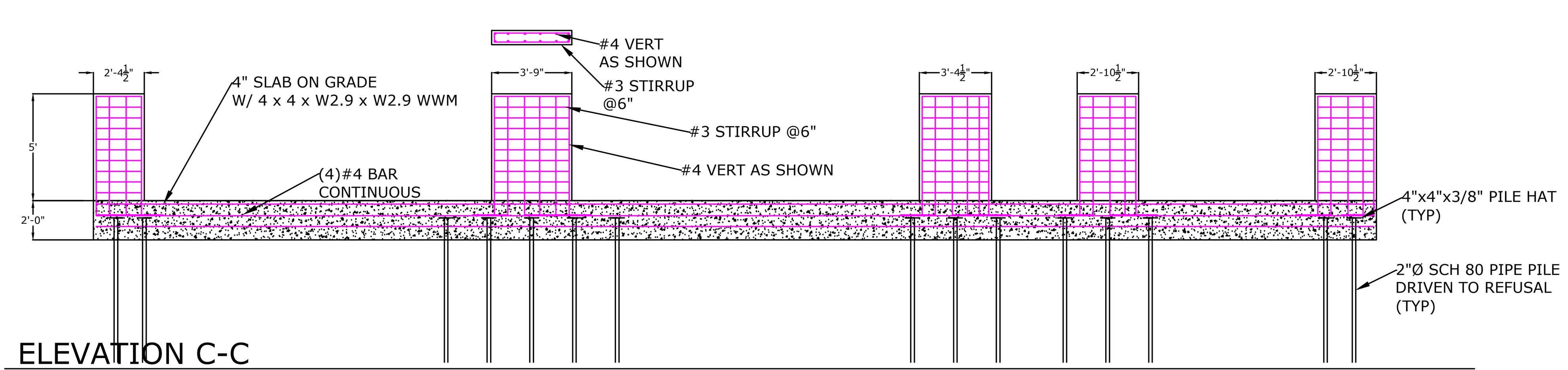
SECTION B-B



SECTION A-A



SPREAD FTNG DETAIL



ELEVATION C-C

DUFRENSE RESIDENCE
11143 Rolling Bay Walk NE
Bainbridge Island, WA

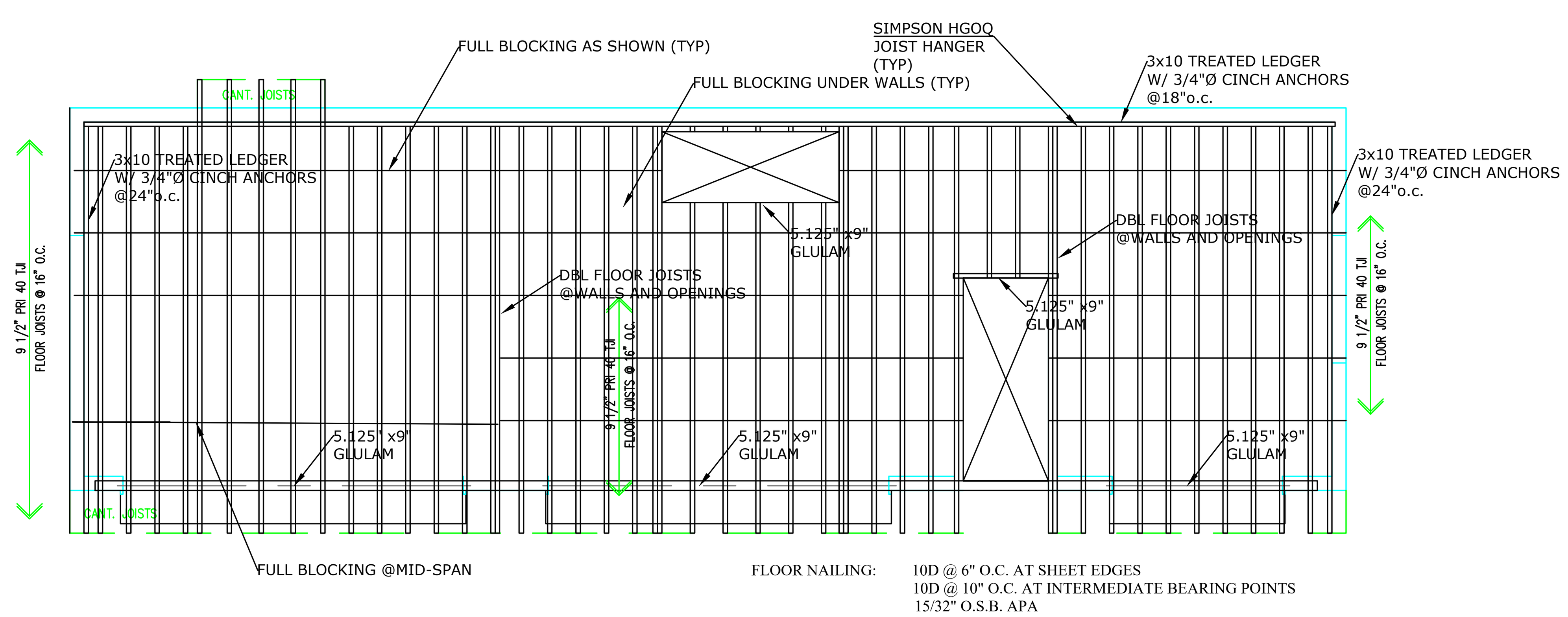
Soldier Pile & Timber Lagging Retaining Wall

LUCIA ENGINEERING, INC.
7307 12th Avenue N.E.
Seattle, Washington 98115
PHONE: (206) 790-8039
E-MAIL: joe@luciaeng.com

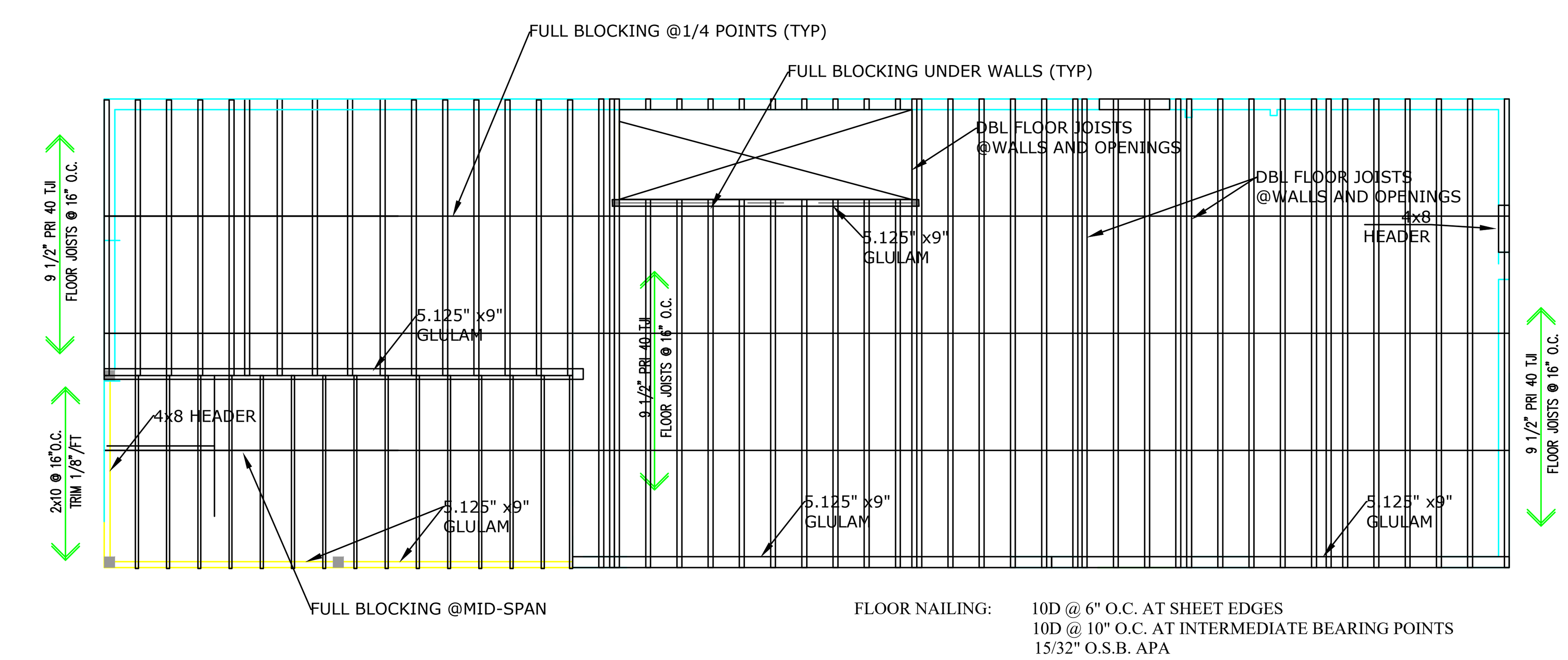
JOSEPH M. LUCIA
STATE OF WASHINGTON
23314
REGISTERED PROFESSIONAL ENGINEER
11-15-17

Number	Date	By	Description
2	11-15-17		Bainbridge Island, WA

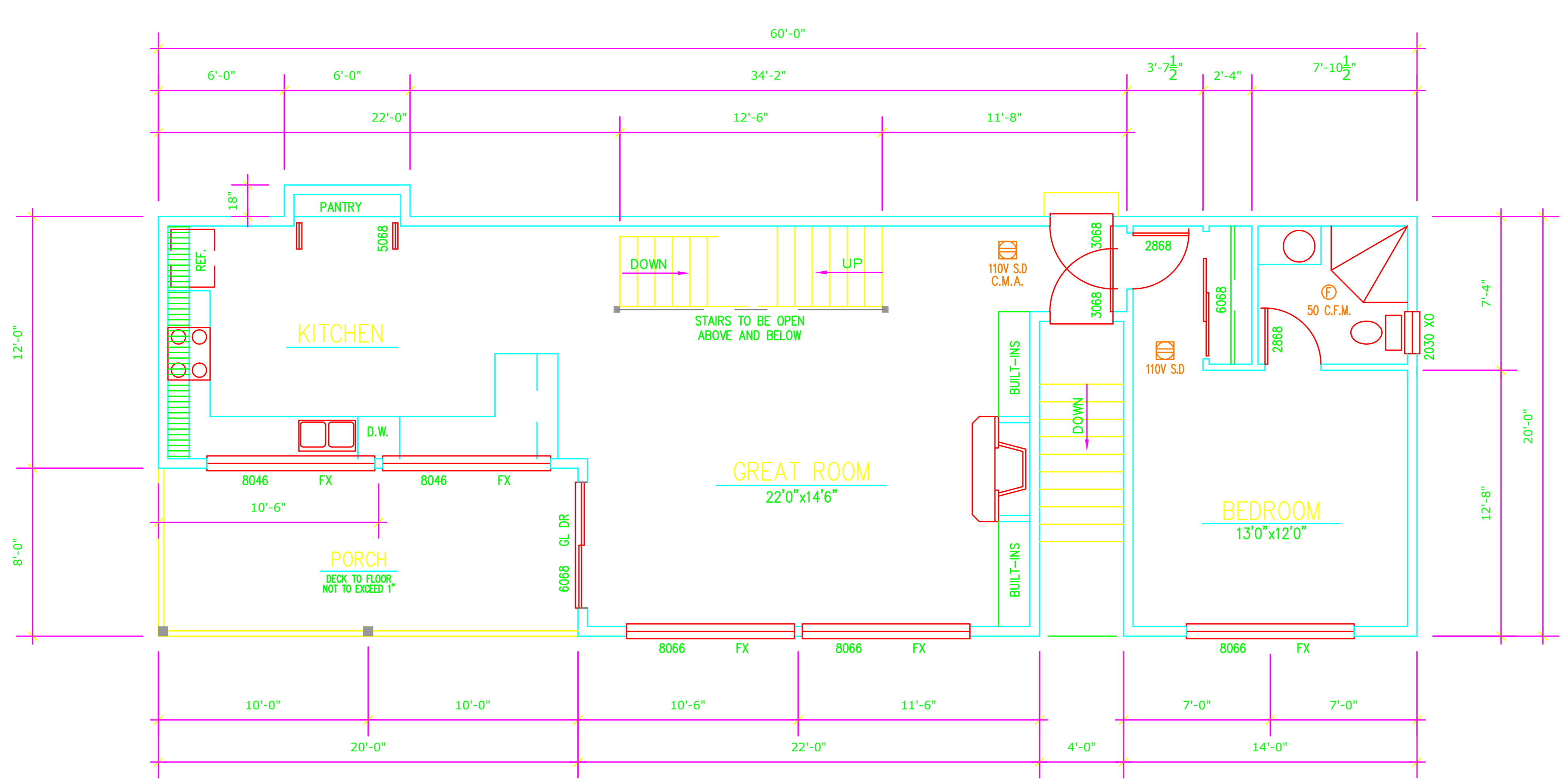
SHEET
SH-10.0



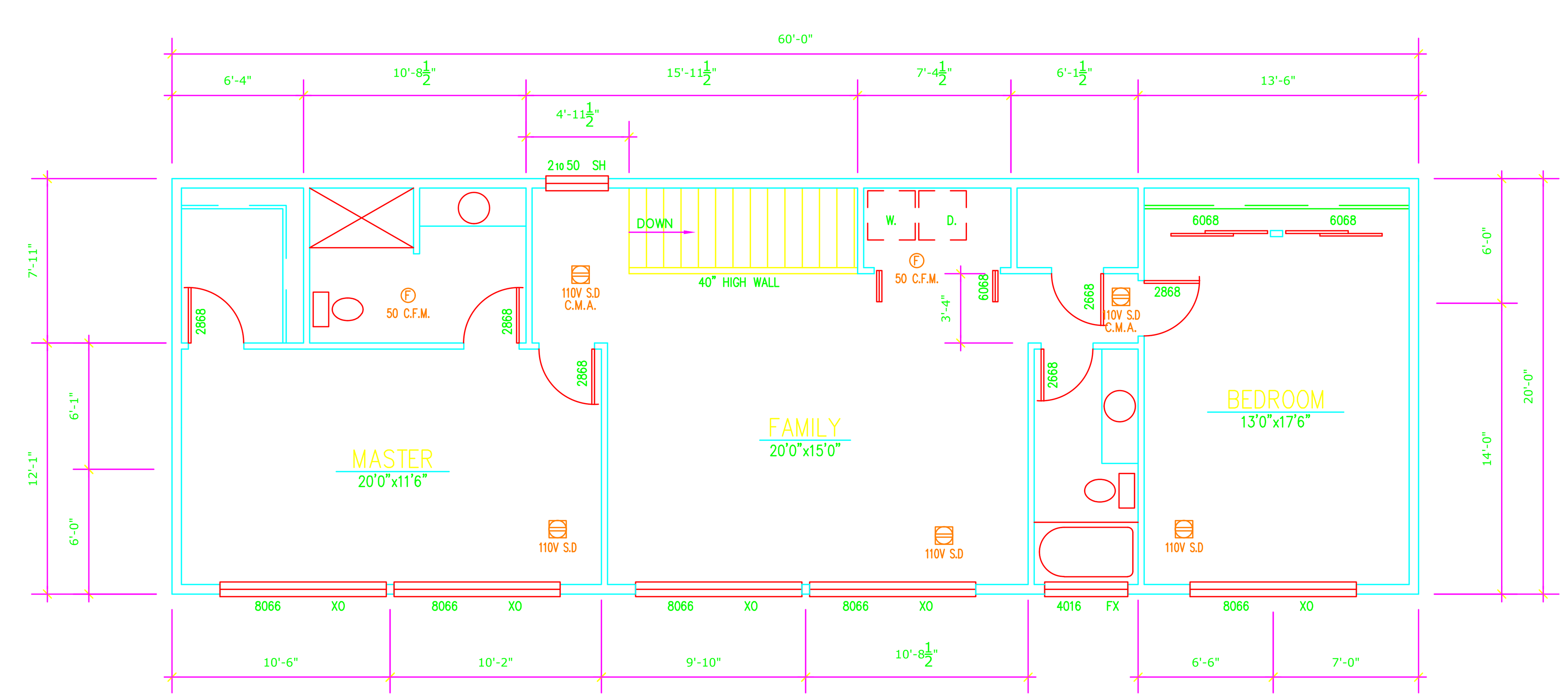
MAIN FLOOR FRAMING PLAN



UPPER FLOOR FRAMING PLAN



MAIN FLOOR PLAN



UPPER FLOOR PLAN

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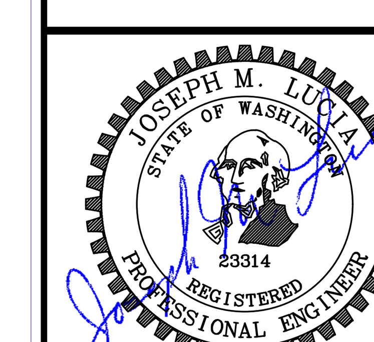

11-15-17

Number	Date	By	Description
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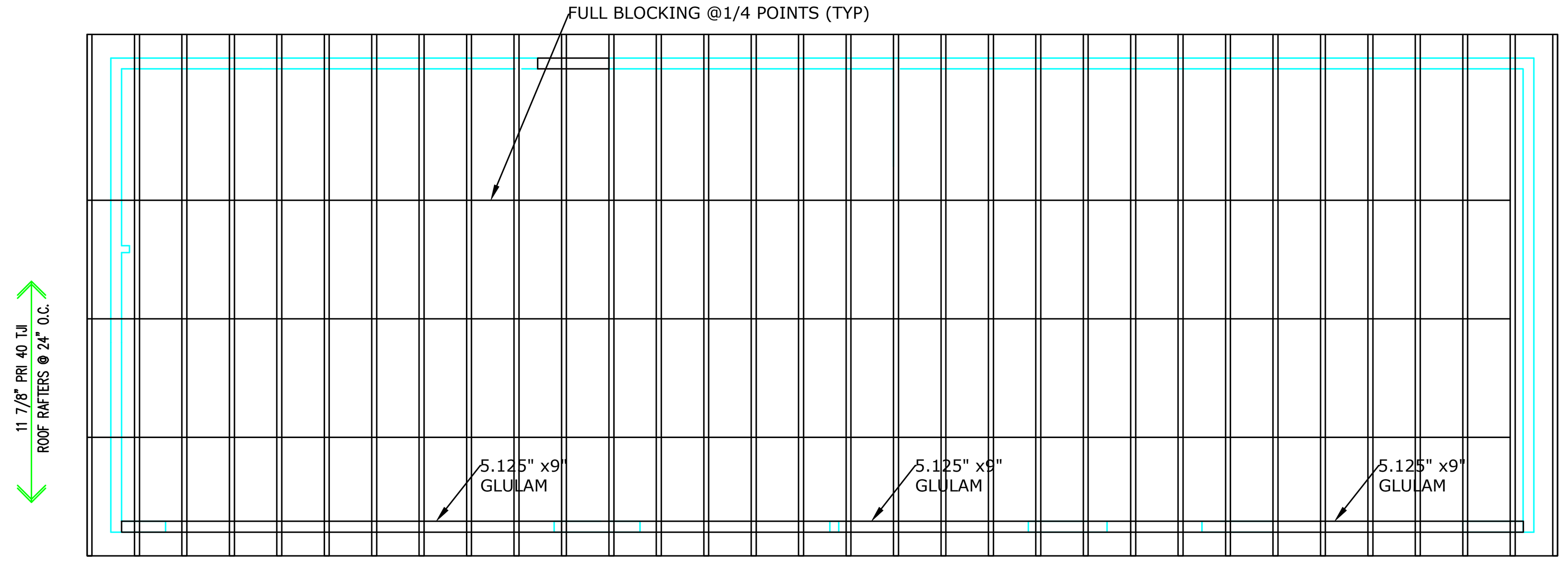
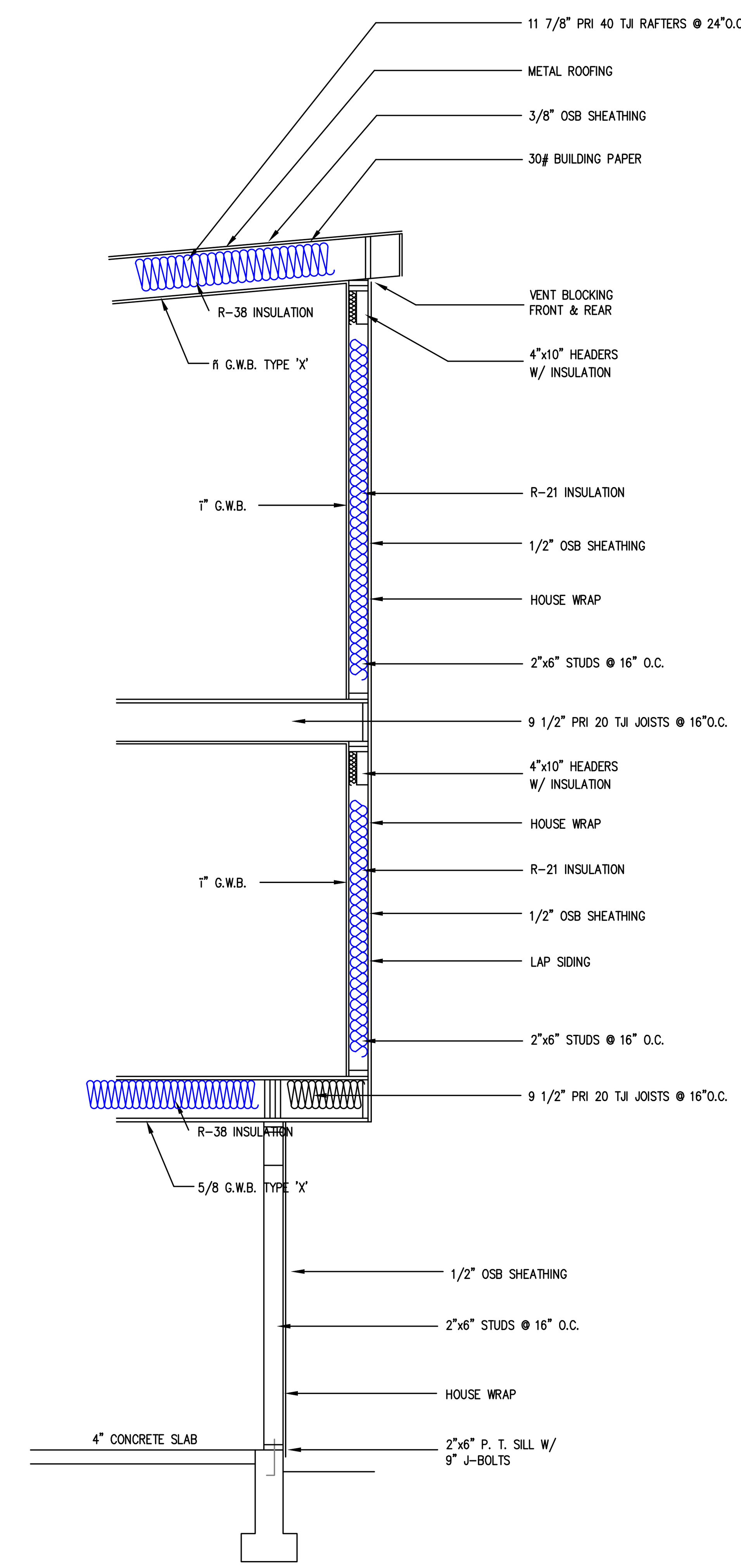
SHEET
SH-11.0

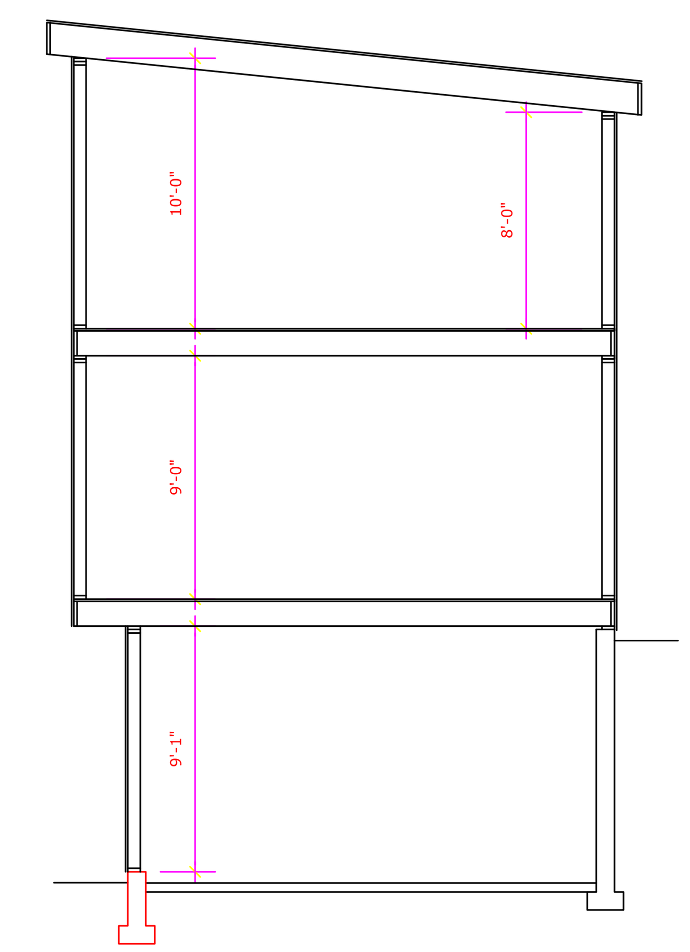


FLOOR NAILING: 8D @ 6" O.C. AT SHEET EDGES
8D @ 10" O.C. AT INTERMEDIATE BEARING POINTS
3/8" O.S.B. APA

ROOF FRAMING PLAN



CROSSECTION



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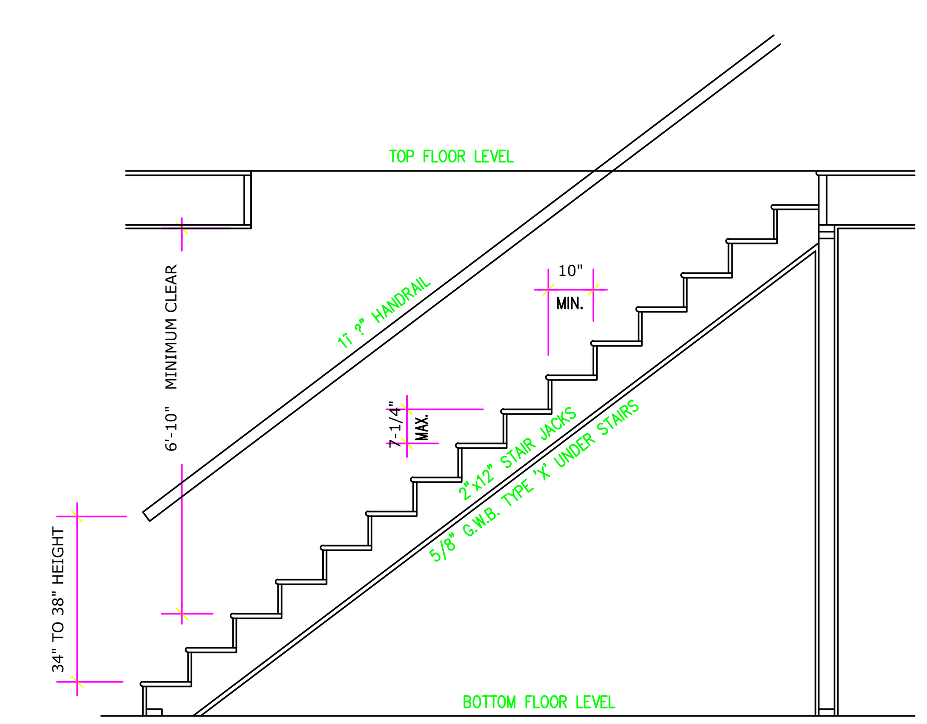

STAIR SECTION

DUFRENSE RESIDENCE
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Soldier Pile & Timber Lagging Retaining Wall

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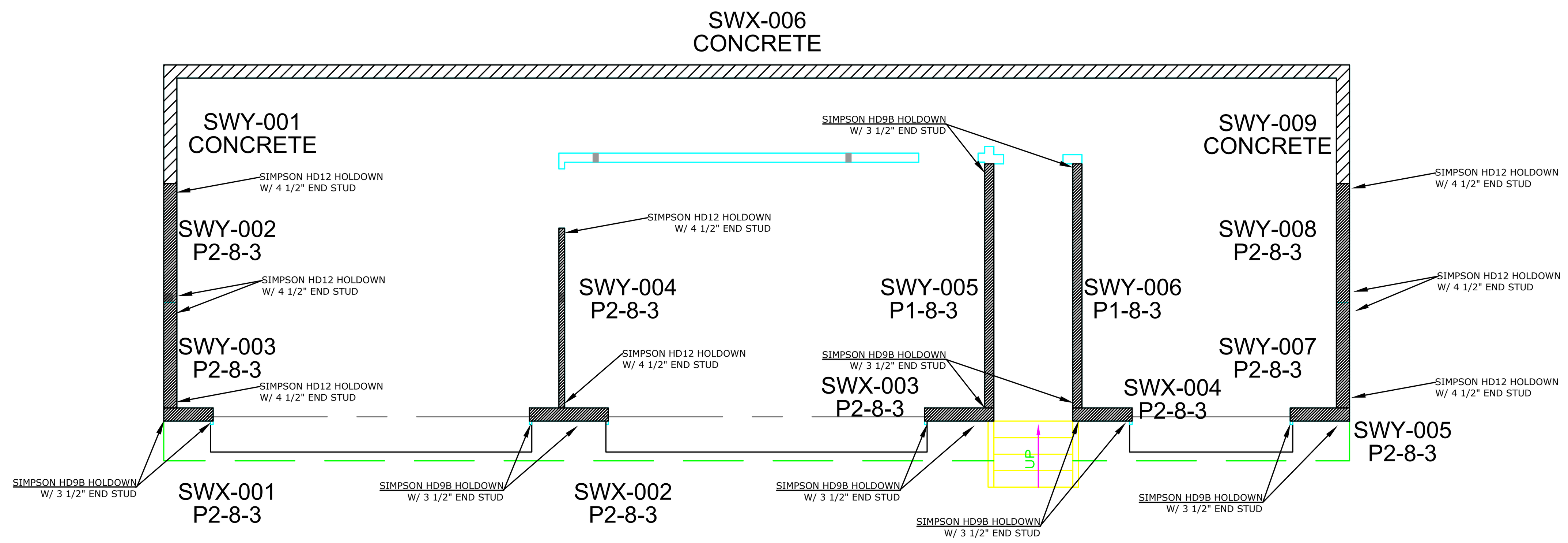
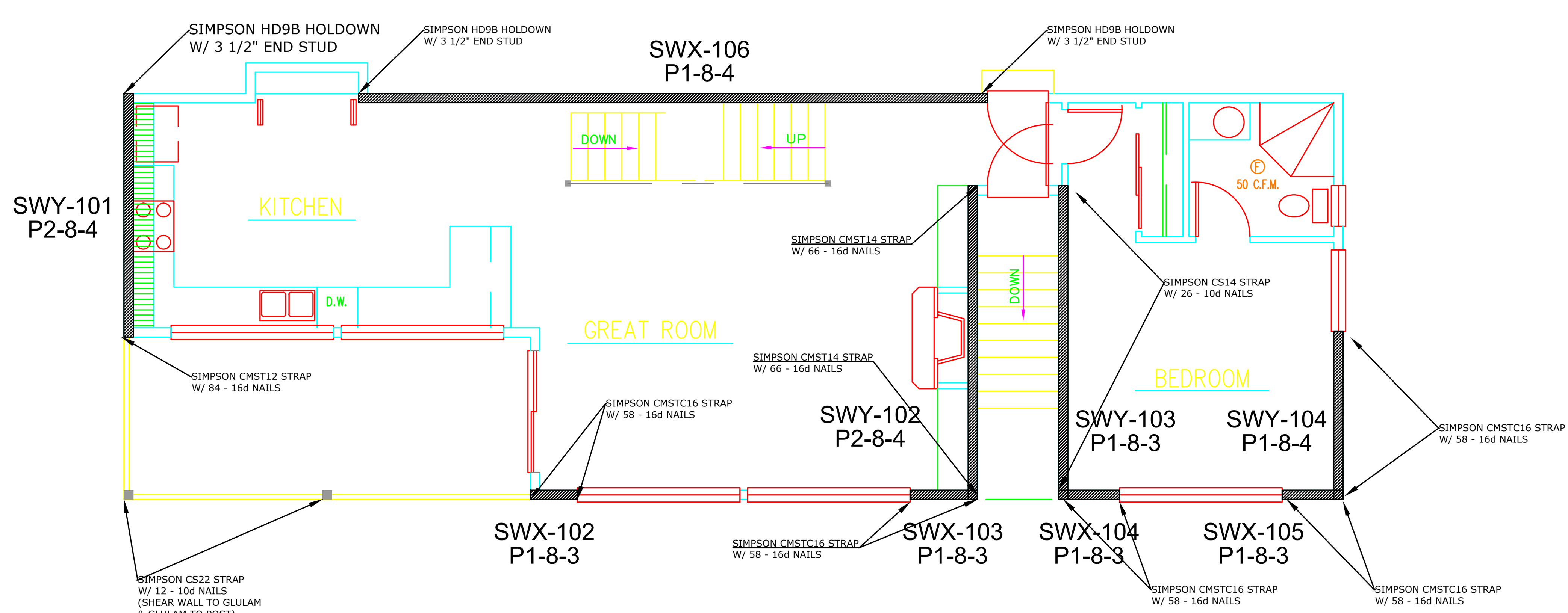
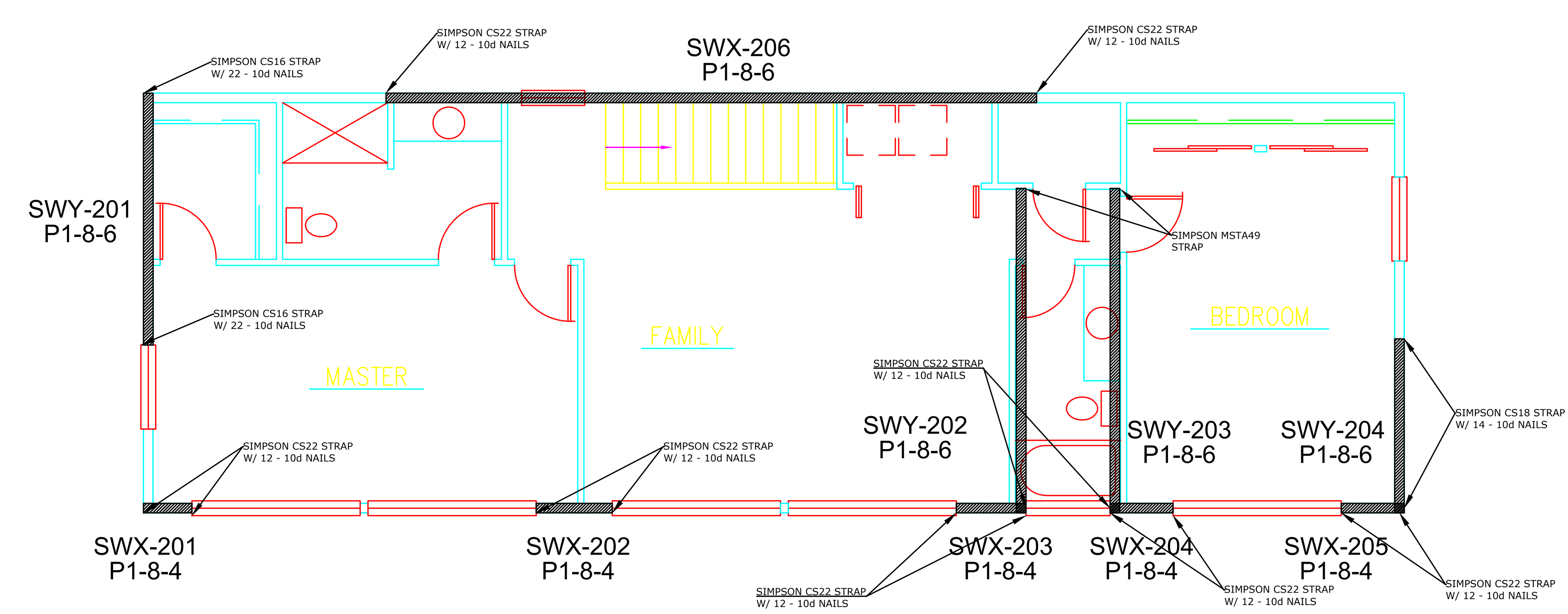
SHEET
SH-12.0



BASEMENT LEVEL SHEAR WALLS



MAIN FLOOR LEVEL SHEAR WALLS



UPPER FLOOR LEVEL SHEAR WALLS

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11-15-17

Number	Date	By	Description
2	11-15-17		Bainbridge Island, WA

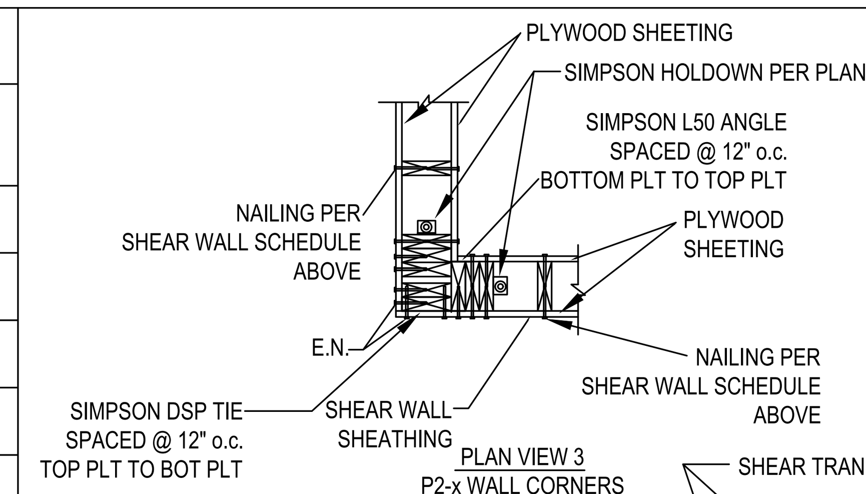
SHEET
SH-13.0

SHEAR WALL SCHEDULE

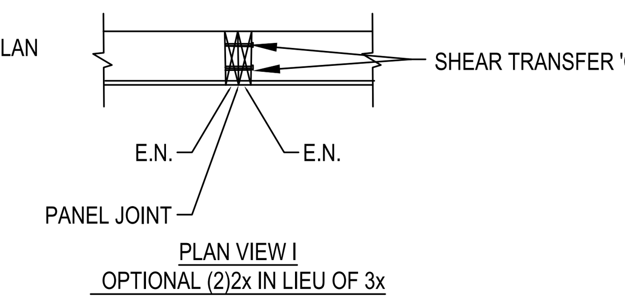
MARK	SHEATHING	NAILING (5)		LUMBER			SHEAR TRANSFER					1.4 INCREASE FOR WIND
		EDGE (E.N.)	FIELD	ALLOWABLE SHEAR	SILL PL	TOP PL'S	"A" SILL PL TO CONC.	"B" BLKG TO TOP PL.	"C" SILL PL RIM/JST/BLKG (F.N.)	"D" SHEAR WALL INTERSECTIONS	CAPACITY	CAPACITY
P1-8-6	15/32" APA RATED SHEATHING, ONE SIDE	8d@6"	8d@12"	2x	2x	(2)2x	5/8 @ 48"	A35@20" OR LPT4 @ 30"	16d @ 5"	16d @ 8"	240 PLF	336 PLF
P1-8-4	15/32" APA RATED SHEATHING, ONE SIDE	8d@5"	8d@12"	2x	2x	(2)2x	5/8 @ 40"	A35@16" OR LPT4 @ 20"	16d @ 5"	16d @ 5"	350 PLF	490 PLF
P1-8-3	15/32" APA RATED SHEATHING, ONE SIDE	8d@4"	8d@12"	2x	3x	(2)2x	5/8 @ 36"	A35@12" OR LPT4 @ 15"	20d @ 4"	16d @ 3 1/2"	450 PLF	360 PLF
P1-8-2	15/32" APA RATED SHEATHING, ONE SIDE	8d@3"	8d@12"	3x(9)	3x	(2)2x	5/8 @ 24"	A35@9" OR LPT4 @ 11"	20d @ 3"	1/2" x4 1/2" LAG @ 9"	585 PLF	819 PLF
P2-8-4	15/32" APA RATED SHEATHING, TWO SIDE	8d@2"	8d@12"	3x(9)	3x	(2)2x	5/8 @ 12"	LPT4 @ 9"	(2)ROWS 20d @ 3"	1/2" x4 1/2" LAG @ 6"	700 PLF	980 PLF
P2-8-3	15/32" APA RATED SHEATHING, TWO SIDE	8d@2"	8d@12"	3x(9)	3x	(2)2x	5/8 @ 12"	LPT4 @ 7"	(2)ROWS 20d @ 3"	1/2" x4 1/2" LAG @ 5"	900 PLF	1260 PLF
P2-8-2	15/32" APA RATED SHEATHING, TWO SIDE	8d@2"	8d@12"	3x(9)	3x	(2)2x	5/8 @ 12"	LPT4 @ 6"	(2)ROWS 20d @ 3"	1/2" x4 1/2" LAG @ 4 1/2"	1200 PLF	1638 PLF

ROOF & FLOOR DIAPHRAGM NAILING SCHEDULE

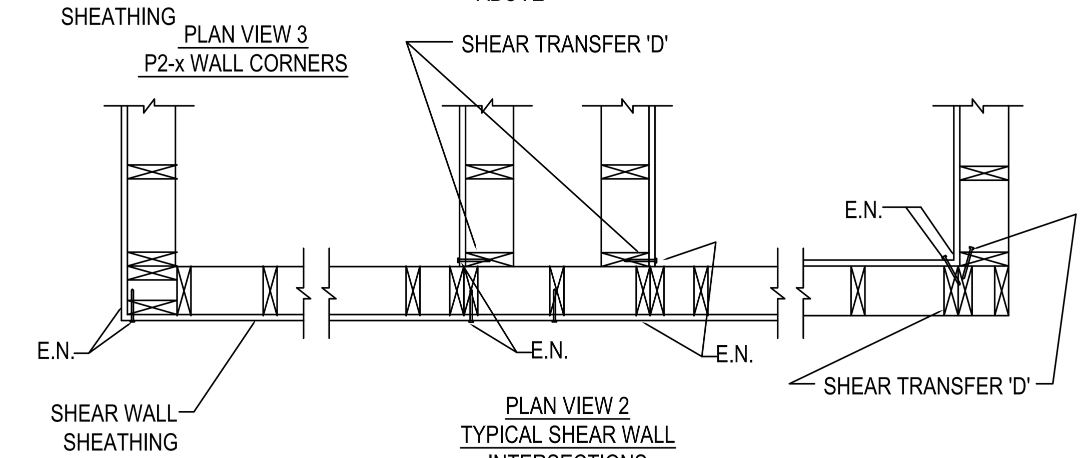
DIA. #	DIAPHRAGM SHEATHING	NAILING (INCHES o.c.) 19/32" SHEATHING W/ 10d COMMON			
		EDGE (E.N.)	FIELD	ALLOWABLE SHEAR (KLF)	NOTES
	UNBLOCKED, OTHER		6	0.21	2x
	UNBLOCKED CASE#1		6	0.28	2x
1	BLOCKED	6	6	0.32	3x
2	BLOCKED	4	6	0.42	3x
3	BLOCKED	2.5	4	0.64	3x
4	BLOCKED	2	3	0.73	3x
5	BLOCKED	2	3	0.82	3x



PLAN VIEW 3
P2-x WALL CORNERS



PLAN VIEW I
OPTIONAL (2)2x IN LIEU OF 3x



PLAN VIEW 2
TYPICAL SHEAR WALL INTERSECTIONS

DIAPHRAGM NOTES:

1. APA RATED SHEATHING, STURD-I-FLOOR EXP1/EXP2/EXT OR C-C/C-D PLYWOOD
2. STRUCTURAL 1 APA RATED SHEATHINGEXP1/EXT OR STRUCT 1 PLYWOOD
3. PROVIDE 3x'S (76mm) AT ADJOINING PANEL EDGES W/NAILS STAGGERED.
4. ALL MEMBERS TO BE 4x MINIMUM W/2 LINES OF FASTENERS (ICBO ER 1952)
5. ALL MEMBERS TO BE 4x MINIMUM W/3 LINES OF FASTENERS (ICBO ER 1952)
6. SPECIAL INSPECTION REQUIRED IN ACCORDANCE WITH ICBO ER 1952
7. PROVIDE BOUNDARY NAILING @ ALL PANEL EDGES, CASES 3,4,5 & 6.
8. ALL MEMBERS TO BE 3x (76mm) MINIMUM.

SHEAR WALL FRAMING NOTES:

1. IN ADDITION TO THE TYPICAL WALL FRAMING REQUIREMENTS PROVIDE FRAMING AT SHEAR WALLS AS INDICATED.
2. SEE SCHEDULE FOR SHEATHING AND NAILING REQUIRED. SCHEDULE ASSUMES HEM-FIR OR BETTER LUMBER. STAGGER PANEL JOINTS EACH SIDE OF WALL WHERE SHEATHING IS REQUIRED BOTH SIDE OF WALL.
3. STUD BLOCKING THICKNESS SHOWN ARE MINIMUM SIZES BASED ON SHEAR WALL NAILING REQUIREMENT. PROVIDE LARGER STUD WHERE REQUIRED OTHERWISE.
4. BLOCK ALL PANEL EDGES.
5. 10d SHALL BE 0.148x3". 8d SHALL BE 0.131X2 ½". DRIVE ALL NAILS FLUSH WITH THE FACE OF . TOLERANCE IS +1/16 to -0
6. PLATES ON CONCRETE SHALL BE TREATED. SEE GENERAL STRUCTURAL NOTES.
7. NAIL OR LAG SHEATHING & STUD AT SHEAR WALL INTERSECTION AS INDICATED.
8. WHERE ONLY ONE HOLDOWN IS SPECIFIED LOCATE ON OPENING SIDE OF HOLDOWN STUDS. SEE WALL ELEVATION AT RIGHT.
9. (2)2x MAY BE USED IN LIEU OF 3x AT PANEL JOINTS. STITCH NAIL THE STUDS TOGETHER PER SHEAR TRANSFER 'C'. SEE 'PLAN VIEW 1'. REFER TO APA TECHNICAL PUBLICATION TT-076.

TYPICAL WALL FRAMING NOTES:

1. PROVIDE TYPICAL WALL FRAMING INDICATED, EXCEPT WHERE NOTED OTHERWISE.
2. SEE ARCHITECTURAL DRAWINGS FOR FIRE BLOCKING AND BACKING FOR FINISHES AND FURNISHINGS.

TYPICAL ROOF & FLOOR DIAPHRAGM FRAMING NOTES:

1. ROOF AND FLOOR DIAPHRAGMS ARE UNBLOCKED, U.L.N. AND NAILED ACCORDING TO THE FASTENING SCHEDULE OF IBC TABLE 2304.9.1.

HEADER END NAILING

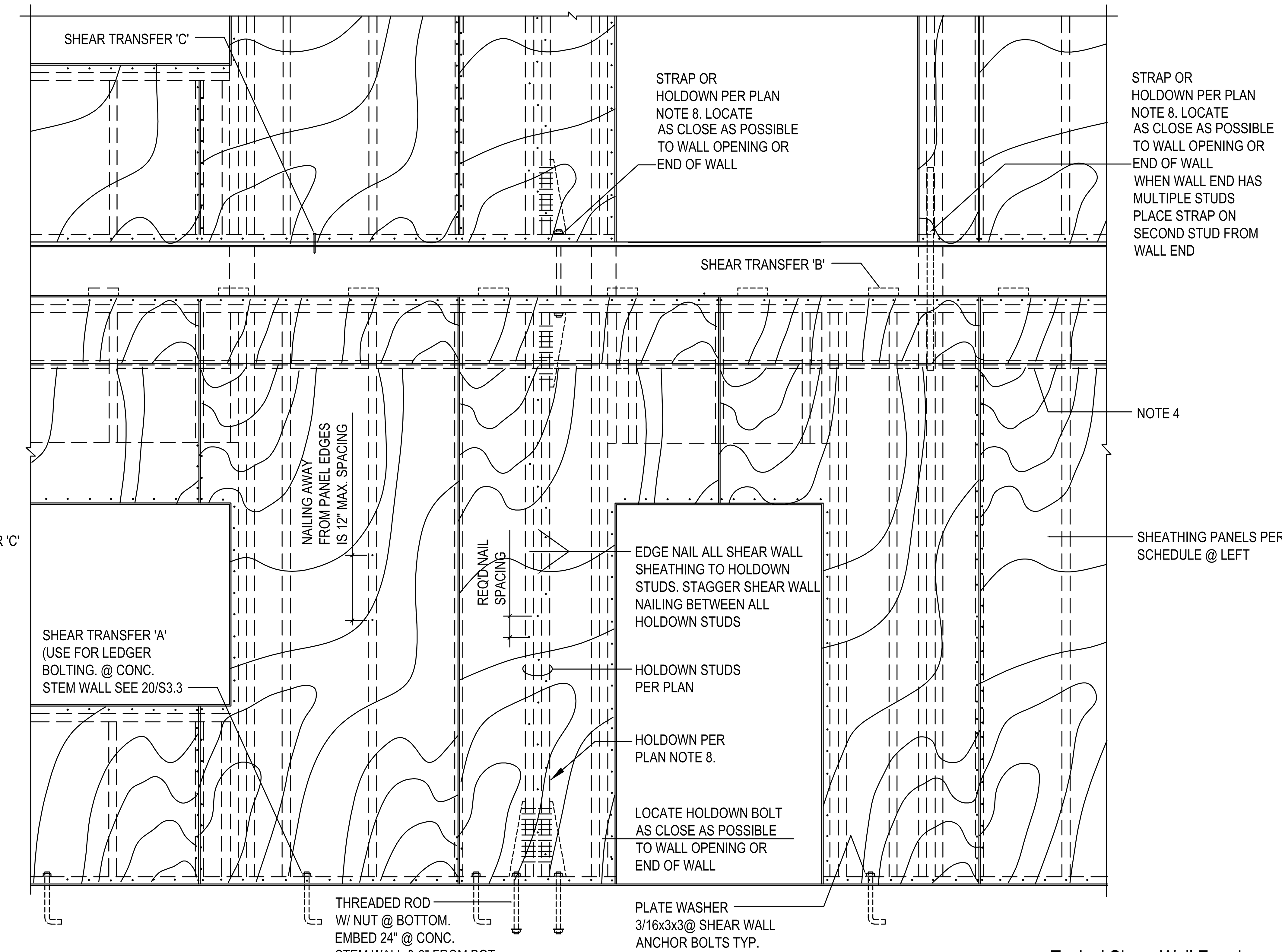
NOMINAL DEPTH	END ATTACHMENT
4	(4)16d
6	(6)16d
8	(8)16d
10	(10)16d
12	(12)16d
14	(14)16d
16	(16)16d
18	(18)16d

ROUGH WINDOW SILL

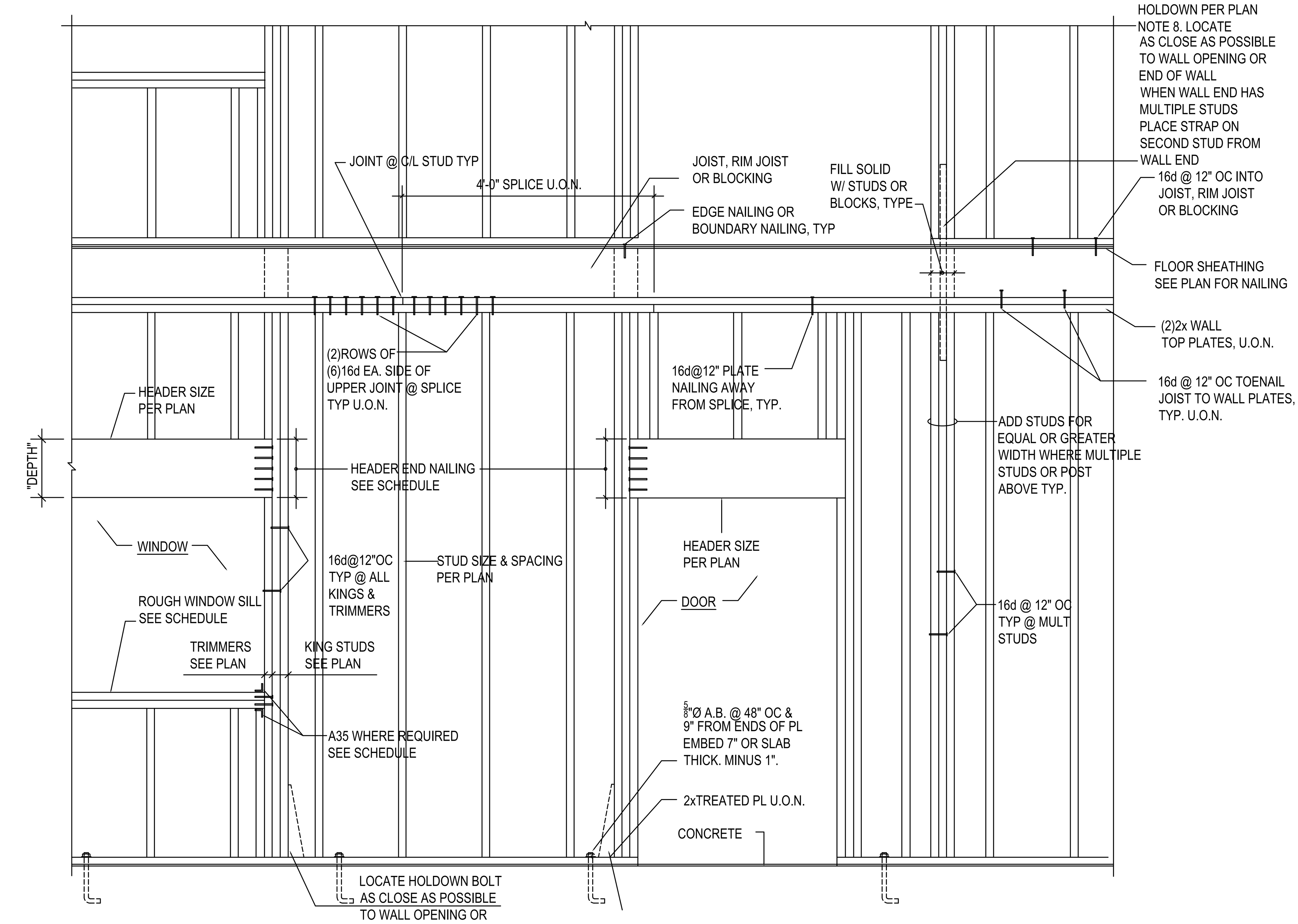
HORIZ ROUGH OPENING	NUMBER OF SILLS REQUIRED	END ATTACHMENT	REF.
0 TO 6'	1	(2)16d END NAIL	20/S6.1
> 6'	2	(2)16d END NAIL, +A35 EA END @ EA SILL	20/S6.1

MINIMUM NAILING SCHEDULE

CONNECTION	NAILS
1. Joist to sill or girder, toenail	(3) 8d
2. Bridging to joist, toenail each end	(2) 8d
3. 1" x 6" sub floor or less to each joist, face nail	(2) 8d
4. Wider than 1"x6" sub floor to each joist, face nail	(3)8d
5. 2" subfloor to joist or girder, blind and face nail	(2)16d
6. Sole plate to joist or blocking, typical face nail	16d at 16" o.c.
Sole plate to joist or blocking, at braced wall panels	(3)16d per 16"
7. Top plates to stud, end nail	(4)16d
8. Stud to sole plate	(4)8d, toenail or (2) 16d, end nail
9. Double stud, face nail	16d at 24" o.c.
10. Double top plates, typical face nail	16d at 16" o.c.
Double top plates, lap splice	(8)16d
11. Blocking between joist or rafters to top plate, toenail	(3)8d
12. Rim joist to top plate, toenail	8d at 6" o.c.
13. Top plates, laps and intersections, face nail	(2)16d
14. Continuous header, two pieces	16d at 16" o.c. along each edge
15. Ceiling joist to plate, toenail	(3)8d
16. Continuous header to studs, toenail	(4)8d
17. Ceiling joist, lap over partitions face nail	(3)16d
18. Ceiling joist to parallel rafters, face nail	(3)16d
19. Rafter to plate, toenail	(3)8d
20. 1" brace to each stud and plate, face nail	(2)8d
21. 1"X8" sheathing or less to each bearing , face nail	(2)8d
22. Wider than 1"x8" sheathing to each bearing face nail	(5)8d
23. Built up corner studs	16d at 24" o.c.
24. Built up girder and beams	



Typical Shear Wall Framing



Typical Wall Framing
Scale: ⅜"=1'-0, U.O.N.

DUFRENSE RESIDENCE
11143 Rolling Bay Walk NE
Bainbridge Island, WA

Soldier Pile & Timber Lagging Retaining Wall

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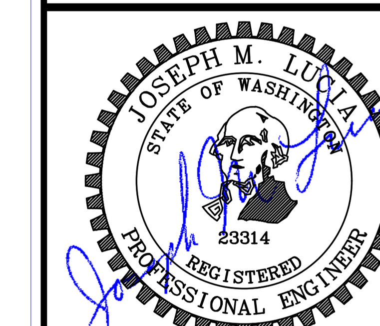


11-15-17

Number	Date	By	Description
2	11-15-17		Bainbridge Island, WA

SHEET
SH-14.0

#B109 STORMWATER MANAGEMENT

For City Use Only:
Date Stamp

Applicant's Name: MARGARET DUFRESNE Address: 3912 NE STATE HIGHWAY 104 POULSBO, WA 98370
Applicant Phone #:______________________ e-mail: ______________________
Site Assessor Tax Parcel #: 20055-03-24-0119
Site Address: 11143 Rolling Bay Walk NE Bainbridge Island, WA 98110

All information in this worksheet is required to be filled out for your permit application to be accepted.

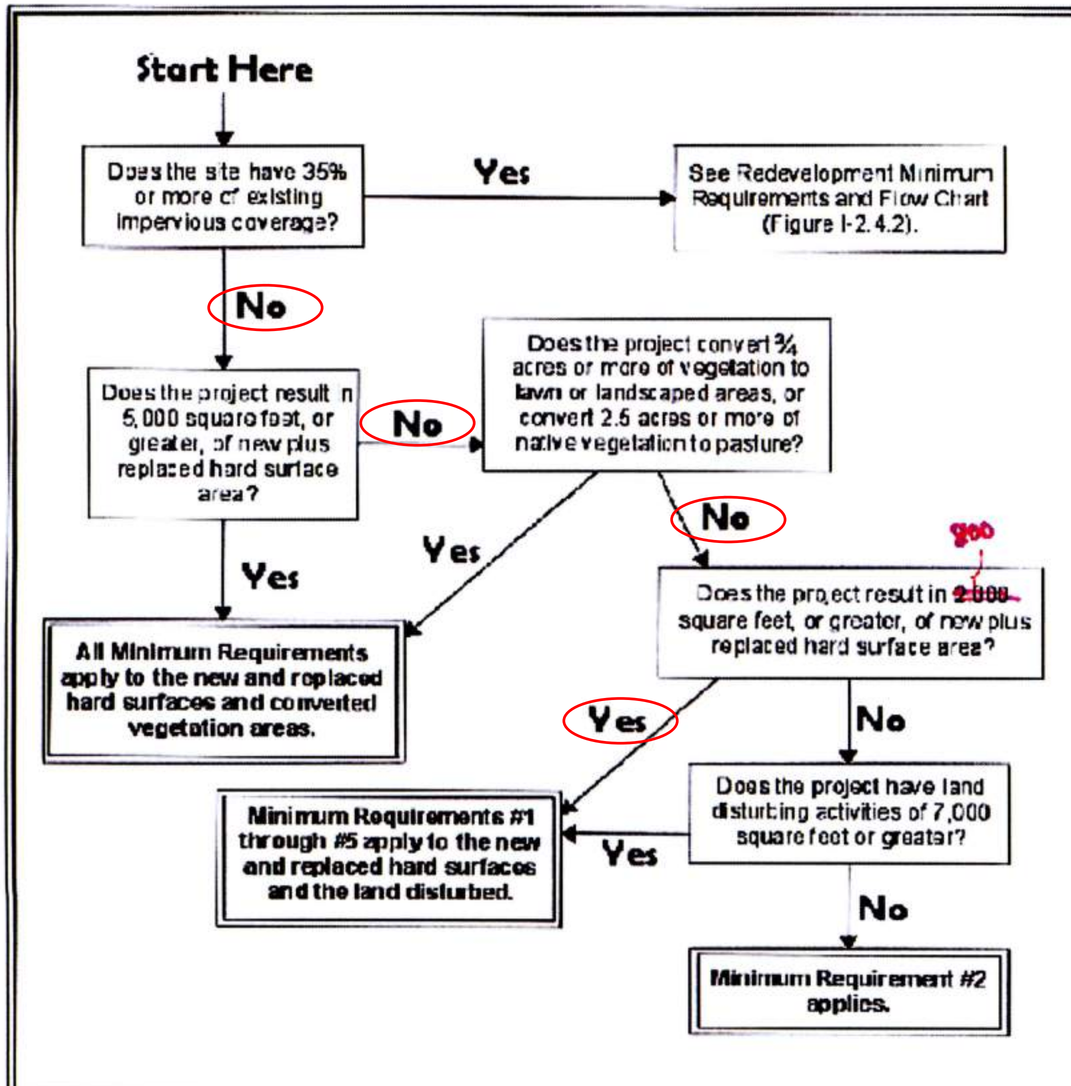
Section 1 General Information

1. Existing Site Conditions: SITE WAS DEVELOPED WITH A SINGLE FAMILY RESIDENCE UNTIL 1997. SITE IS VEGETATED WITH GRASS, A FEW SMALL ALDERS, AND BLACKBERRY BUSHES.
2. Proposed Site Development Activity: PROPOSED SITE DEVELOPMENT INCLUDES CONSTRUCTION OF A SINGLE FAMILY RESIDENCE, DRIVEWAY, AND ATTACHED GARAGE.
3. Total Size of property: 0.44 ACRES
4. Existing hard coverage on the site (%): 8%
5. Proposed (new + replaced) hard surface area on site: 1660 square feet.
6. Total proposed land disturbance area: 5,800 square feet.
7. Area converted from native vegetation to lawn, landscaping or pasture: 0 square feet.
8. Water Purveyor (if applicable):______________________
9. Sanitary Sewer Purveyor (if applicable): N/A
10. Adjacent or onsite water bodies: ___pond ___wetland ___stream/creek X shoreline

Review flow charts attached and determine what Minimum Requirements apply to your project?

Minimum Requirements:

- ☐ **#1-#9 go to Section 2**
- ☒ **#1-#5 go to Section 2**
- ☐ #2 **go to Section 4**



DEPARTMENT OF ECOLOGY
State of Washington

Figure I-2.4.1
Flow Chart for Determining Requirements for New Development

Revised June 2015

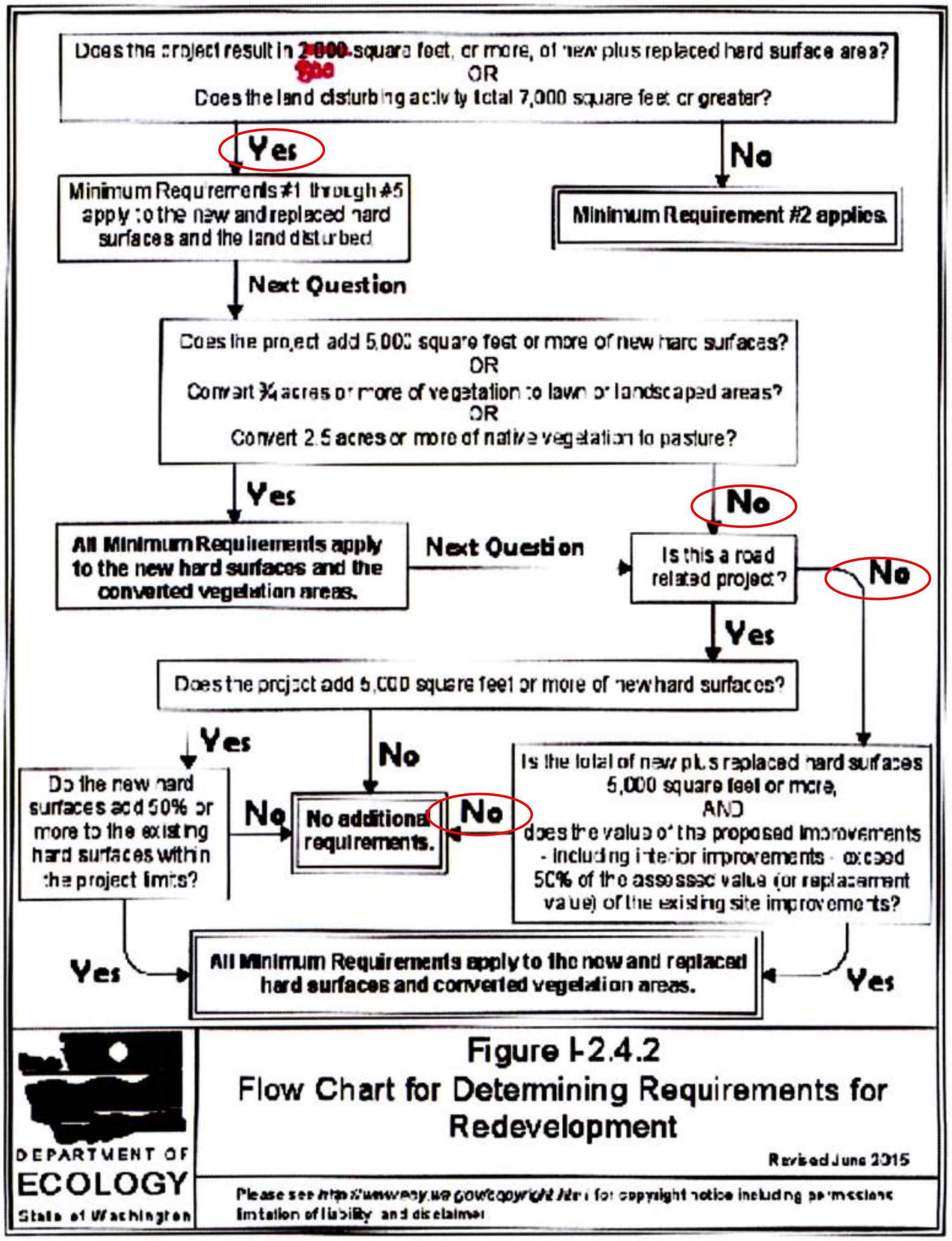


Figure I-2.4.2
Flow Chart for Determining Requirements for Redevelopment

Revised June 2015

3 | Page

Section 2 – Site Assessment

Site assessment shall follow the steps outlined in the "*2012 Low Impact Development Technical Guidance Manual for Puget Sound*"

Surveyor (Registered land surveyor required): ____________________

Soil Report Prepared by: PERRONE CONSULTING, INC.

Certification: ____________________

Native Vegetation and Soil Plan Prepared by: ____________________

Certification: ____________________

Preliminary Drainage Report Prepared by: BROWNE WHEELER ENGINEERS

Certification: PE

Submittals

This submittal checklist is intended to assist you in preparing and submitting a complete application. Once your application is determined to be counter complete, a review for technical completeness is conducted and you may be required to submit additional information in order to proceed with further review of your application.

Submittal Requirements

Use the column to the left to check off items included with your application. More detailed submittal descriptions are provided on the following pages of this document.

✓	Required Submittal Items	Number
	1. Surveyed Existing Site Plan	2 original paper
✓	2. Soils Report	2 original paper
	3. Native Vegetation and Soil Protection Area Plan	2 original paper
✓	4. Drainage Report	2 original paper
✓	5. Site Plan	2 original paper
	6. Other technical repots as applicable, including but not limited to: ○ Geotechnical report ○ Wetlands delineation report and mitigation plan ○ Other ____________	3 original paper

Site Assessment/Analysis Requirements

Detailed application requirements are noted below; full details are not provided due to limited space. Please note that additional items or information may be required if the review process indicates more information is needed to evaluate the project. Follow and submit in accordance with "*Low Impact Development (LID) Technical Guidance Manual for Puget Sound*", Chapter 2 Site Assessment.

Survey Site Plan Requirements:

- ☐ Project datum and two project benchmarks identified.
- ☐ Scale
- ☐ Existing topography, including existing structures, for the site and extending 50 feet beyond project boundaries. Existing topography for adjacent rights-of-way must be included for the full width of right-of-way. Contours as follows:
    - Up to 10 percent slopes, two-foot contours.
    - Over 10 percent to less than 20 percent slopes, 5-foot contours.
    - 20 percent or greater slopes, 10-foot contours.
    - Elevations shall be at 25-foot intervals.
- ☐ Property lines, right-of-way and easements are clearly identified.
- ☐ Existing public and private development, including utility infrastructure on and adjacent (if publicly available) to the site.
- ☐ Major hydrologic features with streams, wetland, and water body survey and classification report showing wetland and buffer boundaries consistent with COBI requirements.
- ☐ Flood hazard areas on or adjacent to the site, if present.
- ☐ Geologic hazard areas and associated buffer requirements.
- ☐ Aquifer and wellhead protection areas on or adjacent to the site, if present.
- ☐ Topographic features that may act as natural stormwater storage, infiltration or conveyance.

Soils Report:

- ☐ Soil Report prepared by a certified soil scientist, professional engineer, geologist, hydrogeologist or engineering geologist registered in the State of Washington or suitably trained persons working under the supervision of the above professionals. The report will identify:
    - Underlying soil texture and stratigraphy on the site. Tests for accessing and assessing on-site soil texture and stratigraphy include soil surveys, soil test pits, small-scale Pit Infiltration Test (PIT) or soil borings. Grain size analysis may be substituted for infiltration tests on soils unconsolidated by glacial advance.
    - Determine if depth to hydraulic restriction layer under rain gardens or permeable pavement is within one foot of the bottom (subgrade surface) of the infiltration areas, using a monitoring well or excavated pit. This analysis should be performed in the winter season (December 1 through April 1). The optimum time to test for depth to seasonally high groundwater is late winter (e.g. March) and shortly after an extended wet period. Historic site information and evidence of high groundwater can also be used.
    - **For Sites Required to Meet Minimum Requirements 1-5 per BIMC 15.20.060:** Infiltration rates of on-site soils. Infiltration rates for rain gardens, bioretention areas or permeable pavement installations must be assessed using septic style pit tests, small-scale PIT or grain size analysis (if unconsolidated soils). See *2012 LID Technical Guidance Manual for Puget Sound.*
    - **For Sites Required to Meet Minimum Requirements 1-9 per BIMC 15.20.060:**
        - Saturated hydraulic conductivity (Ksat) of site soils.
        - Detailed logs for each test pit or test hole and a map showing the location of the pits or holes.

- Location of monitoring wells if site assessment cannot confirm that seasonal high groundwater or hydraulic restricting layer is greater than 5 feet below the bottom of the bioretention or permeable pavement.
- Analysis of interflow potential and conveyance.
- Follow *2012 LID Technical Guidance Manual for Puget Sound* for additional requirements.

Native Vegetation or Soil Protection Area:

- ☐ Include a survey of native protection areas proposed for the site, if any. Survey of existing native vegetation cover will be prepared by a licensed landscape architect, arborist, qualified biologist.
- ☐ Identify any forest areas on the site.
- ☐ Provide a plan for protection of the area.

Drainage Report:

- ☐ Proposed plan for permanent stormwater management.
- ☐ Proposed staging to minimize site disturbance and impacts.
- ☐ Proposed stormwater management plan during construction.

Site Plan:

- ☐ Plan sheet size 18"x24" or 24"x36"
- ☐ All items provided in survey site plan.
- ☐ Proposed structure.
- ☐ Proposed utilities.
- ☐ Other proposed hard surfaces (driveway, parking, sidewalks and pathways).
- ☐ Proposed access points.
- ☐ Location of proposed stormwater facilities.

Section 3 -Stormwater Management Requirements

(Underline text corresponds to the 2012 (Rev. 2014) Stormwater Management Manual for Western Washington (SWMMWW))

Projects triggering only Minimum Requirements #1 through #5 shall either:

a. Use On-site Stormwater Management BMPs from List #1 for all surfaces within each type of surface in List #1; or

b. Demonstrate compliance with the LID Performance Standard. Projects selecting this option cannot use Rain Gardens. They may choose to use Bioretention BMPs as described in Chapter V-7 - Infiltration and Bioretention Treatment Facilities to achieve the LID Performance Standard.

Projects triggering Minimum Requirements #1 through #9, must

a. meet the requirements in I-2.5.5 Minimum Requirement #5: On-site Stormwater Management.; and

b. either

    1. Low Impact Development Performance Standard and BMP T5.13: Post-Construction Soil Quality and Depth; or

    2. List #2

Low Impact Development (LID) Performance Standard

Stormwater discharges shall match developed discharge durations to pre-developed durations for the range of pre-developed discharge rates from 8% of the 2-year peak flow to 50% of the 2-year peak flow. Refer to the Standard Flow Control Requirement section in Minimum Requirement #7 for information about the assignment of the pre-developed condition. Project sites that must also meet minimum requirement #7 – flow control - must match flow durations between 8% of the 2-year flow through the full 50-year flow.

List #1: On-site Stormwater Management BMPs for Projects Triggering Minimum Requirements #1 through #5

For each surface, consider the BMP's in the order listed for that type of surface. Use the first BMP that is considered feasible. No other On-site Stormwater Management BMP is necessary for that surface. Feasibility shall be determined by evaluation against:

1. Design criteria, limitations, and infeasibility criteria identified for each BMP in the SWMMWW; and

2. Competing Needs Criteria listed in Chapter V-5 - On-Site Stormwater Management.

Lawn and landscaped areas:

- Post-Construction Soil Quality and Depth in accordance with BMP T5.13: Post-Construction Soil Quality and Depth.

Roofs:

1. Full Dispersion in accordance with BMP T5.30: Full Dispersion, or Downspout Full Infiltration Systems in accordance with BMP T5.10A: Downspout Full Infiltration

2. Rain Gardens in accordance with BMP T5.14A: Rain Gardens, or Bioretention in accordance with BMP T7.30: Bioretention Cells, Swales, and Planter Boxes. The rain garden or bioretention facility must have a minimum horizontal projected surface area below the overflow which is at least 5% of the area draining to it.

3. Downspout Dispersion Systems in accordance with BMP T5.10B: Downspout Dispersion Systems

4. Perforated Stub-out Connections in accordance with BMP T5.10C: Perforated Stub-out Connections

Other Hard Surfaces:

1. Full Dispersion in accordance with BMP T5.30: Full Dispersion

2. Permeable pavement[1] in accordance with BMP T5.15: Permeable Pavements, or Rain Gardens in accordance with BMP T5.14A: Rain Gardens, or Bioretention in accordance with BMP T7.30: Bioretention Cells, Swales, and Planter Boxes. The rain garden or bioretention facility must have a minimum horizontal projected surface area below the overflow which is at least 5% of the area draining to it.

3. Sheet Flow Dispersion in accordance with BMP T5.12: Sheet Flow Dispersion, or Concentrated Flow Dispersion in accordance with BMP T5.11: Concentrated Flow Dispersion.

List #2: On-site Stormwater Management BMPs for Projects Triggering Minimum Requirements #1 through #9

For each surface, consider the BMPs in the order listed for that type of surface. Use the first BMP that is considered feasible. No other On-site Stormwater Management BMP is necessary for that surface. Feasibility shall be determined by evaluation against:

1. Design criteria, limitations, and infeasibility criteria identified for each BMP in this manual; and

2. Competing Needs Criteria listed in Chapter V-5 - On-Site Stormwater Management.

Lawn and landscaped areas:

- Post-Construction Soil Quality and Depth in accordance with BMP T5.13: Post-Construction Soil Quality and Depth.

Roofs:

1. Full Dispersion in accordance with BMP T5.30: Full Dispersion, or Downspout Full Infiltration Systems in accordance with BMP T5.10A: Downspout Full Infiltration.

2. Bioretention (See BMP T7.30: Bioretention Cells, Swales, and Planter Boxes) facilities that have a minimum horizontally projected surface area below the overflow which is at least 5% of the total surface area draining to it.

3. Downspout Dispersion Systems in accordance with BMP T5.10B: Downspout Dispersion Systems

4. Perforated Stub-out Connections in accordance with BMP T5.10C: Perforated Stub-out Connections

Other Hard Surfaces:

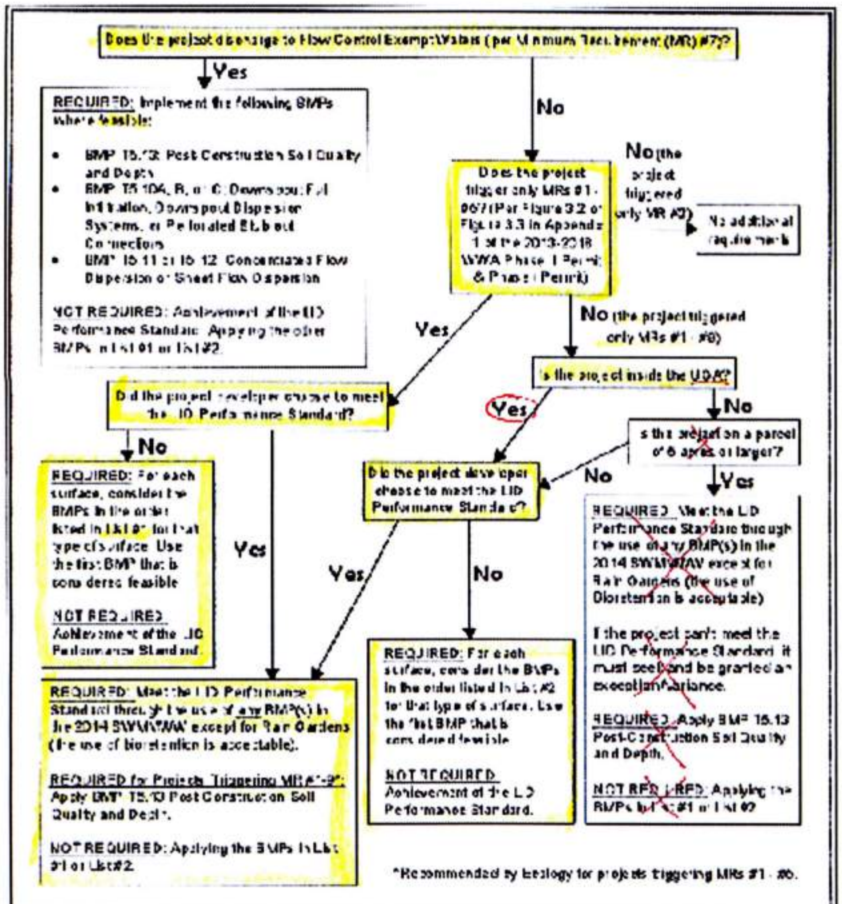
1. Full Dispersion in accordance with BMP T5.30: Full Dispersion

2. Permeable pavement[1] in accordance with BMP T5.15: Permeable Pavements

3. Bioretention BMP's (BMP T7.30: Bioretention Cells, Swales, and Planter Boxes) that have a minimum horizontally projected surface area below the overflow which is at least 5% of the total surface area draining to it.

4. Sheet Flow Dispersion in accordance with BMP T5.12: Sheet Flow Dispersion, or Concentrated Flow Dispersion in accordance with BMP T5.11: Concentrated Flow Dispersion

[1] This is not a requirement to pave these surfaces. Where pavement is proposed, it must be permeable to the extent feasible unless full dispersion is employed.



DEPARTMENT OF
ECOLOGY
State of Washington

Figure I-2.5.1
Flow Chart for Determining LID MR #5 Requirements

Revised June 2016

UGA - URBAN GROWTH AREA. ALL OF BAINBRIDGE ISLAND IN UGA

http://www.ecy.wa.gov/programs/wq/stormwater/manual/2014SWMMWWinteractive/Co... 12/28/2016

Section 4 – MR #2 Stormwater Pollution Prevention Plan (SWPPP) Narrative

Every Construction Stormwater Pollution Prevention Plan (SWPPP) must address the **13** required elements from the Washington State Department of Ecology *SWMMWW*.

Check the suggested BMP you will use to satisfy the required element and **identify location on the stormwater site plan**. If an element does not apply to your proposal, provide a written justification identifying the reason an element is not applicable to the proposal.

1. **Preserve Vegetation/Mark the Area Disturbed by Construction Activity.** Describe the total disturbed area (grading, building pad, driveway, septic installation, etc.) and reference how you will clearly mark the area of disturbance.
   - ☐ BMP C101 – Preserving Natural Vegetation
   - ☐ BMP C102 - Buffer Zones
   - ☐ BMP C103 – High Visibility Plastic or Metal Fence
   - ☑ BMP C104 – Stake and Wire Fence

2. **Establish Construct Access**. Describe construction access.
   - ☐ BMP C105 – Stabilized Construction Entrance
   - ☐ BMP C106 – Wheel Wash
   - ☐ BMP C107 – Construction Road/Parking Area Stabilization
   - ☑ Not applicable – Existing access will prevent tracking of sediment onto public right-of-way

3. **Control Flow Rates.** If there is substantial grading and/or the potential for stormwater runoff to flow off site during construction, then one of the two BMPs must be identified and shown on the site plan.
   - ☐ BMP C240 – Sediment Trap
   - ☐ BMP C241 – Temporary Sediment Pond
   - ☑ Not applicable – Very little grading and/or site does not experience site runoff during storm events

4. **Install Sediment Controls.** When there is grading on a site and the site is sloped, there is a potential for sediment to leave the site during storm events. Please identify a BMP below if your site has any slope to it.
   - ☐ BMP C231 – Brush Barrier
   - ☐ BMP C232 – Gravel Filter Berm
   - ☑ BMP C233 – Silt Fence
   - ☐ BMP C234 – Vegetated Strip
   - ☐ BMP C235 – Straw Wattles
   - ☐ Site is flat and no potential for sediment to leave the site exists

5. **Stabilize Soils**. All exposed soil must be protected from rainfall and wind erosion. From October 1 through April 30, no soil shall remain exposed and unworked for more than 2 days. From May 1 to September 30, no soils shall remain exposed and unworked for more than 7 days.
   - ☑ BMP C120 – Temporary and Permanent Seeding
   - ☑ BMP C121 – Mulching
   - ☐ BMP C122 – Nets and Blankets
   - ☑ BMP C123 – Plastic Covering

6. **Project Slopes.** If the property has slopes, they must be protected from erosion if work is done on or near them.
   - ☐ BMP C120- Temporary and Permanent Seeding
   - ☑ BMP C130 – Surface Roughening
   - ☐ BMP C131 Gradient Terraces
   - ☐ Not Applicable – The property does not have any slopes nor are there any slopes within 100 Feet of the project boundaries

   BMP C204- PIPE SLOPE DRAIN WILL BE USED TO PROTECT THE CUT SLOPE MADE DURING CONSTRUCTION OF THE HOUSE.

7. **Protect Drain Inlets.** Storm drains shall be protected from sediment entering them.
   - ☑ C220 – Storm Drain Inlet Protection
   - ☐ Not Applicable – There are no storm drains on the property or within 100 feet of the stabilized construction access.

8. **Stabilize Channels and Outlets.** If temporary on-site conveyance channels are used, they must be stabilized to protect against erosion.
   - ☑ BMP C202 – Channel Lining
   - ☐ BMP C209 – Outlet Protection
   - ☐ Not Applicable – Temporary on-site conveyance channels are not used for this project.

9. **Control Pollutants.** All pollutants shall be handled and disposed of in a manner that does not cause contamination of stormwater. Please identify any BMP used for the project.
   - ☑ BMP C151 – Concrete Handling
   - ☐ CMP C152 – Sawcutting and Surfacing Pollution Prevention
   - ☐ Above BMP not expected to be necessary, however all necessary precautions will be taken to ensure pollutants are handled and disposed of in a safe manner

10. **Control De-Watering.** If the site is expected to experience ponding and/or foundation is left in a manner that encourages water ponding, then the applicant shall make necessary plans to discharge the water in a manner that ensures it is safely cleaned before being discharged. Describe the plan for dewatering below.
    - ☐ Not applicable. Site does not experience ponding and foundation will be kept dry such that water accumulation does not occur.

THE CONTRACTOR SHALL DEVELOP A DEWATERING PLAN TO PREVENT SEDIMENT LADEN WATER FROM ENTERING THE BEACH.

11. **Maintain BMPs.** All temporary and permanent erosion and sediment control BMPs shall be maintained and repaired as needed to assure continued performance of their intended function.
   - ☑ BMPs will be checked weekly and immediately after storm events.
   - ☐ Other: ______________________________

____

12. **Managing the Project.** Phasing of the project is encouraged to prevent soils from being exposed for extended periods of time. Please describe how you will be planning your project to ensure that construction impact and soil exposure is limited.

THE PROJECT WILL BE PHASED TO THE MAXIMUM EXTENT PRACTICABLE. NOTES ON PLAN SPECIFY THAT NO SOILS SHALL REMAIN EXPOSED OR UNWORKED FOR MORE THAN 2 DAYS DURING THE WET SEASON AND 7 DAYS DURING THE DRY SEASON.

13. **Protect Low Impact Development BMPs.** Phasing of the project is encouraged to prevent soils from being exposed for extended periods of time. Please describe how you will be planning your project to ensure that construction impact and soil exposure is limited.

NOTES ON PLAN SPECIFY HOW TO PROTECT THE PERVIOUS PAVER AREA.

February 23, 2017

Margaret Dufresne
3912 NE State Highway 104
Poulsbo, WA 98370

Re: Dufresne Residence at 11143 Rolling Bay Walk

Dear Margaret:

This letter presents a drainage plan for your proposed residence at 11143 Rolling Bay Walk NE Bainbridge Island, WA 98110.

EXISTING CONDITIONS

The project site consists of one waterfront parcel totaling 0.44 acres, located southwest of Rolling Bay Walk and the Puget Sound.

The property is located on a northeast facing slope. The uphill portion of the site is undeveloped and vegetated with second growth forest.

From the southwest property line, grade drops approximately 76-feet, spanning a horizontal distance of about 100-feet to the northeast. At this point there is a two tiered soldier pile wall catchment system. The soldier pile wall system consists of a tiered 5-foot and 10-ft wall approximately 14-ft apart. The grade is approximately 50-percent from Rolling Bay Walk, up to the soldier pile wall system. The site is located in a geotechnically hazardous area due to the steep slopes and past landslide of the shoreline bluff.

The lower portion of the site was previously developed with a single family residence that was severely damaged by a landslide in 1997 and demolished thereafter. Currently, the lower portion of the property is vegetated with blackberry brambles, a few small alders, and grass.

Soils on site are mapped as 70-percent Dystric Xerothents, at 45 to 70-percent slopes and about 30-percent beaches, by USDA NRCS. The geotechnical report by Perrone Consulting, Inc., dated February 14, 2016, indicates that the lower portion of the site is underlain by landslide debris overlying beach deposits over glacial deposits. The upper portion of the site to the southwest of the soldier pile wall consists of dense glacial till overlying glacial advance outwash deposits. Based on their report ground water elevation is approximately 5-feet below Rolling Bay Walk.

Currently there is shared access that runs along Rolling Bay Walk.

Runoff appears to sheet flow offsite over Rolling Bay Walk to the Puget Sound.

PROPOSED DRAINAGE SYSTEM

The proposed project includes construction of a residence, garage, and driveway apron. The project will create about 1,660-square feet of new hard surface area. The anticipated disturbed area is approximately 5,800-square feet.

The driveway will be constructed with pervious pavers. A small portion of pavers (4-foot strip) will be exposed to the sky; the remainder will be under the roof eave. The roof drains will discharge onto splash blocks, and runoff will flow over Rolling Bay Walk.

Rolling Bay Walk, along the property frontage will be overlaid with asphalt. This will allow the water to sheet flow to the beach as opposed to flowing to notches in the concrete bulkhead. The existing pavement will not be removed to overlay the road, so this area is not considered new or replaced hard surface.

THRESHOLD DETERMINATION

The new plus replaced hard surface area (1660-square feet) is greater than 800-square feet but less than 5,000-square feet, the converted pervious area (0-square feet) is less than 3/4-acre, and the converted forest to pasture area (0-square feet) is less than 2.5-acres. Using these quantities, Minimum Requirements 1 through 5 apply to the new and replaced hard surface area, and the land disturbed.

See table 1 for a description of how each Minimum Requirement is met.

Landscape areas disturbed by the construction will be amended to meet BMP T5.13 Post-Construction Soil Quality and Depth.

EROSION CONTROL

The Contractor will be responsible for maintaining erosion control facilities on the site during construction and for ensuring that sediment does not leave the site. The general principles of construction pollution prevention are:

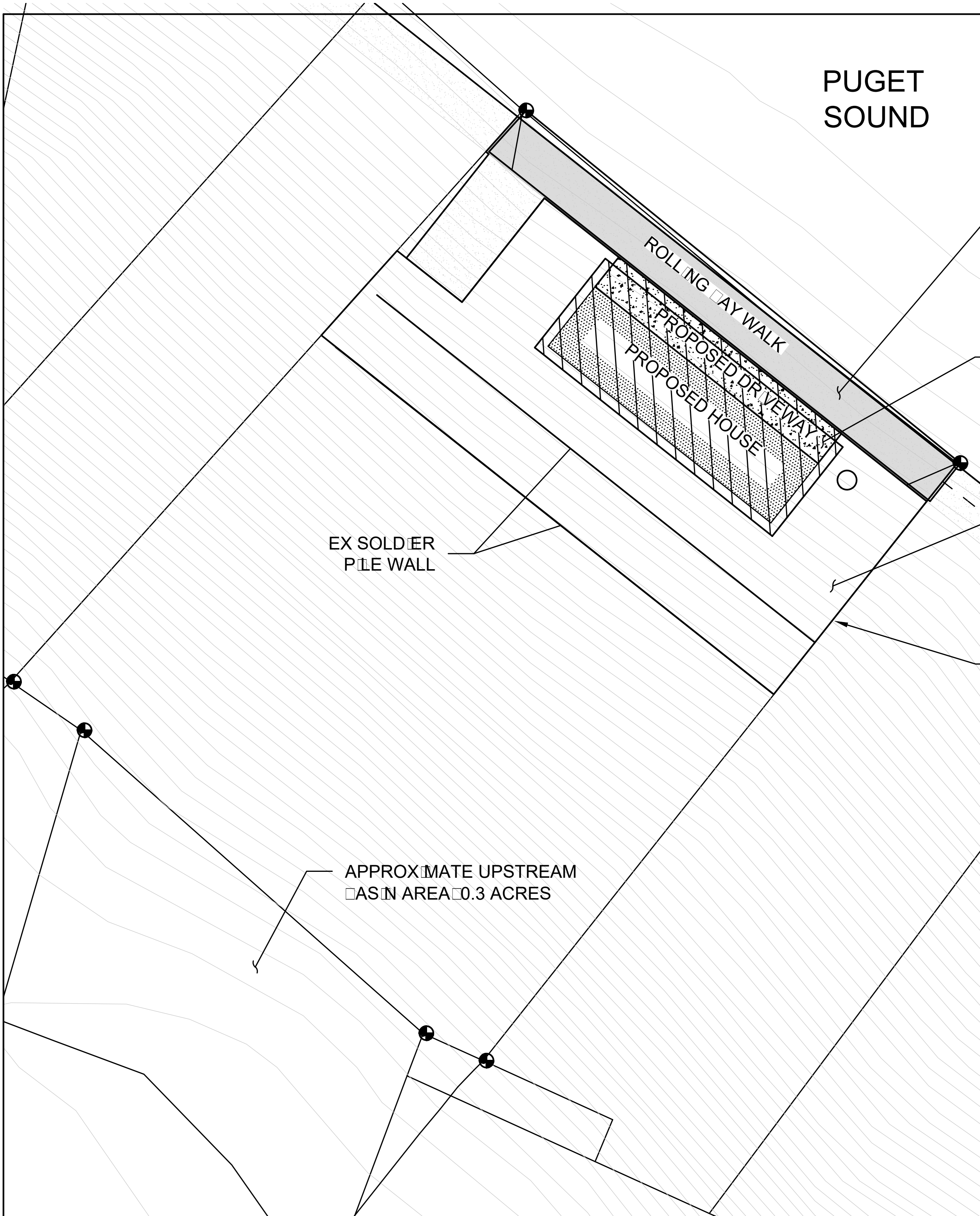
- Retain native vegetation
- Prevent erosion rather than treat sediment laden water.
- Employ site specific best management practices (BMPs)
- Divert upslope runoff around disturbed area
- Phase construction operations to reduce total amount of disturbance at one time
- Amend soils before seeding
- Minimize the slope length and steepness of disturbed areas
- Reduce runoff velocities
- Prevent the tracking of sediment off site
- Employ BMPs that address not only erosion but also other potential pollutants.

The plan shows a number of BMPs which we believe are the minimum required to prevent erosion. It should be noted that other measures may be needed to minimize the movement of sediment and shall be put in place as needed. To prevent erosion the contractor should take special care to ensure that exposed soils are covered in accordance with the plans. Clearing limits are shown on the plan with silt fence and clearing limits fence. The contractor should install and maintain fencing along the limits. The contractor should also ensure that disturbance outside of the limits does not occur unless needed. Rolling Bay Walk will be used as a stabalized constuction entrance. The existing gravel parking area at the north side of the site will be used for lay down area and construction parking. A silt fence will be installed on along the downhill edge of the project to prevent sediment from leaving the site during construction. Table 2 describes how the minimum requirements for construction stormwater pollution prevention are addressed on the plan. If the contractor needs to employ additional BMPs they should reference the *SWMMWW*, 2012 edition for additional information.

Very truly yours,

BROWNE WHEELER ENGINEERS, INC.

2/23/2017



N

THRESHOLD AREA PLAN

SCALE: 1"=30'

0 5 10 20 30 60

THRESHOLD AREAS
DUFRESNE RESIDENCE
BUILDING PERMIT APPLICATION

BROWNE • WHEELER
ENGINEERS, INC
241 ERICKSEN AVENUE NE
BAINBRIDGE ISLAND, WA 98110
P 206.842.0605 INFO@BrowneWheeler.COM

FILE NAME: DRAINAGE PLAN.DWG PLOT DATE: 2/23/2017 10:38 AM PLOT DEVICE: DWG TO PDF.PC3 PAGE SETUP: ---- PLOTTED BY: KELSEY

Table 1
Summary of Stormwater Minimum Requirements

Minimum Requirement	Comment
1. Stormwater site plan	A stormwater site plan is presented.
2. Construction stormwater pollution prevention	See report, plan and Table 2.
3. Source control of pollutants	Not applicable.
4. Preservation of natural drainage systems and outfalls	The stormwater onsite will continue to discharge to the same location.
5. On-site stormwater management	Permeable pavement will be used for the driveway. House roof drains will be connected to splash blocks that discharge to Rolling Bay Walk.
6. Runoff treatment	Not applicable
7. Flow control	Not applicable
8. Wetlands protection	Not applicable
9. Basin and watershed planning	Not applicable
10. Operation and maintenance	Not applicable

Table 2
Construction Stormwater Pollution Prevention Elements-Minimum Requirement 2

Element	Comment
1. Mark clearing limits	Clearing limits are marked by silt fence and clearing limits fence.
2. Construction entrance	The Rolling Bay Walk will be used as a temporary construction entrance.
3. Control flow rates	Runoff will discharge directly to salt water.
4. Sediment controls	A silt fence will be installed along the downhill edge of the project site.
5. Stabilize soils	Soil cover requirements are specified in the plan set notes.
6. Protect slopes	A pipe slope drain, along with a swale, will be installed uphill of the cutslope that will be created during the construction of the house. The steep slope above the house will be utilize surface roughening BMP C130 to protect the slope during construction of the drainfield.
7. Protect drain inlets	Notes require that all drain inlets be provided with a filter if used prior to site stabilization.
8. Stabilize channels	Notes require that all channels be stabilized with grass or rock.
9. Control pollutants	Note provided to require control of pollutants.
10. Control dewatering	Notes addressing methods for handling dewatering water are specified on the plan.
11. Maintain BMPs	Note provided regarding maintenance.
12. Manage project	Notes provided regarding scheduling and timing of disturbed soil exposure.
13. Protect LID	Notes on the plan state contractor to protect pervious paver area from compaction and soil contamination.

Duresne Residence
List #1 Evaluation -Residence
2/23/2017

	Infeasibility Criteria	Feasible
T5.30	Full Dispersion	**No**
	100' flow path	No
	Retain > 65% Native and < 10% Impervious	
	<15% slope for any 20 ft	
	<33% slope	
	Flowpaths don't intersect	
	Onsite sewage disposal system setbacks	
T5.10A	Full Infiltration	**No**
	Geotechnical Evaluation: Infiltration Feasible	No
	>3ft permeable soil above groundwater	
	>1 ft clearance from trench bottom and groundwater	
	Sufficient space to construct trench per manual	
	> 200ft to 40% slope	
	<25% slope	
	Down Gradient of septic system	
T7.30	Bioretention	**No**
	Geotechnical Evaluation: Infiltration Feasible?	No
	Meets Building Setbacks	
	<8% slope	
	>50ft from 20% slope or 10ft tall bank	
	>100ft to well	
	>10ft to small onsite sewage disposal system	
	Meets LOSS setbacks	
	>1ft separation to imp. layer if < 5000 PGIS, 10,000 Imp, 3/4 ac perv.	
	>3ft separation to imp. layer if > 5000 PGIS, 10,000 imp, 3/4 ac perv.	
	> 0.3in/hr	
T5.10B	**Downspout Dispersion System**	**No**
	>25ft vegetated flow path	No
	>50ft from >15% slope	
	>5ft setback from structure or property line	
	Flowpaths don't intersect	
	Vegetated flow path	
T5.10C	Perforated Stub-Out Connections	**No-Based on Geotech assesment**
	>1ft separation to imp. Layer	
	Down Gradient of Septic System	
	Slope is <20% at perforated pipe portion	

Dufresne Residence
List #1 Evaluation - Paving
2/23/2017

	Infeasibility Criteria	Feasible	
T5.30	Full Dispersion	**No**	
	Retain > 65% Native and < 10% Impervious	Yes	
	100' flow path	No	
	<15% slope for any 20 ft		
	<33% slope		
	Flowpaths don't intersect		
	Onsite sewage disposal system setbacks		
T5.15	Permeable Pavement	**Yes**	
	Geotechnical Evaluation: Infiltration Feasible?	Yes	
	> 50ft to 20% slope	Yes	
	>100ft to well	Yes	
	>10ft to small onsite sewage disposal system	Yes	
	Meets LOSS setbacks	Yes	
	<5% slope PAS, <10% PCS, 12% PICPS	Yes	
	Native soils meet SSC for pollution generating surfaces	Yes	
	>1ft separation to imp. layer	Yes	
	>5% California Bearing Ratio	Yes	Assumed- Adjacent Rolling Bay Walk appears capable of supporting traffic loads.
	> 0.3in/hr	Yes	Assumed- Geotech indicated that the area is underlain by beach sands.
	<400 ADT not applicable to non traffic bearing surfaces	Yes	
T5.14A	Rain Garden or Bioretention	**N/A**	
	Geotechnical Evaluation: Infiltration Feasible?		
	Meets Building Setbacks		
	<8% slope		
	>50ft from 20% slope or 10ft tall bank		
	>100ft to well		
	>10ft to small onsite sewage disposal system		
	Meets LOSS setbacks		
	>1ft sep. to imp. layer if < 5000 PGIS, 10,000 Imp, 3/4 ac perv		
	>3ft sep. to imp. layer if > 5000 PGIS, 10,000 imp, 3/4 ac perv		
	> 0.3in/hr		
T5.12	Sheet Flow Dispersion	**N/A**	
	>10ft-wide vegetated buffer for up to 20' of width		
	And an additional 10 ft of buffer width for each add. 20' of imp		
	<15% slope		
	>10ft downhill of septic field		
T5.11	Concentrated Flow Dispersion	**N/A**	
	>50ft vegetated flow path to property line, structure, 15% slope		
	>10ft downhill of septic field		

Aspect CONSULTING

November 7, 2017

Margaret Dufresne
3912 Hwy 104
Poulsbo, Washington 98370

Re: Geotechnical Response to Third-Party Review Comments
Dufresne Residence
11143 Rolling Bay Walk NE
Bainbridge Island, Washington 98110
Project No. 170202

Dear Ms. Dufresne:

Aspect Consulting, LLC (Aspect) has received from the City of Bainbridge Island (City) a copy of their third-party review letter (AMEC Foster Wheeler, 2017) for your proposed residential project at 11143 Rolling Bay Walk NE, Bainbridge Island, Washington (Site). We were requested by the City and you to respond to the comments in the third-party review letter.

For a full description of the project and Site, please refer to our original evaluation (Aspect, 2017).

Item 1—Plans Review

We have received updated design documents for the project, including:

- Soldier Pile & Timber Lagging Retaining Wall and House Structural Plans, Sheets SH-1.0 to SH-14.0, prepared by Lucia Engineering, Inc., dated November 1, 2017.
- Drainage Plan, Sheets C1 to C2, prepared by Browne-Wheeler Engineers, Inc., dated February 9, 2017.
- Dufresne Residence OCC Dispersal System Detail Plans, prepared by MBH2O, dated October 19, 2017.

In our opinion, the design drawings listed above appropriately incorporate our geotechnical recommendations and considerations for the project. As requested by the City, a completed copy of the City Permit Issuance “Step-2” Form to reflect our review is attached. Additional discussion of the pertinent design specifics is included below, in response to other review comments.

Item 2 – Current Status of the Existing Landslide Debris Catchment Wall System

As part of our original evaluation (Aspect, 2017), we reviewed the status and condition of the existing landslide debris catchment wall system at the Site. In our opinion, the walls were functioning as intended, with adequate freeboard or catchment for future surficial landslide debris. We recommend the wall system be maintained as described in the design basis report (GeoEngineers, 2005) and the Certification for Final Inspection form (GeoEngineers, 2006). This includes regular monitoring, maintaining the recommended freeboard (described as an upper wall height of approximately 10 feet above the surrounding grade) along the entire alignment of the upper wall at the Site, removal of any accumulated slide debris against the upper wall, and completing repairs to the walls (as needed).

The lower wall was constructed solely for construction access (not landslide debris catchment), but we recommend annual monitoring of the lower wall since it will support the on-site septic system and surrounding ground, and facilitate future maintenance of the upper wall. Removal of slide debris against the upper wall should be accomplished with hand tools and/or small, light duty equipment that is sized to not damage the on-site septic system.

Item 3 – Critical Area Buffer and Setbacks

The majority of the Site is mapped as a greater than 40 percent steep slope geologically hazardous area (critical area/landslide hazard area), with the northeastern edge of the Site mapped as a 15 to 40 percent slope geologically hazardous area. Our findings indicate that the entire Site meets the definition of the landslide hazard area, but with the appropriate landslide mitigation techniques and associated engineering, the Site can be developed in a manner to provide an acceptable level of protection to the proposed residence and associated infrastructure.

Because the proposed residence will be located entirely within the landslide hazard area, we understand a shoreline variance will be applied to the project and that no specific buffer reductions or setback recommendations are required. Our agreement with the proposed residence location at the Site is captured in our response to Item 1 and the attached City Step-2 form.

Item 4 – On-Site Septic Drainfield

We collaborated with your septic designer to complete updates to the on-site septic design. The drainfield will be located between the existing soldier pile retaining walls on the steep slope (above and southwest of the proposed residence). The existing walls include wood lagging facing that is considered free draining. Based on the soil conditions observed during our evaluation, the soils underlying the proposed drainfield and between the existing walls consist of loose, gravelly, silty SAND (SM)[1].

[1] Soil Classification per the Unified Soil Classification System (USCS). Refer to American Society of Testing and Materials (ASTM) D2488.

The updated septic design includes provisions for the following elements to appropriately handle and route effluent flows and to protect the drainfield:

- Impermeable visqueen will be installed along the full depth of the downslope (northeast) side of the sand filter, extending below the exposed height of the lower soldier pile wall to prevent lateral migration of effluent to the face of the exposed portion of the existing lower wall and any seepage through the wood lagging.

- Based on the soil conditions observed, the septic designer has concluded that it is reasonable to assume vertical infiltration of effluent through the sand filter and into the underlying soils. We agree with this assessment.

- A 12-inch-thick cushion layer of sand will be constructed over the top of the lateral dispersion elements of the drainfield. The cushion layer will also be surfaced with StabiliGrid HD™, which is an interconnected lattice of polyethylene grid infilled with crushed rock. This surfacing will provide additional protection to the drainfield from debris or tree falls and allow for light duty equipment to operate over the drainfield during future maintenance activities associated with the landslide debris catchment walls.

- Drainage elements within 20 feet (laterally) of the septic drainfield should consist of composite drain boards (such as MiraDRAIN) or conventional drains (perforated pipes) backfilled with medium sand and not washed rock. This will help ensure all effluent is fully treated and filtered as intended and limit free-draining seepage pathways into the Site drainage and stormwater systems.

Item 5 – Seismic Site Class

To develop the recommended seismic Site Class, we utilized the methods presented in Chapter 20 of the American Society of Civil Engineers (ASCE) 7-10, *Minimum Design Loads for Buildings and Other Structures.* There are a variety of ways to determine the seismic site class, and we chose the method based on the measured and inferred N-values of the upper 100 feet of the Site soil profile.

We used the data from boring B-02, as it was located closer in elevation to the proposed residence foundation than B-0,1 to calculate the seismic site class. Boring B-02 extended to 26.4 feet below ground surface (bgs) and encountered very dense Advance Outwash deposits with N-values exceeding 60 to 100 blows per foot below 13.5 feet bgs. Using our geologic experience, we inferred that since the Advance Outwash deposits are glacially overridden, any underlying soils have also been glacially overridden; therefore, the soils conditions below the extents of boring B-02 could be assumed to exhibit N-values equal to or greater than 100 blows per foot (the final N-value measured at 25 feet bgs on B-02). We also anticipated that the proposed residence structure will be supported by foundations embedded into the very dense Advance Outwash deposits and not gaining support for the loose upper landslide deposits.

Taking this information on aggregate, we calculated the average field standard penetration resistance (N_{avg}) over the upper 100 feet of the Site subsurface profile below the proposed residence location is just greater than 50 blows per foot, justifying a seismic Site Class C. However, the calculation is close to the cutoff between seismic Site Class C and Site Class D and is reliant on inferences made below the depth of the available exploration data and based on our geologic experience in the vicinity of the Site. Therefore, we agree with the reviewer and as a conservative measure, recommend the use of seismic Site Class D for design of the structure according the parameters shown below in Table 1.

Table 1. Revised Seismic Design Parameters

Design Parameter	Recommended Value
Seismic Site Class	D
Short Period Spectral Acceleration (S_S), g	1.352
1-Second Period Spectral Acceleration (S_1), g	0.531
Peak Ground Acceleration (PGA), g	0.554
Site Coefficient (F_{PGA})	1.000
Site Coefficient (F_a)	1.0
Site Coefficient (F_v)	1.5
Adjusted Peak Ground Acceleration ($A_s = PGA*F_{PGA}$), g	0.554 (Site Class D)
Design Short Period Spectral Acceleration (S_{Ds}), g	0.901 (Site Class D)
Design 1-Second Period Spectral Acceleration (S_{D1}), g	0.531 (Site Class D)

Item 6 – Tsunami Hazards

Due to the low-lying shoreline location of the Site, there is a tsunami hazard at the Site. Tsunamis are large waves generated by an earthquake, submarine landslide, or other disturbance. The magnitude of the tsunami hazard is dependent on the tide stage at the time of the earthquake and type of event generating the tsunami. Generally, within the Puget Sound, the primary source of potential tsunamis is a crustal earthquake originating within the Seattle Fault zone and the associated submarine ground deformation. Depending on the extent of surface rupture along the Seattle Fault during an earthquake, the rupture and shaking can cause underwater landslides and earth movement that in turn is capable of generating tsunamis within the Puget Sound.

Recent studies [(DNR, 2003); (USGS, 2017)] have estimated tsunami wave heights or depth of inundation along shorelines ranging from of about 3 to 12 feet high interior to the central part of the Puget Sound. Depending on the tide stage at the time of tsunami occurrence, the Site may be unaffected (low tide) or partially flooded (high tide) with potential for soil erosion and wave impact forces on the structure.

Based on section 16.20.060(K)(4)(e) of the Bainbridge Island Municipal Code (BIMC), we understand the City will provide applicants for development in low-lying shoreline areas and other areas where flood elevation is controlled by tide level with information on tsunami hazards.

Item 7 – Foundation Recommendations

Due to the liquefaction hazard at the Site, we've recommended the proposed residence be supported by pile foundations. We understand from the provided design plans that the residence structure will be supported on three sides by soldier piles that will act as foundation support, temporary shoring during construction, and permanent walls for the basement. The remaining side (northeast) and interior walls and footings will be supported by 2-inch-diameter steel pin piles.

The reviewer has asked that we clarify if these recommendations represent the "highest standard of safety feasible as stated in the code." We are not aware of this specific language in the relevant building codes and ask for clarification on the source of the language from the reviewer. Our understanding of the seismic design provisions in the International Building Code (IBC) and ASCE 7-10 in the context of this project is that life safety must be preserved during and following the Maximum Considered Earthquake (MCE, 2,475-year return period).

The word "feasible" is a subjective descriptor; however, in our opinion, our foundation recommendations do reflect the highest standard of safety feasible in the context of the project. Taking into account the Site conditions, existing landslide debris catchment walls, the proposed shoring wall, the identified liquefaction hazard, and shoreline geometry, it is our opinion that support of the residence with a combination of soldier piles and small diameter steel "pin" piles will result in a foundation configuration that will maintain the structural integrity of the residence such that life safety is preserved during and following the MCE.

Item 8—Construction Monitoring

We are available to provide geotechnical engineering and monitoring services during construction. The integrity of the foundation and shoring walls depends on proper Site preparation and construction procedures. In addition, engineering decisions may have to be made in the field in the event that variations in subsurface conditions become apparent.

References

American Society for Testing and Materials (ASTM), 2012, American Society of Testing Materials Annual Book of Standards, Vol. 4.08, West Conshohocken, Pennsylvania.

AMEC Foster Wheeler, 2017, Geotechnical Review of Documents, Dufresne – PLN50287 SVAR, 11143 Rolling Bay Walk NE, Bainbridge Island, Washington, dated October 5, 2017.

American Society of Civil Engineers (ASCE), 2013, Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers Standard 7-10, 2013.

Aspect Consulting, LLC (Aspect), 2017, Geotechnical Report, Dufresne Residence, 11143 Rolling Bay Walk NE, Bainbridge Island, Washington, dated July 20, 2017.

GeoEngineers, Inc., 2005, Design Consultation, Proposed Slide Debris Catchment Wall, 11129 and 11139 Rolling Bay Walk, Bainbridge Island, Washington, June 13, 2005.

GeoEngineers, Inc., 2006, Geotechnical Construction Observation Services, Slide Debris Catchment Wall, Low Tide, LLC Property at 11099 Rolling Bay Walk, Bainbridge Island, Washington, September 8, 2006.

International Building Code (IBC), 2015, International Building Code. Prepared by International Code Council, May 2014.

U.S. Geological Survey (USGS), 2017, Washington Earthquake Fact Sheet, Website, Accessed on October 26, 2017, https://pubs.usgs.gov/fs/FS-047-96.

Washington State Department of Natural Resources (DNR), 2003, Tsunami Hazard Map of the Elliot Bay Area, Seattle, Washington: Modeled Tsunami Inundation from a Seattle Fault Earthquake, Washington State Division of Geology and Earth Resources Open File Report 2003-14, by Walsh, T.J, Titov, V.V, Venturato, A.J., Mofjeld, H.O., and Gonzalez, F.I.

Limitations

The conclusions and recommendations provided above are based on the information collected during our previous Site visits, limited subsurface exploration data at the Site, our experience with the local geology near the Site, and pertinent available published data. Within the limitations of scope, schedule, and budget, our services have been performed for Margaret Dufresne (Client), and this letter was prepared in accordance with generally accepted geotechnical engineering and engineering geology practices in effect in this area at the time our letter report was prepared. This letter report does not represent a legal opinion. No other warranty, expressed or implied, is made.

All reports prepared by Aspect Consulting for the Client apply only to the services described in the Agreement(s) with the Client. Any use or reuse by any party other than the Client is at the sole risk of that party, and without liability to Aspect Consulting. Aspect Consulting's original files/reports shall govern in the event of any dispute regarding the content of electronic documents furnished to others.

We believe this addresses all questions from the third-party reviewer and should allow your permit to continue to process. If you have any questions, please do not hesitate to call.

Sincerely,

Aspect consulting, LLC

ANDREW JOHN HOLMSON
STATE OF WASHINGTON
49471
REGISTERED
PROFESSIONAL ENGINEER
11/7/2017

Andrew J. Holmson, PE
Senior Geotechnical Engineer
aholmson@aspectconsulting.com

Henry H. Haselton, PE
Principal Geotechnical Engineer
hhaselton@aspectconsulting.com

Attachments

City of Bainbridge Island Permit Issuance "Step-2" Form

V:\170202 Dufresne Residence\Deliverables\Response Letter\Dufresne Residence Third Party Review Response Letter.doc

REQUIREMENTS FOR CONSTRUCTION IN GEOLOGICALLY HAZARDOUS AREAS

PERMIT ISSUANCE FORM

"Step 2"

GEOTECHNICAL FORM FOR PERMIT ISSUANCE : **TO BE COMPLETED BY A <u>LICENSED ENGINEER</u> IN THE STATE OF WASHINGTON QUALIFIED IN THE SPECIALTY OF GEOTECHNICAL ENGINEERING AND SUBMITTED PRIOR TO PERMIT ISSUANCE <u>ATTACHED AS A PART OF A LETTER FROM THE GEOTECHNICAL ENGINEER BEARING THE ENGINEER'S SEAL.</u>**

Project Number: ________________ Permit Number: PLN50287 SVAR

Applicant's or Project Name : Dufresne Residence (Margaret Dufresne)

Project street address: 11143 Rolling Bay Walk NE

Geotechnical Engineer's Name and License # Andrew J. Holmson #49471

Engineer's Telephone #: (206) 780-7731

Signature & Date: Andrew Holmson 11/7/2017

> This form is required prior to issuance of the permit. At the time of permit issuance the recommendations of the Geotechnical Engineer are required to be incorporated into the project plans. This form is to facilitate your assurance that you reviewed the plans and found your recommendations in them.

1) "I have reviewed the plans and design elements related to the geotechnical aspects of the project and have observed field staked locations of proposed structures subject to the recommendations of the Geotechnical Report. I have found the plans, design and locations to be in general accordance with the recommendations of the Geotechnical Report."

   I agree with the above statement Initial : AH

   State any variations from your original recommendations in implementing these plans:

2) In accordance with Chapter 16.20.150 of the City of Bainbridge Island Municipal Code, the Geotechnical Engineer must be able to make the following statements regarding implementation of their recommendations in the plans. Concurrence with the following statements shows that the project plans implement or include all of your required recommendations and that they meet the requirements of the City of Bainbridge Island Municipal Code; please initial each of the below statements if you fully concur that they have been met in your professional opinion:

a. The proposed activity shall not create a net increase in geological instability, either on- or off-site, which is defined as follows:
i. The subject parcel shall not be less stable after the planned development than before; and
ii. The adjacent parcels shall not be less stable after the planned development than before.
Initial AH

b. The proposed activity shall not increase the risk of life safety due to geological hazards above professionally acceptable levels.*
Initial AH

c. The proposed activity shall not increase the risk due to geological hazards above professionally acceptable levels for:*
i. Property loss of any habitable structures or their necessary supporting infrastructure on-site or;
ii. Risk to any off-site structures or property of any kind.
Initial AH

*d. Proposed buildings shall be constructed using appropriate engineering methods that respond to the geologic characteristics specific to the site in order to achieve the highest standard of safety feasible**.*
Please explain how this requirement has been met: The residence will be protected from the landslide hazard at the site by the existing tiered soldier pile wall debris catchment system. The calculated global factor of safety exceeds 1.5 for static conditions and 1.0 for seismic conditions. The residence will be supported on three sides by soldier piles that will act as foundation support, temporary shoring during construction, and permanent walls for the basement. The remaining side (northeast) will be supported by 2-inch-diameter steel pin piles. The combination of soldier piles and pin piles will result in a foundation configuration that will maintain the structural integrity of the residence such that life safety is preserved during and following the Maximum Considered Earthquake (MCE, 2,475 year return period). The construction recommendations presented in the report and should be incorporated into the construction methodology to maintain local stability and provide for competent foundation support.

Pursuant to BIMC 16.20.150 E.1 the City Engineer will generally accept:

* a "professionally acceptable level" of risk as a static factor of safety of 1.5 for new construction or 1.25 for remodel, and a seismic factor of safety of 1.0 for the design earthquake (using the USGS 2002 probabilistic ground motion values for 2% in 50 yr.)
** the "highest standard of safety feasible" has been achieved if all buildings are designed to meet the IBC as appropriate to withstand all events up to the above defined "professionally acceptable level" of risk and a reasonable explanation is provided.

3) The Engineering Department requires the Geotechnical Engineer to communicate the overall risk of failure and impact to the proposed structures in a quantified manner, along with the level of certainty in this quantification. This quantification may be a factor of safety or some other quantification acceptable to the City Engineer, and needs to communicate to the owner (and future owners) an understandable, estimated or calculated risk to or from their proposed activities or structures in some relevant time frame (often the lifetime of the structure).

   Please provide your determination of the above required level of risk here: ______ ____Factors of safety against deep-seated slope movement effecting the proposed residence exceeds 1.5 static and 1.0 seismic. The use of soldier piles and pin piles driven to refusal mitigate for liquefaction hazards associated with the maximum considered earthquake (MCE) or an earthquake with a return interval of 2,475 years. The risk of occurrence of landslide-related damage to the proposed structure is less than 1-in-100 years and the risk of occurrence of significant liquefaction or earthquake-related damage (collapse) to the proposed structure is less than 1-in-2,475 years; however, structural and cosmetic repairs may be needed following the MCE.

   ____

4) Storm-water:

   ☐ **a. Landslide hazard areas or erosion hazard areas.** I confirm that the Erosion and Sediment Control Plan in the application dated: February 9, 2017 (AH) is in accordance with my recommendations.

   ☐ **b. Landslide hazard areas, erosion hazard areas, area of influence.** I confirm that the Surface & Storm Water Management Plan in the permit application dated: (☐N/A) is in accordance with my recommendations.

5) Address any special geotechnical conditions requested by the City after the City's review of the geotechnical report and permit application forms here (attach more sheets as needed and any supporting documentation):

SUBMITTAL REQUIREMENTS

Two (2) copies of the completed forms, **two (2)** copies of the recorded indemnification forms, **two (2)** copies of all supporting documents (Geotechnical analysis, erosion and sediment control plan, etc.). **Proposals will not be considered further for each step of the permit until form packets are complete**

Section 3 – Prescriptive Whole House Ventilation (IRC M1508)

Type of System: ☒ Continuous (fan runs continuously) ☐ Intermittent (fan runs _____% of the time)
☐ Integrated Forced Air System 60 cfm airflow rate required

Section 4 – Energy Credits.

Chapter 4 of the IECC requires additional energy efficiency to your project using a "credit" system. In order to determine how many credits apply to your project, please identify the number of credits required from **Table 1**. Then, select enough credits from **Table 2** to meet that requirement. Please note: The building permit drawings shall reflect required heating equipment type, minimum equipment efficiency and any other requirements to show compliance.

Table 1

⇩		Description	Required Credit
	#1	. Dwelling units less than 1500 square feet in conditioned floor area with less than 300 square feet of fenestration area. Additions to existing building greater than 500 square feet of heated floor area but less than 1500 square feet.	1.5
	#2	All dwelling units that are not included in #1 or #3. Exception: Dwelling units serving R-2 occupancies shall require 2.5 credits.	3.5
	#3	Dwelling units exceeding 5000 square feet of conditioned floor area. Exception: Dwelling units serving R-2 occupancies shall require 2.5 credits.	4.5

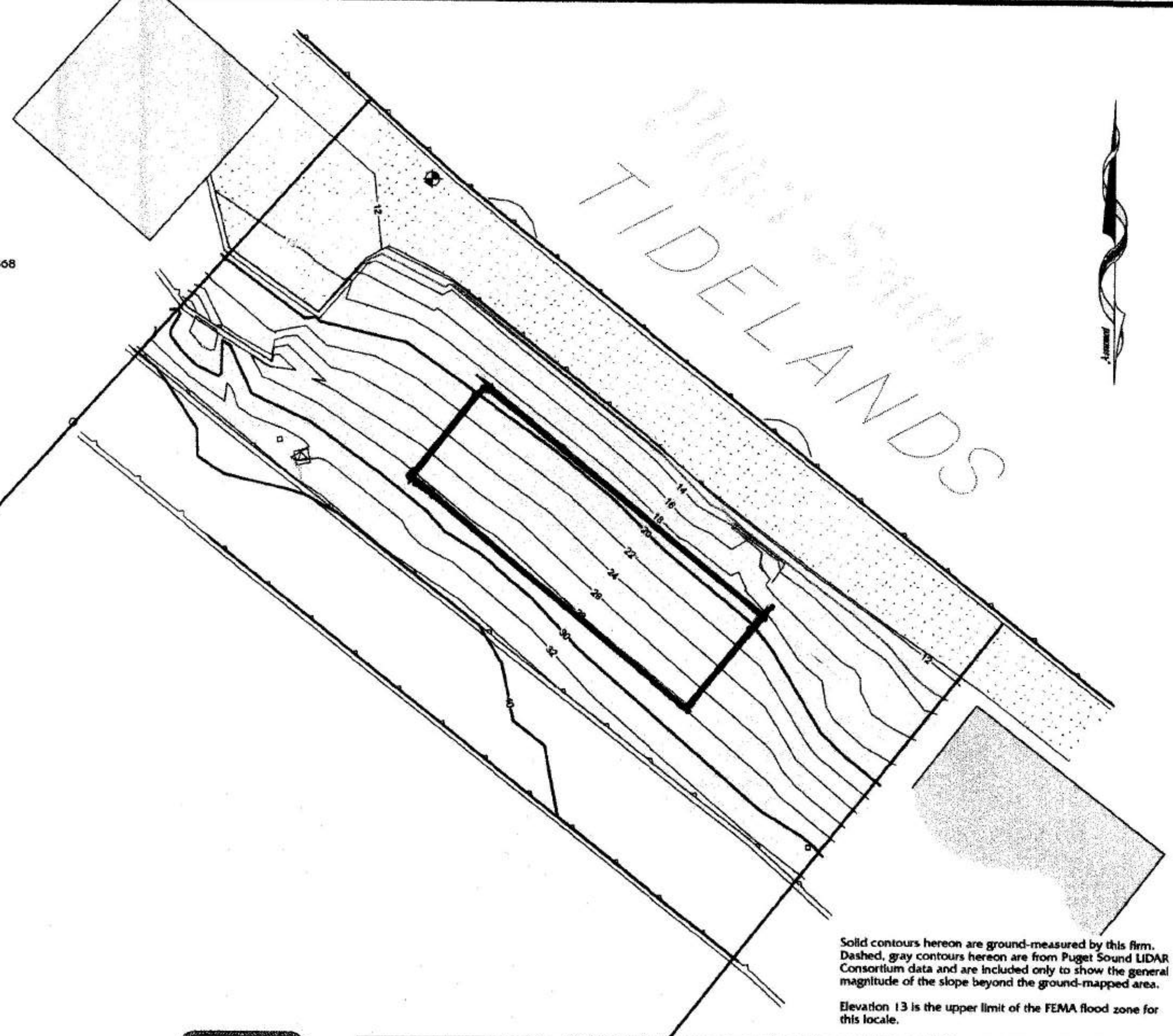
Total credits 3.5, 2.5 credits required

Table 2

⇩	Option	Description	Credit(s)
	1a	EFFICIENT BUILDING ENVELOPE 1a: Prescriptive compliance is based on Table R402.1.1 with the following modifications: Vertical fenestration U = 0.28 Floor R-38 Slab on grade R-10 perimeter and under entire slab Below grade slab R-10 perimeter and under entire slab **or** Compliance based on Section R402.1.4: Reduce the Total UA by 5%.	0.5
	1b	EFFICIENT BUILDING ENVELOPE 1b: Prescriptive compliance is based on Table R402.1.1 with the following modifications: Vertical fenestration U = 0.25 Wall R-21 plus R-4 Floor R-38 Basement wall R-21 int plus R-5 ci Slab on grade R-10 perimeter and under entire slab Below grade slab R-10 perimeter and under entire slab **or** Compliance based on Section R402.1.4: Reduce the Total UA by 15%.	1.0
	1c	EFFICIENT BUILDING ENVELOPE 1c: Prescriptive compliance is based on Table R402.1.1 with the following modifications: Vertical fenestration U = 0.22 Ceiling and single-rafter or joist-vaulted R-49 advanced Wood frame wall R-21 int plus R-12 ci Floor R-38 Basement wall R-21 int plus R-12 ci Slab on grade R-10 perimeter and under entire slab Below grade slab R-10 perimeter and under entire slab	2.0

⇩	Option	Description	Credit(s)
		or Open loop water source heat pump with a maximum pumping hydraulic head of 150 feet and minimum COP of 3.6 To qualify to claim this credit, the building permit drawings shall specify the option being selected and shall specify the heating equipment type and the minimum equipment efficiency.	
	3d[b] 2 mini splits 1 ea floor	HIGH EFFICIENCY HVAC EQUIPMENT 3d: Ductless Split System Heat Pumps, Zonal Control: In homes where the primary space heating system is zonal electric heating, a ductless heat pump system shall be installed and provide heating to the largest zone of the housing unit. To qualify to claim this credit, the building permit drawings shall specify the option being selected and shall specify the heating equipment type and the minimum equipment efficiency.	1.0
	4	HIGH EFFICIENCY HVAC DISTRIBUTION SYSTEM: All heating and cooling system components installed inside the conditioned space. This includes all equipment and distribution system components such as forced air ducts, hydronic piping, hydronic floor heating loop, convectors and radiators. All combustion equipment shall be direct vent or sealed combustion. For forced air ducts: A maximum of 10 linear feet of return ducts and 5 linear feet of supply ducts may be located outside the conditioned space. All metallic ducts located outside the conditioned space must have both transverse and longitudinal joints sealed with mastic. If flex ducts are used, they cannot contain splices. Flex duct connections must be made with nylon straps and installed using a plastic strapping tensioning tool. Ducts located outside the conditioned space must be insulated to a minimum of R-8. Locating system components in conditioned crawl spaces is not permitted under this option. Electric resistance heat and ductless heat pumps are not permitted under this option. Direct combustion heating equipment with AFUE less than 80% is not permitted under this option. To qualify to claim this credit, the building permit drawings shall specify the option being selected and shall specify the heating equipment type and shall show the location of the heating and cooling equipment and all the ductwork.	1.0
	5a	EFFICIENT WATER HEATING 5a: All showerhead and kitchen sink faucets installed in the house shall be rated at 1.75 GPM or less. All other lavatory faucets shall be rated at 1.0 GPM or less.c To qualify to claim this credit, the building permit drawings shall specify the option being selected and shall specify the maximum flow rates for all showerheads, kitchen sink faucets, and other lavatory faucets.	0.5
	5b	EFFICIENT WATER HEATING 5b: Water heating system shall include one of the following: Gas, propane or oil water heater with a minimum EF of 0.74 **or** Water heater heated by ground source heat pump meeting the requirements of Option 3c. **or** For R-2 occupancy, a central heat pump water heater with an EF greater than 2.0 that would supply DHW to all the units through a central water loop insulated with R-8 minimum pipe insulation. To qualify to claim this credit, the building permit drawings shall specify the option being selected and shall specify the water heater equipment type and the minimum equipment efficiency.	1.0

SINCE 1976



Legend

- Found ¾" iron pipe with surveyor's i.d. cap no. 21368
- Project benchmark: Steel spike; Elevation: 11.39
- Building
- Concrete flatwork
- Retaining wall
- Pad-mounted power transformer
- Telephone riser
- 0-15% slope
- 15-20% slope
- 20-40% slope
- 40-100% slope

DATUM
NAVD88 2011
SmartNet GPS RTN

10 0 5 10 20 40
1" = 10'

12/8/2016

Solid contours hereon are ground-measured by this firm. Dashed, gray contours hereon are from Puget Sound LIDAR Consortium data and are included only to show the general magnitude of the slope beyond the ground-mapped area.

Elevation 13 is the upper limit of the FEMA flood zone for this locale.

MACLEARNSBERRY, Inc.
LAND SURVEYORS • CONSULTANTS
1100 NW Thompson Road, Suite 301, Poulsbo, WA 98370
phone: 206 427-0506
www.sealandsurvey.com

Client	**Margaret Dufresne**		
	Rolling Bay Walk **Topographic Survey & Slope Analysis**		
Drawn by: BAM	Date December 5, 2016		Job No. 16057
Checked by: BAM	Scale 1" = 10'		Sheet 1 of 1

BUILDING SITE APPLICATION (BSA)
BUILDING CLEARANCE (BC)

KITSAP PUBLIC HEALTH DISTRICT

BUILDING DEPARTMENT OFFICIAL COPY

** Please keep this notice attached to application.**

Submit this attached green-stamped Official approved application provided by KPHD, with an additional photocopy that you provide to your local Building Department when applying for your Building Permit.

Please note that there is a $10.00 fee to replace this Official application for Building Permit submittal.

For DCD Purposes: **Concurrent Review: Yes/No**

BSA New/Alt Revised expiration date **BC/BC Revision** **BC Exemption**

BSA Revision/Redesign **BC Sewer**

K:\EH\Onsite\Applications and Forms\In-house forms\Approval Notices\OSS_BSA-BCApprovedBUILDINGDEPTCopy.pub

KITSAP PUBLIC
HEALTH DISTRICT

345 6th Street, Suite 300
Bremerton, WA 98337
360-728-2235

HEALTH OFFICER DECISION

Memo Number: 318627
RP ACCT ID: 2606333
Application Type: BSA- New
Expiration Date: 12/31/2017

SITE INFORMATION

Site Address - Street, City, Zip Code:	Assessor Tax Account Number:
11143 ROLLING BAY WALK NE, Bainbridge Island, WA 98110	41560010041006

APPLICANT INFORMATION

Name:	Phone Number:	E-Mail:
DUFRESNE MARGARET A		

Applicant Mailing Address - Street, City, State, Zip Code.
9335 22ND AVE NW, SEATTLE, WA 98117

CONTRACTOR OF RECORD

Company:	Phone Number:
MILLER BAY WATER COMPANY	(360) 598-3505

WAIVERS

Waiver Type	Memo Number	Notes
BSA- Waiver Class A	318628	Reviewed & approved by JK
BSA- Waiver Class A	318629	Reviewed & approved by JK

HEALTH OFFICER DECISION FOR ONSITE SEWAGE SYSTEM

	Name of Inspector:	Date:
Approved (See Conditions Below)	KERRIE Yanda	10/23/2014

Expiration date extended by 60 days to 12/31/2017

HEALTH OFFICER DECISION FOR WATER SYSTEM

	Name of Inspector:	Date:
Approved (See Conditions Below)	JOHN KIESS, R.S.	10/07/2014

Final Application Decision: Approved

Dufresne Single Family Residence
Site-Specific Impact Analysis Report

Date: February 28, 2017

Prepared for:
Margaret Dufresne
3912 Hwy 104
Poulsbo, Washington 98370

Site address:
11131 and 11143 Rolling Bay Walk NE
Bainbridge Island, Washington

MARINE SURVEYS & ASSESSMENTS
267 Hudson Street
Port Townsend WA 98368

(360) 385-4073

marine.surveys.inc@gmail.com

Contents

List of Figures

Mitigation and Monitoring Plan Figures

Dufresne Site Specific Analysis

All shoreline development, use and activities, regardless of whether a permit is required, must result in no net loss of ecological functions and processes necessary to sustain shoreline resources. To demonstrate that the no net loss standard is met, this project includes a Site-Specific Impact Analysis. This analysis and report identifies existing conditions and ecological functions, impacts from the project, avoidance measures, and proposed mitigation.

1. Project Description

The project proposal includes a 2-story Single Family Residence (SFR; including a garage) and a driveway within the 200 foot shoreline jurisdiction of Bainbridge Island (Figure 1 & 2). The proposed structure will be 60' wide (alongshore) x 20' deep. A 2' overhang on all sides of the building resulting in 64' x 24' of roof area and 1536 square feet of new impervious surface. A 8' x 60' driveway will also be built between the SFR and the existing paved private road. The driveway will be built from permeable pavers so it will not contribute to impervious surfaces, but will introduce 480 square feet of permanent impacts to vegetation.

Because a rain garden cannot be installed at the site due to topography (Figure 3), a planting plan is included with the project to mitigate for negative impacts from the new impervious surface (Appendix A).

Because high tides may overtop the bulkhead, the drainage plan for stormwater at the site includes overlaying the existing paved road surface with a positive slope from the top of the existing bulkhead to your building.

1.1 Location and Regulatory Jurisdiction

The project site is located along the northeastern shoreline of Bainbridge Island (Figure 1) at:

- 11131 and 11143 Rolling Bay Walk NE, Bainbridge Island, Washington;
- Kitsap County Parcels M142441988300 & M142441996300;
- 47.664087, -122.503108

The shoreline designation at this site is "Shoreline Residential." The property is located upland of a bulkhead and 10-foot wide private road and is considered functionally isolated from the shoreline, so ecological buffers do not apply. Typically, a buffer for this area would be 75 feet (Shoreline Residential - Developed Lots- Category A) with Zone 1 of the buffer extending 30 feet from the OHWM based on the nature of vegetation and ecological function present along the shoreline (CoBI 2014). While the shoreline buffers do not apply, the project is within the 200 foot setback for Shoreline Residential nonwater-oriented development and still must meet standards for No Net Loss of ecological function.

The entire property is 118’ wide (alongshore) x 162’ deep and the SFR development is proposed only on the 38’ x 118’ area of property along the shoreline road, which is downslope of a retaining wall and bluff. The entire property is within the 200’ shoreline jurisdiction.

1.2 Project Footprint

The proposed structure is a 2-story structure (with a garage on the lower level to minimize footprint size). The 8’ x 60’ driveway would be adjacent to the road (because it is a private road, building setbacks are not required). The 20’ x 60’ structure (i.e. including the 2-4’ overhang of the eaves) would be adjacent to the driveway, with the foundation 18’ from the top of the bulkhead.

The total new impervious surface of the project will be 1536 square feet and the total area of permanent vegetation removal resulting from the construction of the SFR and driveway will be 1680 square feet (approximately 8.8% of the area of the lot). The project also includes mitigation measures to compensate for these impacts: native plantings (covered in detail in Sections 4 & 5, and in the Mitigation and Planting Plan included in Appendix A).

1.3 Construction Methods

The staging area and access for construction will be the existing road and sideyard. Grading will be minimal and only as required (around the new building only). There will be minimal heavy equipment (a small backhoe for excavation and site preparation, bulldozer, and large trucks).

1.4 Construction Schedule

Construction will begin as soon as possible after all permitting and approvals are complete. Schedule to be determined by contractor and any potential construction work windows that may be included as conditions of the permit.

1.5 Project Implementation

The contractor is yet to be determined. Once a contractor is chosen, the name and contact information will be submitted to the Planning Department before project work begins.

1.6 Best Management Practices (BMPs)

- Contractor will follow Washington State erosion and sediment control BMP’s (Washington State Department of Ecology's Storm Water Management Manual For The Puget Sound Basin, 2012), and:
  - The contractor shall apply all measures necessary to prevent the discharge of sediment-laden water off the project site.
  - The contractor shall inspect and maintain all erosion control facilities regularly, particularly during and following large storms.

- All streets adjacent to this project shall be kept clean of all material deposits resulting from construction.
  - Site work shall be scheduled to minimize the exposure of disturbed soils. All disturbed areas shall be stabilized (clear plastic, mulching, etc.) as quickly as possible after completion of work in the area.
  - From October 1 through April 30, no soils shall remain exposed for more than 2 days. From May 1 through September 30, no soils shall remain exposed for more than 7 days.
  - Care shall be taken to prevent any discharge of sediment-laden water into the stormwater systems. Any inlets which receive runoff should be protected with sediment filters.
  - Petroleum products and other potential pollutants shall be protected to prevent their introduction into site runoff or the storm drainage system.
  - The contractor shall develop a dewatering plan to prevent sediment laden water from discharging to the beach.
  - Clearing limits, if shown, shall be clearly flagged prior to any clearing or construction on the site. During construction, no disturbance beyond the flagged clearing limits shall be permitted. The flagging shall be maintained by the contractor until all construction is approved.
  - All temporary erosion control facilities, including perimeter controls, shall remain in place until final site construction is complete and approval has been received from the city.
- Staging areas for equipment and materials will be located on existing paved street or existing paved turnaround area or within approved clearing limits.
- Soil will not be disturbed within the drip-line of existing trees to be retained. Within 3-feet of the tree drip-line, soil amendments should be incorporated no deeper than 3 to 4-inches to reduce damage to roots.
- Stormwater drainage design will include roof drains leading to a splash block and then onto the pavement, reducing risk of erosion on the site during storm events. The footing drains would also discharge on a splash block on the landward side of the house. (The footing drains may need to be pumped to accomplish this to get positive based on sea level in the area).
- Permeable material (pavers or concrete) will be used for the driveway, decreasing the area of new impervious surfaces.

2. Site Description

2.1 Site visit description and findings

The entire property is 118’ wide (alongshore) x 162’ deep and development is proposed only on the 38’ x 118’ area of property along the shoreline road, which is downslope of a retaining wall and bluff.

A site visit was performed by Marine Surveys & Assessments on February 18, 2017. The property was characterized as an undeveloped lot approximately 0.44 acres in size. The upland ecological functions are disconnected from the shoreline by a concrete drive and bulkhead. The portion of the lot between Rolling Bay Walk and the first of two wood enforcement walls (approximately 4,193 square feet) is covered mostly by invasive species (approximately 85%). Non-native invasive species include: English ivy *(Hedera helix)*, Himalayan blackberry *(Rubus armeniacus)*, butterfly bush *(Buddleia davidii)* and English laurel *(Prunus laurocerasus)*. Other, native vegetation includes: red alder *(Alnus rubra)*, sword fern *(Polystichum munitum)*, fringecup *(Tellima grandiflora)*, moss sp., grass sp. and unidentified herbaceous species (dormant during visit, See Figure A-2). Neighboring properties are developed, with single-family homes immediately adjacent to Rolling Bay Walk, within 12 feet of the OHWM.

2.2 Baseline environmental conditions

The shoreline at the project site is characterized by a seawall and a 10-foot wide paved private road along the top of the seawall. It is classified as estuarine, intertidal, mixed-coarse substrate, partly enclosed, eulittoral zone (WaDNR 2001). The project is located within drift cell KS-14-1, identified as a Bainbridge Island Drift Cell Restoration General Priority (second to lowest priority of 5 priority rankings) in a small stretch of shoreline that does not include feeder bluff activity but is within a High Bluff geomorphologic class (Ecology 2013, CoBI 2004).

The site is in the Rolling Bay - Point Monroe Management Area on Bainbridge Island, where overhanging riparian vegetation covers only approximately 29% of the shoreline with impervious surfaces (e.g., roads, roofs) representing 17% of the riparian zone land cover. Shoreline development in MA-3 is primarily residential in nature. Approximately 38% of shoreline is modified by armoring and 27% of the shoreline has armoring that encroaches into the intertidal zone (CoBI 2004).

Approximately the first 18 feet along the shoreline is an area designated as a FEMA Floodplain ("AE - SFHAS with high Flood Risk") and the upland area is comprised of slopes of <40% (Figure 6).

2.3 Local Species & Habitats

No streams, wetlands, liquefaction zones were identified near the action area (City of Bainbridge Island Critical Areas Web Application, 2017). The Washington Shorezone Inventory shows that patchy eelgrass has been documented along the shoreline near the project site, but no surfgrass, kelp, or salt marsh (WA DNR Shorezone 2001).

Commercial shellfish growing is prohibited along the shoreline adjacent to the project location and the Shellfish Harvest Area is classified as "prohibited" due to a nearby Wastewater Treatment Plant outfall (WA DOH 2017).

The Olive-sided Flycatcher (*Contopus cooperi*) and Spotted Owl (*Strix occidentalis*) are identified on International Union for Conservation of Nature and Natural Resources (IUCN) range maps for Threatened bird species occurring within the Puget Sound watershed, and the Northern Alligator Lizard (*Elgaria coerulea*) for Threatened reptile species (NOAA 2016).

2.4 State Species & Habitats

Surf smelt spawning activity has been identified on a number of beaches along Bainbridge Island, including just north of the project site (ERMA 2017, CoBI 2003). Forage Fish surveys have been conducted by WDFW along the shoreline near the project site, but no evidence of spawning has been documented. The shoreline is highly developed and armored with a seawall and is not suitable spawning habitat for surf smelt or sand lance. No herring spawning has been identified offshore. The WDFW Priority Habitats and Species web mapping application was queried for the site; no sensitive species or habitats were identified at the site or in the immediate area. Geoduck presence is indicated offshore of the site (WDFW 2017b).

2.5 Federal Species & Habitats

The marine area adjacent to the shoreline buffer is designated NMFS Marine Critical Habitat for the following Endangered Species Act federally- listed species:

- Final Nearshore Rockfish Critical Habitat (NMFS, 2014)
- Marine Critical Habitat for Puget Sound Chinook Salmon (NMFS, 2005)
- Southern Resident killer whales (J, K, and L pods) (NOAA 2006).

Marine Critical Habitat for Puget Sound Chinook Salmon also includes the upland areas of Bainbridge Island (ERMA 2017). Impacts analyses are included for Puget Sound Chinook Salmon, juvenile rockfish, and Killer Whale and are included in the following sections. No ESA listed species are indicated in the upland portion of the site or surrounding upland area.

Evolutionarily Significant Unit (ESU) Boundaries (NOAA 2016) show that the upland area of the Bainbridge Island, including the project footprint, is considered accessible habitat for:

- Puget Sound\Strait of Georgia Fall and Winter Chum Salmon (Protection Status: Not Warranted)
- Puget Sound\Strait of Georgia Coho Salmon (Protection Status: Species of Concern)
- Odd Year Pink Salmon (Protection Status: Not Warranted)
- Puget Sound Steelhead (Protection Status: Threatened)

However, no steelhead or other salmonid streams are documented in the area according to WDFW's Salmonid Stock Inventory (SaSI) (NOAA 2016).

According to USFWS, threatened bird species that may occur in the general area of the project location that could potentially be affected by activities in this location include the species below (USFWS 2016):

- Marbled Murrelet (*Brachyramphus marmoratus*)
- Streaked Horned Lark (*Eremophila alpestris strigata*)
- Yellow-billed Cuckoo (*Coccyzus americanus*)

Threatened Bull Trout (*Salvelinus confluentus*) may also occur in the marine environment of this area, but the project is not located within the final designated critical habitat for this species. Several migratory bird species (protected by the Migratory Bird Treaty Act and the Bald and Golden Eagle Protection Act) may occur in the area (USFWS 2016). A full report of USFWS' Information from Planning and Conservation (IPaC) Trust Resources Report can be found in Appendix B.

3. Impact Avoidance, Minimization, and Mitigation Measures

3.1 Mitigation Sequencing

Mitigation sequencing includes the steps taken during project planning and implementation that are meant to find the least environmentally damaging practicable alternative to achieve a project need. Demonstrate the application of required mitigation sequencing with a table or bulleted list of each mitigation action and how the proposal addresses it:

1. *Avoiding the impact altogether by not taking a certain action or parts of an action;*

   Considering the project goals and restrictions of placement, completely avoiding the impact to vegetation and surface permeability within the shoreline jurisdiction was not considered feasible.

2. *Minimizing impacts by limiting the degree or magnitude of the action and its implementation by using appropriate technology or by taking affirmative steps to avoid or reduce impacts;*

   The applicant has consulted with CoBI about the proposed building location and potential impacts and determined there is no alternative location to place a Single Family Residence due to site restrictions including the topography, shoreline restrictions, and site accessibility. As previously stated, the driveway will be paved with a permeable material or pavers, reducing the amount of new impervious area within the shoreline jurisdiction. Additionally, the total footprint of the structure has essentially been reduced by half by designing it as a 2-story building. The BMP's described in the project description also contribute towards minimizing the temporary impacts from construction.

3. *Rectifying the impact by repairing, rehabilitating, or restoring the affected environment;*

The impact area of the Single Family Residence will not be able to be restored; however, any areas affected by temporary impacts due to construction or grading can be returned to their previous state or better.

4. *Reducing or eliminating the impact over time by preservation and maintenance operations;*

A reduction of impacts over time within the footprint of the Single Family Residence and driveway will not occur.

5. *Compensating for the impact by replacing, enhancing, or providing substitute resources or environments;*

Compensatory mitigation is proposed for this project in the form of riparian planting with a ratio of over 1:1. The planting plan is included in Appendix A (Figure A-2).

6. *Monitoring the impact and the compensation project and taking appropriate corrective measures.*

Monitoring and corrective measures are outlined in detail in the planting plan (Appendix A).

4. Impacts of Project

Potential impacts are presented and evaluated here in the context of the City of Bainbridge Island Shoreline Master Program, Federal and State listed species, and Priority Habitats. While permanent impacts to ecological function will be addressed by measures developed through mitigation sequencing described in previous sections, some minor temporary impacts are likely to occur but will be minimized through BMP's.

Permanent Impacts – Vegetation Removal and Impervious Surfaces: The total new impervious surface from the construction of the Single Family Residence and driveway will be 1,536 square feet. Approximately 1,680 square feet of vegetation will be removed in the development of the site. Based on a site visit by MSA, most of the vegetation is non-native invasive species.

Impervious surfaces alter the flow rate, volume, and path that precipitation takes to the water, contributing to damage caused by increased volume and flow rates in stormwater run-off events. Stormwater runoff carries larger loads of fertilizers, pesticides and other pollutants to the marine environment than water that is allowed to drain naturally through soil and vegetation.

Temporary Impact - Water quality: Short term impacts to water quality may occur during construction with disturbance of loose soil and, in the event of rain, may include increased turbidity creating erosion and increased particulates in run off to the marine environment. Increased turbidity can have adverse effects on salmonids and juvenile rockfish; the impact level depends on duration of exposure, concentration of turbidity, the life stage during the increased exposure and the options available for the fish to avoid the plumes. For this project, the impacts are expected to be localized and brief and fish would likely avoid any areas of increased turbidity. Juvenile rockfish have a strong association with kelp and rocky substrate which is not present within the project area and, thus, these fish species are not expected to be impacted. Potential short-term construction-related water quality impacts will be avoided and minimized through stormwater BMP's. Long-term impacts to water quality from run-off related erosion will be minimized including roof drains that lead to a splash block and then onto the pavement, reducing risk of erosion on the site during storm events. The footing drains would also discharge on a splash block on the landward side of the house. Due to the functionally isolated location of the site from the shoreline, it is highly unlikely that Southern Resident Killer Whales, humpback whales, or leatherback sea turtles would be affected by water quality impacts related to the project.

Temporary Impact - Noise: Air noise levels will be increased during heavy equipment use and may have temporary behavioral impacts to birds and other wildlife, primarily avoidance of the area. In-water noise levels will not be affected. City noise ordinances will be observed.

5. Mitigation Summary and No Net Loss Analysis

The BMP's described in the project description and the additional minimization measures developed through mitigation sequencing are intended to prevent temporary and permanent project-related impacts to shoreline ecological functions and values in order to achieve City of Bainbridge Island No Net Loss criteria. However, permanent impacts to vegetation and natural drainage will occur.

Although rain gardens are the preferred mitigation approach for an increase to impervious surfaces within the shoreline jurisdiction, a rain garden was not feasible due to site topography. Other site constraints also precluded a raingarden to be situated 50' from steep slope, 50' from drainfield, property line and sufficient distance from sideyard setbacks. To reduce impervious surface at the site, Marine Surveys & Assessments recommended using permeable pavement or pavers for the driveway. To compensate for the balance of new impervious surface and vegetation impacts, the planting plan (Appendix A) includes installation of native plants. The proposed single-family residence will add 1,536 square feet of impervious surface. Approximately 1,680 square feet of mostly-invasive vegetation will be removed in the development of the site. Compensatory mitigation proposed with the planting plan would cover a total of 2,496 square feet (880 Sq Ft in zone 1 and 1,616 Sq Ft in zone 2) of with native plantings. The planting area is 38% larger than the area to be impacted by impervious surface and a multi-layered canopy of native plants will replace invasive species.

Through the avoidance and minimization measures and mitigation described herein, the proposed Dufresne Single Family Residence and driveway is expected to achieve No Net Loss for ecological function along the shoreline within the City of Bainbridge Island.

References

CoBI 2014. City of Bainbridge Island Shoreline Master Program Update. Ordinance Number 2014-04.

CoBI 2014a. City of Bainbridge Island Official Shoreline Designation Map. November 18, 2014.

CoBI 2003. Bainbridge Island Nearshore Assessment: Summary of Best Available Science. PNWD-3233. Prepared for the City of Bainbridge Island, Bainbridge Island, WA, by Battelle Marine Sciences Laboratory, Sequim, WA.

CoBI 2004. Bainbridge Island Nearshore Habitat Characterization & Assessment, Management Strategy Prioritization, and Monitoring Recommendations. Battelle Memorial Institute Richland, WA. November 2004.

CoBI 2010. Bainbridge Island Current and Historic Coastal Geomorphic/Feeder Bluff Mapping. Prepared for: City of Bainbridge Island Planning and Community Development. Prepared by: Coastal Geologic Services Inc. April 22, 2010.

CoBI 2012. Shoreline Restoration Plan - City of Bainbridge Island. Prepared for City of Bainbridge Island by Herrara. July 2012.

Cooke 1997. A Field Guide to the Common Wetland Plants of Western Washington & Northwestern Oregon. Sarah S. Cooke (Editor).

Ecology 2013. Department of Ecology: MacLennan., A. Johannessen, J.W., Williams, S.A., Gerstel, W., Waggoner, J.F., and Bailey, A., 2013, Feeder Bluff Mapping of Puget Sound, prepared by Coastal Geologic Services, Bellingham, for Washington Department of Ecology, Olympia WA, 117 pp and map folio.

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WDFW 2017. Washington Department of Fish and Wildlife Priority Habitats and Species report. Available at: http://wdfw.wa.gov/mapping/phs/. Olympia, Washington. Accessed 02/06/2017.

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USFWS 2017. IPaC Trust Resources Report. Generated 02/06/2017, IPaC v3.0.8

Figure 1. Dufresne Single Family Residence Vicinity Map

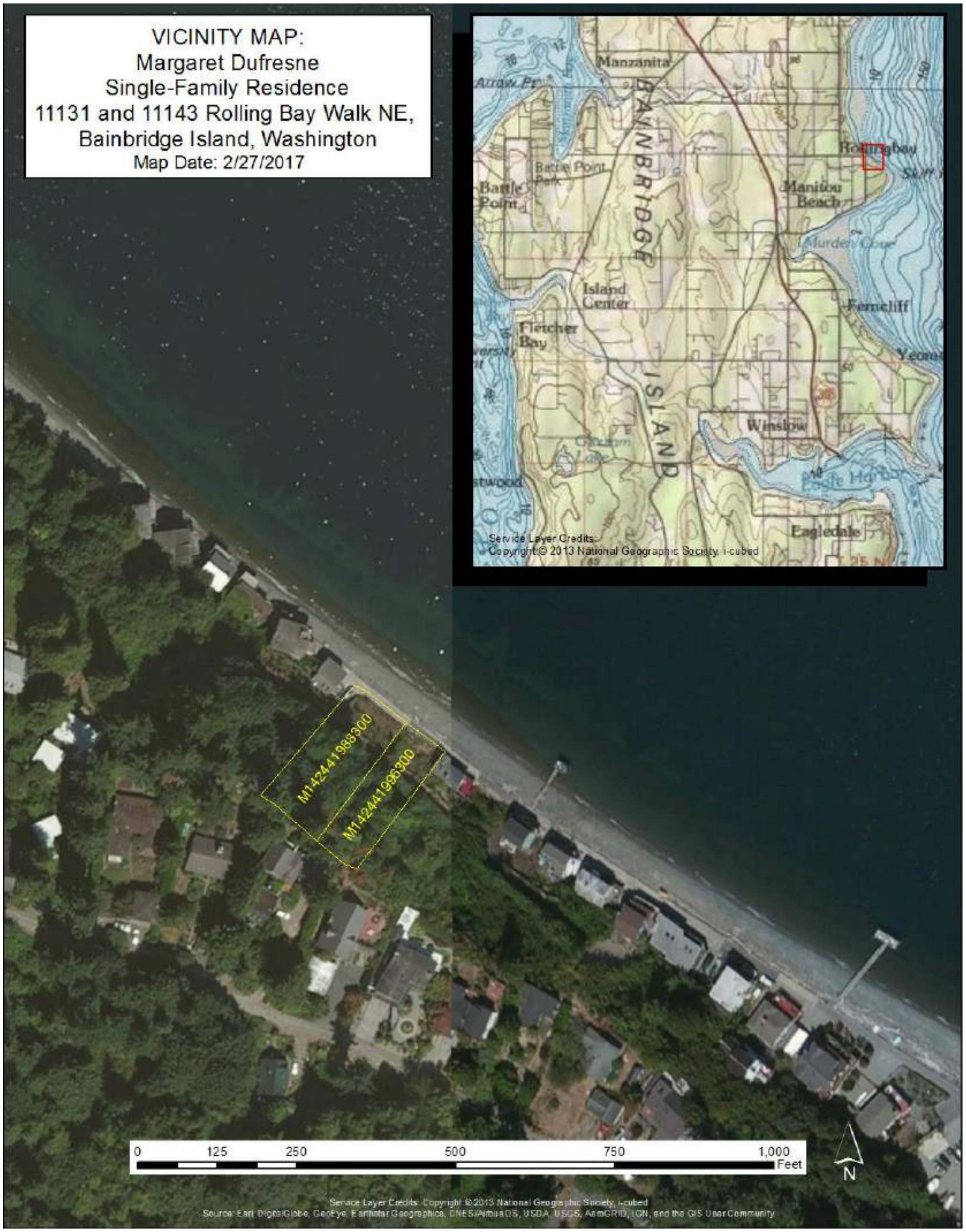


Figure 2. Dufresne Site Plan with impact areas

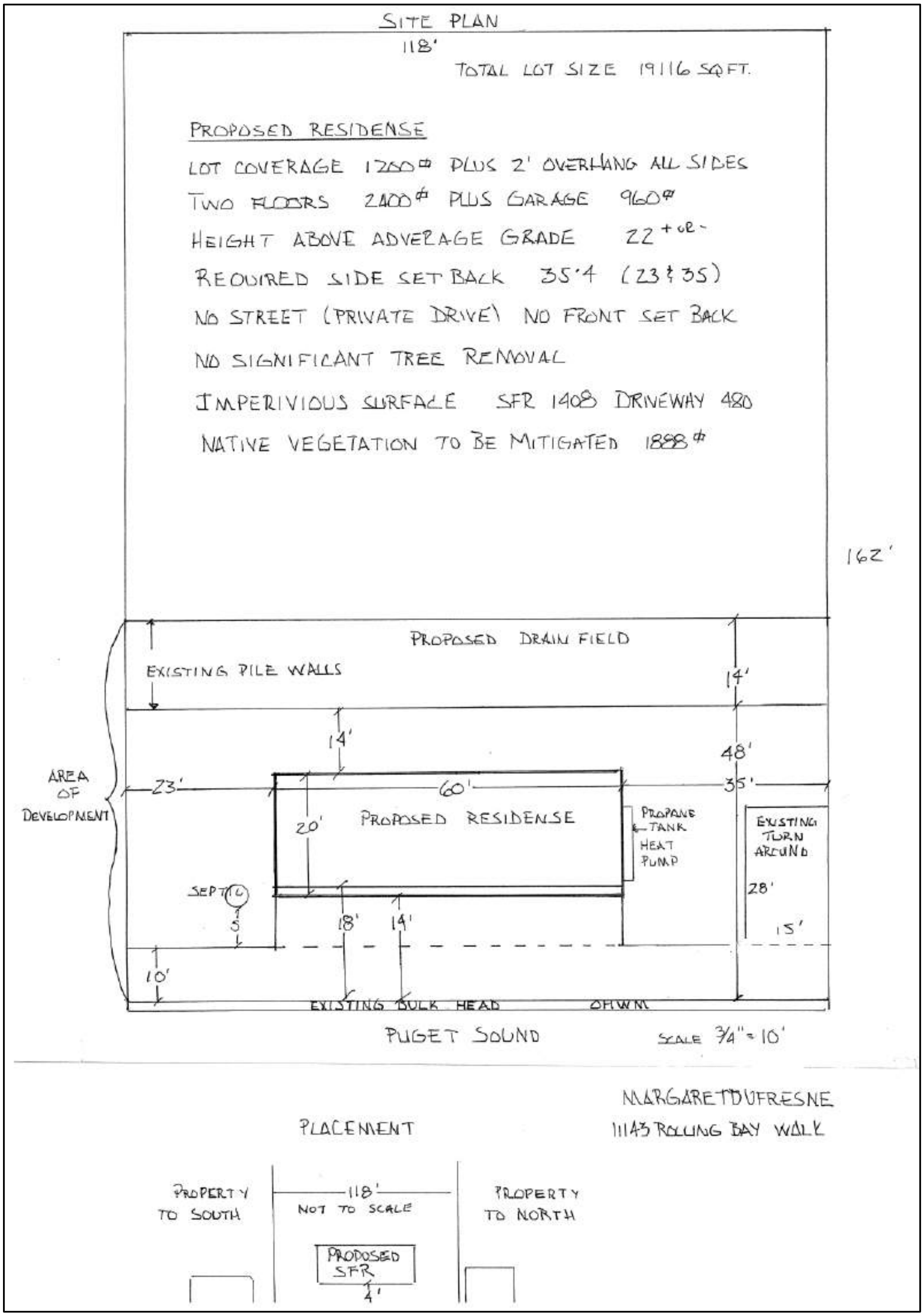


Figure 3. Profile view of Dufresne property and proposed structure.

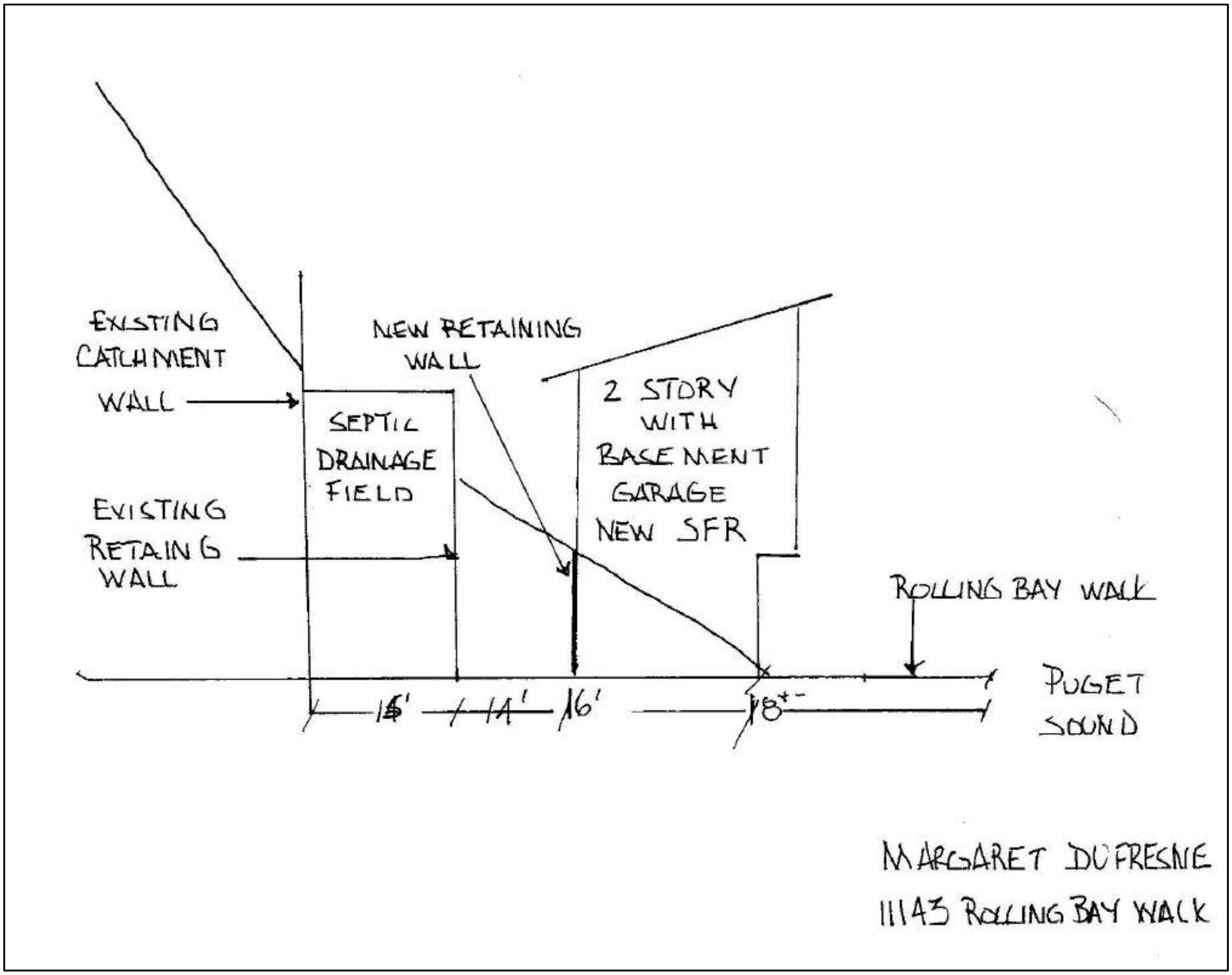


Figure 4. Official Shoreline Designation: Shoreline Residential

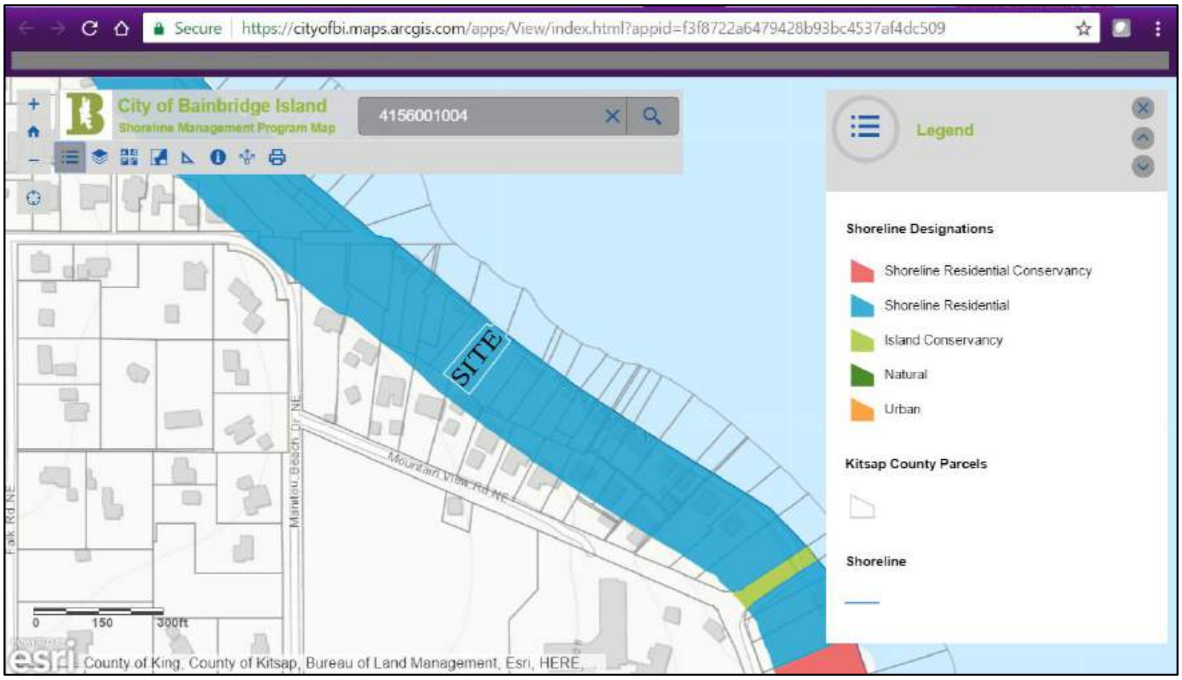


Figure 5. Site located outside Bainbridge Island Aquatic Conservancy Zones

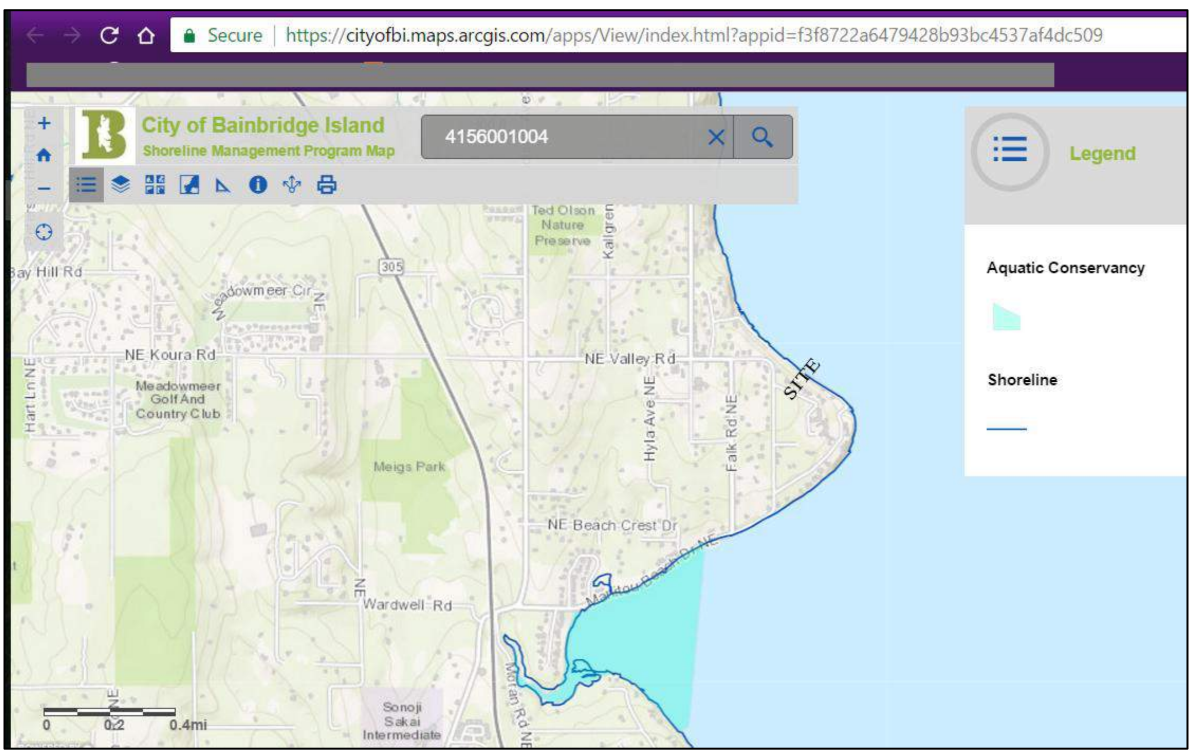


Figure 6. Geomorphic Shore Type

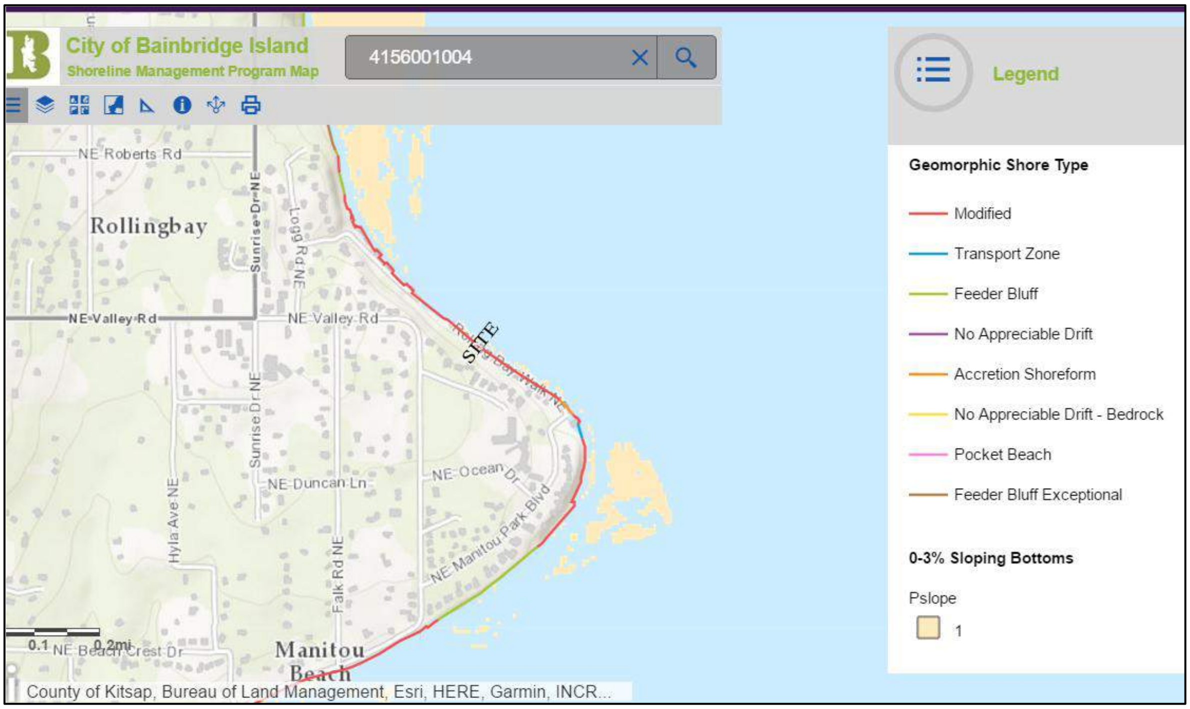


Figure 7. FEMA Flood Hazard, steep slopes, and documented landslides at and near project location.

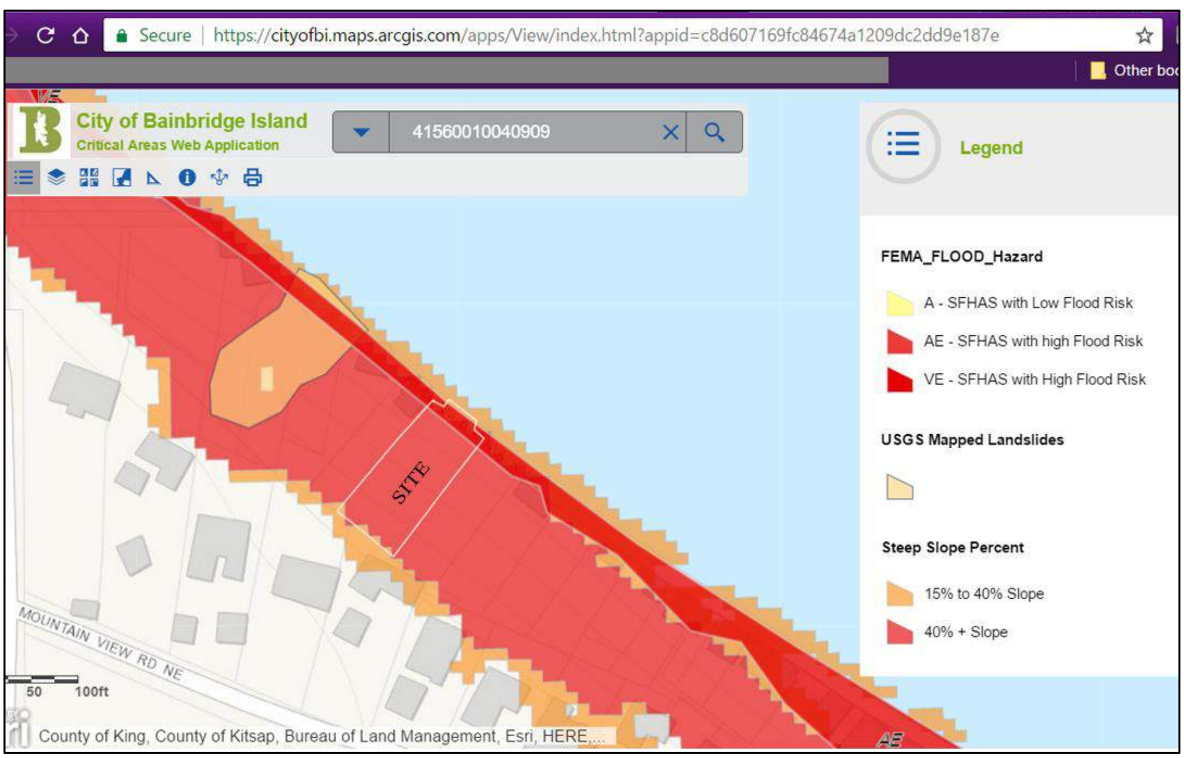
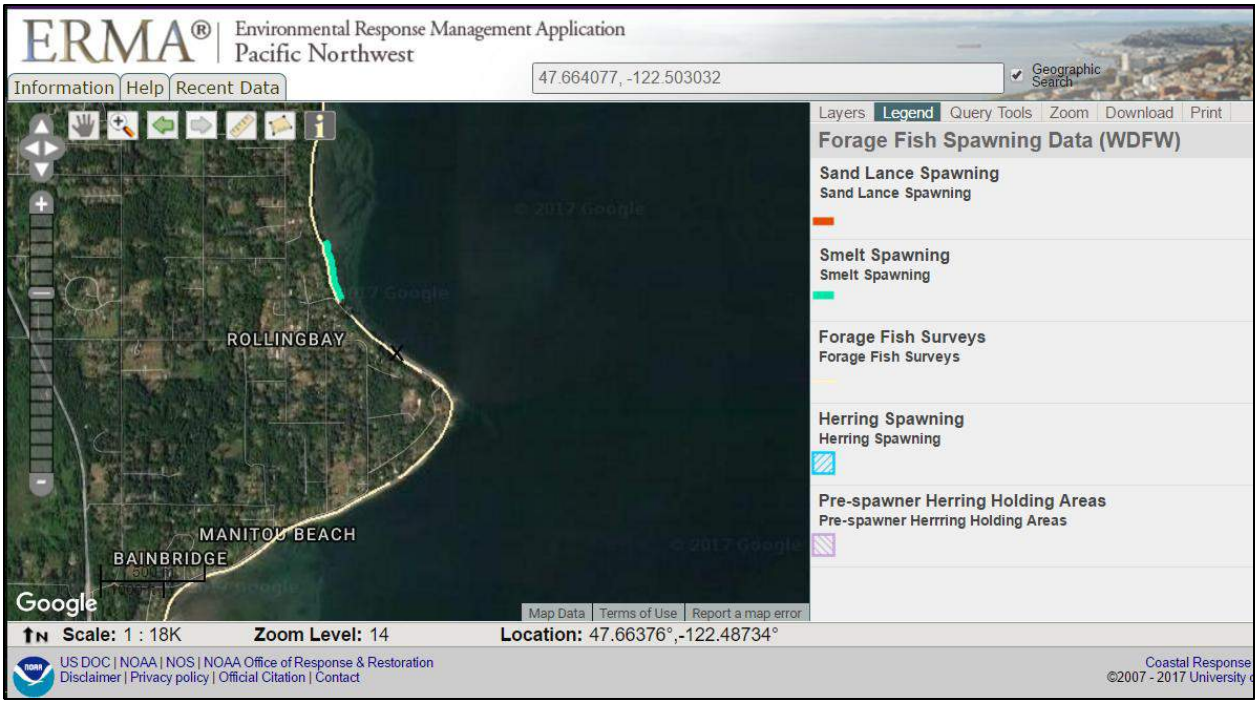


Figure 8. WDFW forage fish spawning habitat data



Appendix A: Dufresne Mitigation and Monitoring Plan

A. Site and Project Description

The property located at 11143 Rolling Bay Walk NE on Bainbridge Island is an undeveloped lot that is approximately 0.44 acres in size. The upland ecological functions are disconnected from the shoreline by a concrete drive and bulkhead. The portion of the lot between Rolling Bay Walk and the first of two wood enforcement walls (approximately 4,193 square feet) is covered mostly by invasive species (approximately 85%). Non-native invasive species include: English ivy *(Hedera helix)*, Himalayan blackberry *(Rubus armeniacus)*, butterfly bush *(Buddleia davidii)* and English laurel *(Prunus laurocerasus).* Other, native vegetation includes: red alder *(Alnus rubra)*, sword fern *(Polystichum munitum)*, fringecup *(Tellima grandiflora)*, moss sp., grass sp. and unidentified herbaceous species (dormant during visit). Neighboring properties are developed, with single-family homes immediately adjacent to Rolling Bay Walk, within 12 feet of the OHWM.

B. Mitigation Plan

Mitigation is required for this site, as the proposed structure will be built within the buffer on a shoreline of statewide significance. The proposed single-family residence will add 1,536 square feet of impervious surface. Approximately 1,680 square feet of vegetation will be removed in the development of the site. Most of the vegetation is non-native invasive species with the exception of some sword fern and a clump of very small red alder (Figure A-1).

As per the City of Bainbridge Island Single Family Residence Shoreline Mitigation Manual, the clearing of vegetation and the addition of impervious surfaces are impacts requiring mitigation. Mitigation measures include removal of existing impervious surface and replanting, or creating a rain garden to compensate for new impervious surface and planting multistoried native vegetation within the shoreline buffer to replace the functions lost by removing vegetation. As there is no existing impervious surface to remove and the lot is too small to support a rain garden, a native planting plan is proposed. A total of approximately 2,496 square feet (880 sf in zone 1 and 1,616 sf in zone 2) of native plant installation is proposed. The planting area is 38% larger than the area to be impacted by impervious surface. A multi-layered canopy of native plants will replace invasive species.

The plants selected for this site are all native plants that, once established will improve the overall value of the functions in this critical area buffer. Soil stability, nutrient input, and wildlife habitat were all important factors. Following is a table showing the plant species and numbers for the planting areas. Plants will be selected from a regional native plant nursery.

Plant List

Quantity	**Botanical Name**	**Common Name**	**Size**
2	*Acer circinatum*	vine maple	1 Gal Pots
Approx. 274 on 3' Centers	*Fragaria chiloensis*	coastal strawberry	Plugs or 3.5" Pots
6	*Holodiscus discolor*	ocean spray	1 Gal Pots
7	*Mahonia aquifolium*	tall Oregon grape	3.5" Pots
19 on 3' Centers	*Polystuchum munitum*	sword fern	3.5" Pots
32 on 4' Centers	*Rosa nutkana*	Nootka rose	1 Gal Pots
64 on 4' Centers	*Symphoricarpos albus*	snowberry	1 Gal Pots or Live Stakes

Specifications outlined in the drainage plan regarding soil amendments shall be followed:

- Areas to be disturbed shall have the topsoil stripped and stockpiled. All disturbed areas that are to be landscaped shall have the stockpiled soil replaced or shall be amended with compost.
- Areas where the topsoil will be replaced shall have the subsoils scarified to 4 inches and a minimum depth of 8 inches of topsoil shall be placed on top of subsoils. 1-inch of composted material shall be tilled into the top 4 inches of topsoil.
- Amending the subsoil shall consist of scarifying the top 8-inches of soil and tilling in the amount of compost described below.

- Landscaped areas (10% organic content): Place and till 3-inches of composed material into top 5-inches of soil. Rake beds smooth, remove rocks larger than 2-inches in diameter and mulch areas with 2-inches of organic mulch.
- Mature compost shall be Grade A compost meeting WSDOT STD Spec 9-14.4(8).

Plants should be installed in late fall or early spring following the construction work. During these times plants are semi-dormant and soils are easier to work. Plants will be laid out by hand generally following the spacing specified on the planting plan map (Figure A-2). The plants will be installed by digging a one to two foot hole, loosening the soil, placing plant in ground after loosening soil around root ball. The hole must be deep enough to ensure that roots are straight, but not so deep as to bury plants too far above the root collar. Once the plant is in place the hole will be backfilled and tamped lightly. Mulch should be applied 3" deep around plants, being careful not to touch stem of plant.

No extraordinary measures are proposed at this time to protect the installed plants other than mulching, weeding and watering. Substitutions might be necessary for species or individuals that cannot be found at local nurseries. All plant substitutions will be approved by the project biologist prior to installation to ensure their suitability for the site.

C. Performance Standards

Performance standards are measurable criteria for determining if the goals and objectives of the mitigation project are being achieved. If the proposed benchmarks are not achieved by comparing the surveys to the mitigation goals, then contingency plans will need to be implemented.

Performance Standard # 1 (survival rate): Immediately after planting, all plants will be counted and documented. At the end of each growing season (late Aug- early Sept) plots will be visited and a count of surviving plants will be documented. The percent survival for the plots will be calculated by dividing the total number of plants after planting by the total number of surviving plants at the end of the season. Photo stations for each replanting site will be determined and a photograph of each transplant location will be taken on an annual basis. Individual plants that die must be replaced with native species in order to meet the survival performance standards.

Performance Standard # 2 (percent cover): The percent cover standard will be monitored by looking at each monitoring unit of the enhanced areas from above and estimating the area covered by the individual species. The percent cover within an area can be quantified as a total greater than 100% because plants (in tree, high/low shrub and herbaceous layers) overlap in cover.

Performance Standard #3 (invasive removal): All areas where invasive plants were removed will be surveyed visually and categorized with photo stations. This is to ensure that 0% (none) of the targeted Invasive species (English Ivy, Himalayan Blackberry and Butterfly Bush) will be present and have not reestablished within each monitoring year.

D. Monitoring Plan

An as-built drawing and report will be submitted as documentation of the implementation of the approved planting plan within one month of installation. The plan will include vegetation description and photo documentation from established photo stations. A panoramic photo of the entire mitigation site will also be provided. Photos should be taken June - August, during the growing season. Monitoring will take place over a period of five years at the end of the growing season (late August or early September) of each monitoring year. The performance standards will be monitored by measuring plots in zones within the planting area that will be established and mapped after planting occurs, on the as-built plan. There will be photo points for each plot and they will be referenced on the as-built plan. Each year, the photo points that are established at each site, will be used for comparison. Photos will be taken at all points for all years as visual documentation of the performance standards progress, or lack of. In addition to photos at designated points, photo documentation must include a panoramic view of the entire planting area. Submitted photos must be formatted on standard 8 1/2 ″ by 11″ paper, dated with the date the photo was taken, and clearly labeled with the direction from which the photo was taken. The photo location points must be identified on an appropriate drawing. Collected data and photos will be compiled into an annual Riparian Planting report each year and submitted by November 30 of each monitoring year for five years. Each annual monitoring report shall include written and photographic documentation on plant mortality and replanting efforts and must document whether the performance standards are being met. Monitoring results will determine whether or not contingency measures will be needed.

Performance Standards #1 & 2

Year 1: Achieve 100% survival success of replanted natives into mitigation areas
Year 2: Achieve 80% survival success at end of second year into mitigation areas
Year 3: Achieve 80% survival success at end of third year into mitigation areas
Year 4: Achieve 80% survival success at end of fourth year into mitigation areas
Year 5: Achieve 80% survival success at end of fifth year into mitigation areas

Performance Standard #3

Years 1-5: Achieve 100% removal of non-native invasive species.

E. Maintenance and Contingency

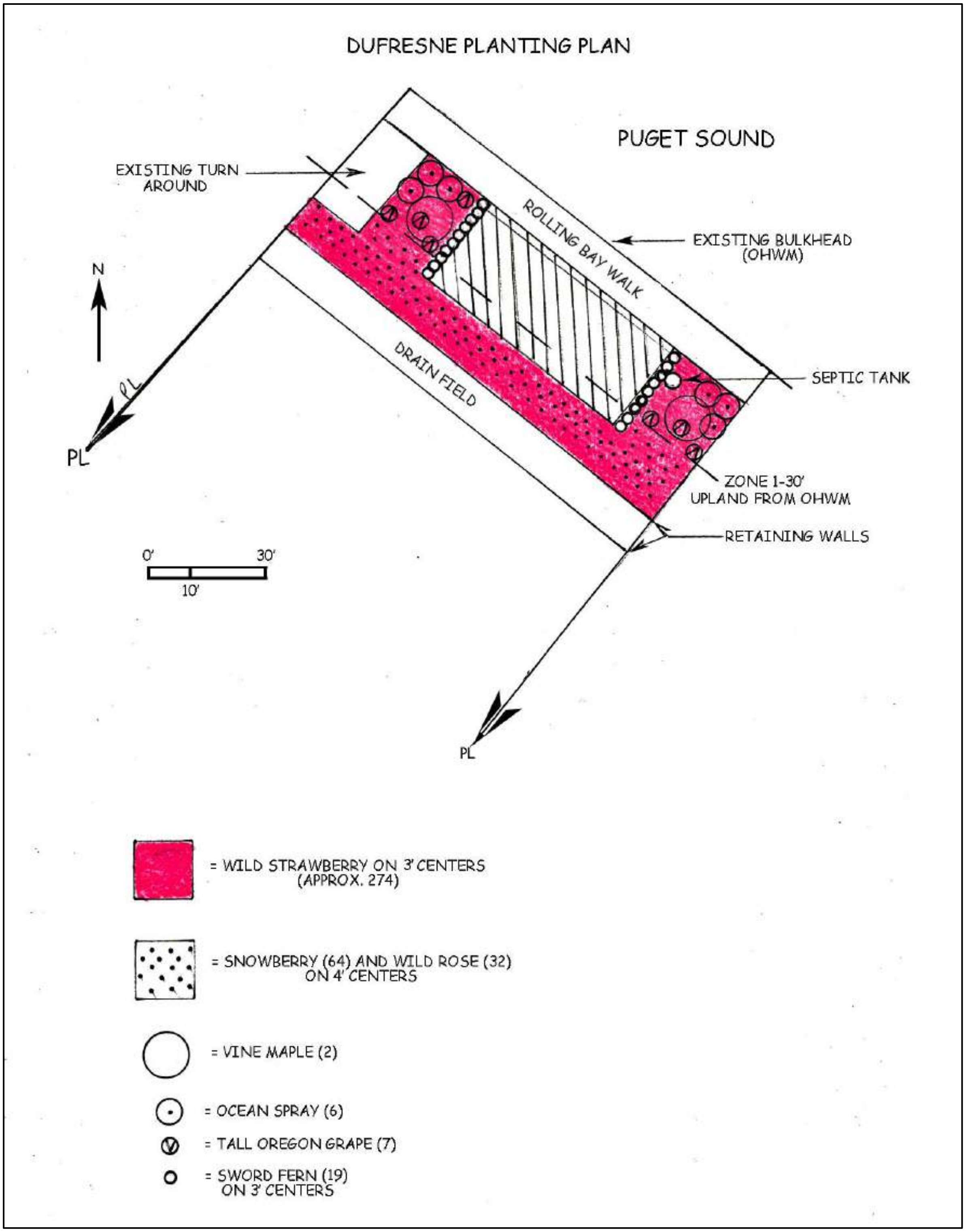
Maintenance shall occur at least twice during the growing season to ensure the survival of all native species within the mitigation area, including volunteer natives. Watering by hand or sprinkler may be necessary during year number one until the plants are established. Water requirements will depend on the timing of planting with the seasons and weather conditions. Once plants are established, extra watering may not be necessary. Hand weeding will be necessary around all plants that are being monitored for survival and coverage. If the required survival rate is not met by the end of any monitoring year, plants lost to mortality will be replaced to achieve the percentage cover performance standard described above. Prior to replacement, an appropriate assessment will be performed to determine if the survival was affected by species/site selection, animal damage, or some other factor. Subsequent contingency actions must be designed to respond directly to the stressor(s), which are increasing mortality of planted native species. If a particular species is shown not to endure site conditions then another, more appropriate species will be selected. If excessive damage is observed, protective measures will be introduced. Monitoring years may be added if significant re-planting becomes necessary. Monitoring on an annual basis for five years will occur with photographs to determine the survival rate of the transplanted area. If 100% success is achieved before reaching the five-year mark, monitoring will continue without extra replanting efforts. Within the five year time period, transplanting will occur on an annual basis to replace any plants that are lost until 80% success is achieved.

Figure A-1. Site Photos

Building Site and Property to NW

From Building Site Looking at Rolling Bay Walk to the East

Figure A-2. Planting Plan Map



Appendix B: USFWS’ Information from Planning and Conservation (IPaC) Trust Resources Report

IPaC **U.S. Fish & Wildlife Service**

IPaC resource list

Location

Kitsap County, Washington

Local office

Washington Fish And Wildlife Office

(360) 753-9440
(360) 753-9405

510 Desmond Drive Se, Suite 102
Lacey, WA 98503-1263

http://www.fws.gov/wafwo/

Endangered species

This resource list is for informational purposes only and should not be used for planning or analyzing project level impacts.

Section 7 of the Endangered Species Act **requires** Federal agencies to *"request of the Secretary information whether any species which is listed or proposed to be listed may be present in the area of such proposed action"* for any project that is conducted, permitted, funded, or licensed by any Federal agency.

A letter from the local office and a species list which fulfills this requirement can only be obtained by requesting an official species list either from the Regulatory Review section in IPaC or from the local field office directly.

For project evaluations that require USFWS concurrence/review, please return to the IPaC website and request an official species list by creating a project and making a request from the Regulatory Review section.

Listed species[1] are managed by the Endangered Species Program of the U.S. Fish and Wildlife Service.

1. Species listed under the Endangered Species Act are threatened or endangered; IPaC also shows species that are candidates, or proposed, for listing. See the listing status page for more information.

The following species are potentially affected by activities in this location:

Birds

NAME	STATUS
Marbled Murrelet Brachyramphus marmoratus There is a **final** critical habitat designated for this species. Your location is outside the designated critical habitat. http://ecos.fws.gov/ecp/species/4467	Threatened

Streaked Horned Lark Eremophila alpestris strigata There is a **final** critical habitat designated for this species. Your location is outside the designated critical habitat. http://ecos.fws.gov/ecp/species/7268	Threatened
Yellow-billed Cuckoo Coccyzus americanus There is a **proposed** critical habitat for this species. Your location is outside the proposed critical habitat. http://ecos.fws.gov/ecp/species/3911	Threatened

Fishes

NAME	STATUS
Bull Trout Salvelinus confluentus There is a **final** critical habitat designated for this species. Your location is outside the designated critical habitat. http://ecos.fws.gov/ecp/species/8212	Threatened

Critical habitats

Potential effects to critical habitat(s) in this location must be analyzed along with the endangered species themselves.

This location overlaps the critical habitat for the following species:

NAME	TYPE
Chinook Salmon Oncorhynchus (=Salmo) tshawytscha http://ecos.fws.gov/ecp/species/8091#crithab	Final designated
Chinook Salmon Oncorhynchus (=Salmo) tshawytscha http://ecos.fws.gov/ecp/species/8091#crithab	Final designated
Chinook Salmon Oncorhynchus (=Salmo) tshawytscha http://ecos.fws.gov/ecp/species/8091#crithab	Final designated
Chinook Salmon Oncorhynchus (=Salmo) tshawytscha http://ecos.fws.gov/ecp/species/8091#crithab	Final designated
Killer Whale Orcinus orca http://ecos.fws.gov/ecp/species/3380#crithab	Final designated

Migratory birds

Birds are protected under the Migratory Bird Treaty Act[1] and the Bald and Golden Eagle Protection Act[2].

Any activity that results in the take (to harass, harm, pursue, hunt, shoot, wound, kill, trap, capture, or collect, or to attempt to engage in any such conduct) of migratory birds or eagles is prohibited unless authorized by the U.S. Fish and Wildlife Service[3]. There are no provisions for allowing the take of migratory birds that are unintentionally killed or injured.

Any person or organization who plans or conducts activities that may result in the take of migratory birds is responsible for complying with the appropriate regulations and implementing appropriate conservation measures.

1. The Migratory Birds Treaty Act of 1918.
2. The Bald and Golden Eagle Protection Act of 1940.
3. 50 C.F.R. Sec. 10.12 and 16 U.S.C. Sec. 668(a)

Additional information can be found using the following links:

- Birds of Conservation Concern http://www.fws.gov/birds/management/managed-species/birds-of-conservation-concern.php
- Conservation measures for birds http://www.fws.gov/birds/management/project-assessment-tools-and-guidance/conservation-measures.php

- Year-round bird occurrence data http://www.birdscanada.org/birdmon/default/datasummaries.jsp

The migratory birds species listed below are species of particular conservation concern (e.g. Birds of Conservation Concern) that may be potentially affected by activities in this location, not a list of every bird species you may find in this location. Although it is important to try to avoid and minimize impacts to all birds, special attention should be made to avoid and minimize impacts to birds of priority concern. To view available data on other bird species that may occur in your project area, please visit the AKN Histogram Tools and Other Bird Data Resources.

NAME	SEASON(S)
Black Swift Cypseloides niger http://ecos.fws.gov/ecp/species/8878	Breeding
Peregrine Falcon Falco peregrinus http://ecos.fws.gov/ecp/species/8831	Breeding
Rufous Hummingbird selasphorus rufus http://ecos.fws.gov/ecp/species/8002	Breeding
Short-eared Owl Asio flammeus http://ecos.fws.gov/ecp/species/9295	Year-round
Western Grebe aechmophorus occidentalis http://ecos.fws.gov/ecp/species/6743	Wintering

What does IPaC use to generate the list of migratory bird species potentially occurring in my specified location?

Landbirds:

Migratory birds that are displayed on the IPaC species list are based on ranges in the latest edition of the National Geographic Guide, Birds of North America (6th Edition, 2011 by Jon L. Dunn, and Jonathan Alderfer). Although these ranges are coarse in nature, a number of U.S. Fish and Wildlife Service migratory bird biologists agree that these maps are some of the best range maps to date. These ranges were clipped to a specific Bird Conservation Region (BCR) or USFWS Region/Regions, if it was indicated in the 2008 list of Birds of Conservation Concern (BCC) that a species was a BCC species only in a particular Region/Regions. Additional modifications have been made to some ranges based on more local or refined range information and/or information provided by U.S. Fish and Wildlife Service biologists with species expertise. All migratory birds that show in areas on land in IPaC are those that appear in the 2008 Birds of Conservation Concern report.

Atlantic Seabirds:

Ranges in IPaC for birds off the Atlantic coast are derived from species distribution models developed by the National Oceanic and Atmospheric Association (NOAA) National Centers for Coastal Ocean Science (NCCOS) using the best available seabird survey data for the offshore Atlantic Coastal region to date. NOAANCCOS assisted USFWS in developing seasonal species ranges from their models for specific use in IPaC. Some of these birds are not BCC species but were of interest for inclusion because they may occur in high abundance off the coast at different times throughout the year, which potentially makes them more susceptible to certain types of development and activities taking place in that area. For more refined details about the abundance and richness of bird species within your project area off the Atlantic Coast, see the Northeast Ocean Data Portal. The Portal also offers data and information about other types of taxa that may be helpful in your project review.

About the NOAANCCOS models: the models were developed as part of the NOAANCCOS project: Integrative Statistical Modeling and Predictive Mapping of Marine Bird Distributions and Abundance on the Atlantic Outer Continental Shelf. The models resulting from this project are being used in a number of decision-support/mapping products in order to help guide decision-making on activities off the Atlantic Coast with the goal of reducing impacts to migratory birds. One such product is the Northeast Ocean Data Portal, which can be used to explore details about the relative occurrence and abundance of bird species in a particular area off the Atlantic Coast.

All migratory bird range maps within IPaC are continuously being updated as new and better information becomes available.

Can I get additional information about the levels of occurrence in my project area of specific birds or groups of birds listed in IPaC?

Landbirds:

The Avian Knowledge Network (AKN) provides a tool currently called the "Histogram Tool", which draws from the data within the AKN (latest,survey, point count, citizen science datasets) to create a view of relative abundance of species within a particular location over the course of the year. The results of the tool depict the frequency of detection of a species in survey events, averaged between multiple datasets within AKN in a particular week of the year. You may access the histogram tools through the Migratory Bird Programs AKN Histogram Tools webpage.

The tool is currently available for 4 regions (California, Northeast U.S., Southeast U.S. and Midwest), which encompasses the following 32 states: Alabama, Arkansas, California, Connecticut, Delaware, Florida, Georgia, Illinois, Indiana, Iowa, Kentucky, Louisiana, Maine, Maryland, Massachusetts, Michigan, Minnesota, Mississippi, Missouri, New Hampshire, New Jersey, New York, North, Carolina, Ohio, Pennsylvania, Rhode Island, South Carolina, Tennessee, Vermont, Virginia, West Virginia, and Wisconsin.

In the near future, there are plans to expand this tool nationwide within the AKN, and allow the graphs produced to appear with the list of trust resources generated by IPaC, providing you with an additional level of detail about the level of occurrence of the species of particular concern potentially occurring in your project area throughout the course of the year.

Atlantic Seabirds:

For additional details about the relative occurrence and abundance of both individual bird species and groups of bird species within your project area off the Atlantic Coast, please visit the Northeast Ocean Data Portal. The Portal also offers data and information about other taxa besides birds that may be helpful to you in your project review. Alternately, you may download the bird model results files underlying the portal maps through the NOAANCCOS Integrative Statistical Modeling and Predictive Mapping of Marine Bird Distributions and Abundance on the Atlantic Outer Continental Shelf project webpage.

Facilities

Wildlife refuges

Any activity proposed on National Wildlife Refuge lands must undergo a 'Compatibility Determination' conducted by the Refuge. Please contact the individual Refuges to discuss any questions or concerns.

THERE ARE NO REFUGES AT THIS LOCATION.

Fish hatcheries

THERE ARE NO FISH HATCHERIES AT THIS LOCATION.

Wetlands in the National Wetlands Inventory

Impacts to NWI wetlands and other aquatic habitats may be subject to regulation under Section 404 of the Clean Water Act, or other State/Federal statutes.

For more information please contact the Regulatory Program of the local U.S. Army Corps of Engineers District.

WETLAND INFORMATION IS NOT AVAILABLE AT THIS TIME

Data limitations

The Service's objective of mapping wetlands and deepwater habitats is to produce reconnaissance level information on the location, type and size of these resources. The maps are prepared from the analysis of high altitude imagery. Wetlands are identified based on vegetation, visible hydrology and geography. A margin of error is inherent in the use of imagery; thus, detailed on-the-ground inspection of any particular site may result in revision of the wetland boundaries or classification established through image analysis.

The accuracy of image interpretation depends on the quality of the imagery, the experience of the image analysts, the amount and quality of the collateral data and the amount of ground truth verification work conducted. Metadata should be consulted to determine the date of the source imagery used and any mapping problems.

Wetlands or other mapped features may have changed since the date of the imagery or field work. There may be occasional differences in polygon boundaries or classifications between the information depicted on the map and the actual conditions on site.

Data exclusions

Certain wetland habitats are excluded from the National mapping program because of the limitations of aerial imagery as the primary data source used to detect wetlands. These habitats include seagrasses or submerged aquatic vegetation that are found in the intertidal and subtidal zones of estuaries and nearshore coastal waters. Some deepwater reef communities (coral or tuberficid worm reefs) have also been excluded from the inventory. These habitats, because of their depth, go undetected by aerial imagery.

Data precautions

Federal, state, and local regulatory agencies with jurisdiction over wetlands may define and describe wetlands in a different manner than that used in this inventory. There is no attempt, in either the design or products of this inventory, to define the limits of proprietary jurisdiction of any Federal, state, or local government or to establish the geographical scope of the regulatory programs of government agencies. Persons intending to engage in activities involving modifications within or adjacent to wetland areas should seek the advice of appropriate federal, state, or local agencies concerning specified agency regulatory programs and proprietary jurisdictions that may affect such activities.

City of Bainbridge Island/
Bainbridge Island Fire Department

Bainbridge Island Fire Department
6895 Madison Avenue · Bainbridge Island, Washington 98110
(206) 842-7686 · Fax (206) 842-7695

Planning and Community Development
280 Madison Avenue N · Bainbridge Island, Washington 98110
(206) 842-2552 · Fax (206) 780-0955

NEW ADDRESS NOTIFICATION

Date: 12/20/2016

Dear Property Owner,

Based on information provided to the City of Bainbridge Island and the Bainbridge Island Fire Department the address indicated below will be your new Bainbridge Island Address. This letter is your Official confirmation you may begin to display the new address.

Applicant name: MARGARET DUFRESNE   Plat name: MANITOU PARK

Cross street: N/A

TAX ACCOUNT	TAXPAYER	NEW ADDRESS	UNIT TYPE
4156-001-004-1006	DUFRESNE MARGARET A	11143 ROLLING BAY WALK NE	VACANT

New address information has sent to the agencies identified below. Please be aware that there may be a delay in incorporating the information into their records. For many of the below listed agencies, especially the U. S. Postal Service, you may want to contact them directly to confirm the address and arrange for any of their services you may require.

If you have a question concerning the newly assigned address please contact the Addressing Specialist at the City of Bainbridge Island (206) 780-3782.

CC: Applicant, Bainbridge Island Fire Department - Addressing, Bainbridge Island Fire Department - Fire Marshall, City of Bainbridge Island, City of Bainbridge Island - Building Official, City of Bainbridge Island - Finance, City of Bainbridge Island - Utilities, County Treasurer Dept, KC Road Dept Engineering, Kitsap County Auditor / Elections, Kitsap County Elections Division, Puget Sound Engergy, USPS, BI Post Office, USPS, Seattle for COBI

#B106

PLUMBING PERMIT APPLICATION

☐ New ☐ Replacement ☒ Residential ☐ Commercial

Permit # **BLD** ______ Assessor's Account No: 4156-001-004-1006

Site Address: 11143 ROLLING BAY WALK

Owner's Name: Margaret Dufresne Phone: 206 491-3917

Owner Signature: Margaret Dufresne Date: Oct 30, 2017

Contractor: Margaret Dufresne Phone: 206 491-3917

License No: ______ Exp. Date: ______

Unit #	Fixture	Description	Base Fee	Totals
			ISSUANCE FEE:	**$28.20**
1	Water Heater	☒ Electric	$9.87	9.87
	Water Heater	☐ Propane. Propane Vent & Combustion Air	$15.02	
3	Water Closet	Toilet; Bidet or Urinal	$9.87	29.61
3	Bathtub or Shower	Tub with or without shower	$9.87	29.61
3	Bathroom Sink		$9.87	29.61
1	Dishwasher		$9.87	9.87
1	Kitchen Sink	With or without disposal	$9.87	9.87
1	Clothes Washer		$9.87	9.87
	Waste Interceptor		$9.87	
	Drinking Fountain		$9.87	
	Floor Sink/Floor Drains		$9.87	
	Utility Sink/Tray		$9.87	
	OTHER		$9.87	
	OTHER		$9.87	
			TOTAL:	156.51

Issued By: ______ **Date:** ______

Is this application for use in a Mobile/Manufactured Home?
If Yes: A permit from Washington State Department of Labor and Industries, **not** the City of Bainbridge Island, is required for appliances installed within a mobile/manufactured home.

Section 4 – Applicant/Property Owner Information

Property Owner:

Name: Margaret Dufresne Address: 3866 NE HWY 104, Poulsbo WA 98370

Contact Phone #: 206 491-3917 Email Address: margdof@yahoo.com

Applicant: Note: For projects with multiple owners, attach a separate sheet with each owner(s) information and signatures.

☐ Owner ☐ Applicant (other than owner) ☐ Authorized Agent/Representative*

Name: ______________________ Address: ______________________

Contact Phone #: ______________________ Email Address: ______________________

Contractor Washington State allows homeowners to be their own general contractor. However, when choosing a contractor or subcontractor to perform work they are required to be registered with the Washington State Department of Labor and Industries. For more information about choosing and hiring a contractor visit http://www.lni.wa.gov/tradeslicensing/.

☐ Check if this is the Authorized Agent/Representative* for this project.

Name: ______________________ Title: ______________________

License Number: ______________________ Liability Certificate: ______________________

Address: ______________________

Contact Phone #: ______________________ Email Address: ______________________

*I authorize the listed contractor to perform those inspections the City has identified in the self-certification program. (Residential projects only)

Margaret Dufresne ______________

Owner Signature Date

*The authorized agent/representative is the primary contact for all project-related questions and correspondence. The City will email requests and information about the application to the authorized agent/representative and will 'copy' (Cc) the owner noted below. The authorized agent/representative is responsible for communicating information to all parties involved with the application. It is the responsibility of the authorized agent/representative and owner to ensure their mailbox accepts City email (i.e., City email is not blocked or sent to "junk mail"). There may be instances where regular USPS or courier mail is used.

I affirm, under penalty of perjury, that all answers, statements, and information submitted with this application are correct and accurate to the best of my knowledge. I also affirm that I am the owner of the subject site. Further, as owner, I grant permission to any and all employees and representative of the City of Bainbridge Island and other governmental agencies to enter upon and inspect said property as reasonably necessary to process this application.

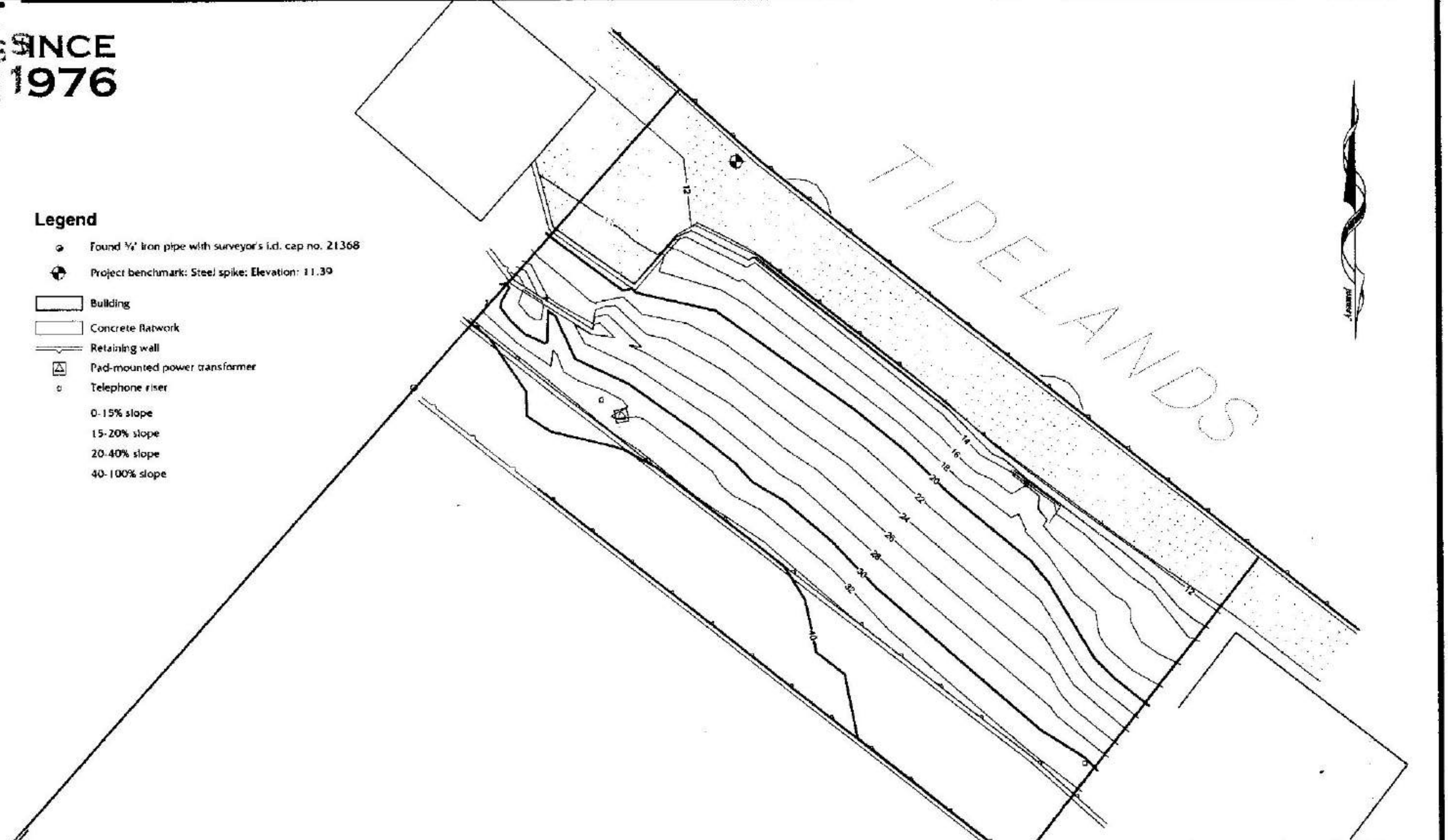
Margaret Dufresne Margaret Dufresne ______________

Print Name (Owner) Signature (Owner) Date

______________________ ______________________ ______________

Print Name (Owner) Signature (Owner) Date

Approvals	Initials	Date
Planning		
Building		
Drainage		
Other		



Solid contours hereon are ground-measured by this firm. Dashed, gray contours hereon are from Puget Sound LiDAR Consortium data and are included only to show the general magnitude of the slope beyond the ground-mapped area.

Elevation 13 is the upper limit of the FEMA flood zone for this locale.

DATUM

NAVD88 2011
SmartNet GPS RTN

10 0 5 10 20 40

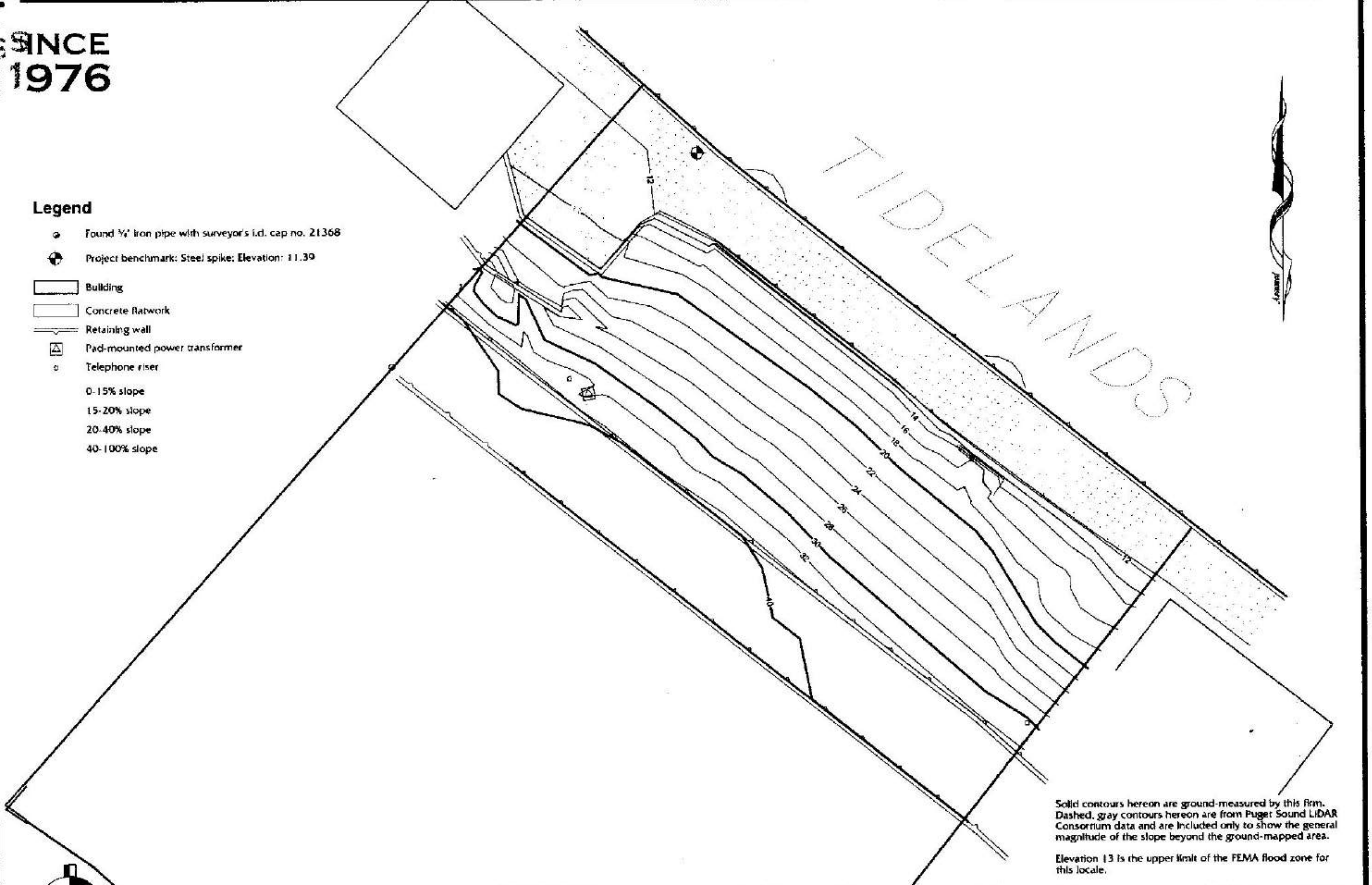
1" = 10'

12/5/2016

MACLEARNSBERRY, Inc.
LAND SURVEYORS • CONSULTANTS
1100 NW Thompson Road, Suite 301, Poulsbo, WA 98370
phone: 206 427-0506
www.sealandsurvey.com

Client	**Margaret Dufresne**	
Rolling Bay Walk		
Topographic Survey & Slope Analysis		
Drawn by: BAM	Date December 5, 2016	Job No. 16057
Checked by: BAM	Scale 1" = 10'	Sheet 1 of 1

SINCE 1976



Legend

- Found ¾" iron pipe with surveyor's i.d. cap no. 21368
- Project benchmark: Steel spike: Elevation: 11.39
- Building
- Concrete flatwork
- Retaining wall
- Pad-mounted power transformer
- Telephone riser
- 0-15% slope
- 15-20% slope
- 20-40% slope
- 40-100% slope

Solid contours hereon are ground-measured by this firm. Dashed, gray contours hereon are from Puget Sound LiDAR Consortium data and are included only to show the general magnitude of the slope beyond the ground-mapped area.

Elevation 13 is the upper limit of the FEMA flood zone for this locale.

DATUM

NAVD88 2011
SmartNet GPS RTN

10 0 5 10 20 40

1" = 10'

12/5/2016

MACLEARNSBERRY, Inc.
LAND SURVEYORS • CONSULTANTS
1100 NW Thompson Road, Suite 301, Poulsbo, WA 98370
phone: 206 427-0506
www.sealandsurvey.com

Client **Margaret Dufresne**		
Rolling Bay Walk **Topographic Survey & Slope Analysis**		
Drawn by: BAM	Date December 5, 2016	Job No. 16057
Checked by: BAM	Scale 1" = 10'	Sheet 1 of 1

KPUD
CONNECTING KITSAP

KITSAP PUBLIC UTILITY DISTRICT
1431 FINN HILL RD
PO BOX 1989
POULSBO, WA 98370
OFFICE 360-779-7656
FAX 360-779-3284

11/30/2016

MARGARET A DUFRESNE
3912 NE STATE HWY 104
POULSBO, WA 98370

Re:	Address:	LOT 4 ROLLING BAY WALK NE
	Tax ID #:	41560010041006
	Reference #:	23-10715-00
	Account #:	014738-000 & 014737-000
	Owner:	MARGARET A DUFRESNE
	Water System:	NORTH BAINBRIDGE

To Whom It May Concern:

This "Proof of Service" letter is meant to inform you that the above referenced property is currently served with a single water connection by PUD #1 of Kitsap County. This water