Application for a Site Plan Review

Municipality of Anchorage
Planning Department
PO Box 196650
Anchorage, AK 99519-6650

PETITIONER*	PETITIONER REPRESENTATIVE (IF ANY)
Name (last name first) Ship Creek Development, LLC	Name (last name first) Hoffman, Tony (The Boutet Company)
Mailing Address 1113 West Fireweed Lane	Mailing Address 601 E. 57th Place
Anchorage, AK., 99503	Anchorage, AK., 99518
Contact Phone: Day: 907-351-7825 Night:	Contact Phone: Day: 907-522-6776 Night:
FAX:	FAX:
E-mail: tmedmondson@thepetersengroup.biz	E-mail: thoffman@tbcak.com

*Report additional petitioners or disclose other co-owners on supplemental form. Failure to divulge other beneficial interest owners may delay processing of this application.

PROPERTY INFORMATION

Property Tax #(000-000-00-000): 002-071-27-000, 001-021-07-000

Site Street Address: 761 W. 2nd Ave, Anchorage, AK., 99501

Current legal description: (use additional sheet if necessary)

Lot 2, Block 122 and a portion of Lot 1,Block 122, United States Survey Number 408, as shown on the official U.S. Department of Interior, Bureau of Land Management supplemental plat filed March 22, 1985.

Zoning: PC SL	Acreage: 2.62	Grid # SW1230

SITE PLAN APPROVAL REQUESTED

☐ Special limitation ☐ Public facility ☐ Public facility project landscaping
☒ Other: Site plan approval

I hereby certify that (I am)(I have been authorized to act for) owner of the property described above and that I petition for a site plan review in conformance with Title 21 of the Anchorage Municipal, Code of Ordinances. I understand that payment of the application fee is nonrefundable and is to cover the costs associated with processing this application, and that it does not assure approval of the site plan. I also understand that assigned hearing dates are tentative and may have to be postponed by Planning Department staff, the Planning and Zoning Commission or Urban Design Commission for administrative reasons.

12/15/16	Tony Hoffman
Date	Signature (Agents must provide written proof of authorization)

Application for site plan review continued

Accepted by:	Poster & Affidavit:	Fee	Case Number
FM	2 + affidavit	$5,400	2017-0017

COMPREHENSIVE PLAN INFORMATION

Anchorage 2020 Urban/Rural Services: ☒ Urban ☐ Rural

Anchorage 2020 West Anchorage Planning Area: ☐ Inside ☒ Outside

Anchorage 2020 Major Urban Elements: Site is within or abuts:
☒ Major Employment Center ☒ Redevelopment/Mixed Use Area ☐ Town Center
☐ Neighborhood Commercial Center ☐ Industrial Center
☐ Transit - Supportive Development Corridor

Eagle River-Chugiak-Peters Creek Land Use Classification:
☐ Commercial ☐ Industrial ☐ Parks/opens space ☐ Public Land Institutions
☐ Marginal land ☐ Alpine/Slope Affected ☐ Special Study
☐ Residential at _______ dwelling units per acre

Girdwood- Turnagain Arm
☐ Commercial ☐ Industrial ☐ Parks/opens space ☐ Public Land Institutions
☐ Marginal land ☐ Alpine/Slope Affected ☐ Special Study
☐ Residential at _______ dwelling units per acre

ENVIRONMENTAL INFORMATION (All or portion of site affected)

Wetland Classification:	☒ None	☐ "C"	☐ "B"	☐ "A"	
Avalanche Zone:	☒ None	☐ Blue Zone	☐ Red Zone		
Floodplain:	☒ None	☐ 100 year	☐ 500 year		
Seismic Zone (Harding/Lawson):	☐ "1"	☐ "2"	☐ "3"	☒ "4"	☐ "5"

RECENT REGULATORY INFORMATION (Events that have occurred in last 5 years for all or portion of site)

☒ Rezoning - Case Number: A.O. 2014-123
☐ Preliminary Plat ☐ Final Plat - Case Number(s):
☐ Conditional Use - Case Number(s):
☐ Zoning variance - Case Number(s):
☐ Land Use Enforcement Action for
☐ Building or Land Use Permit for
☐ Wetland permit: ☐ Army Corp of Engineers ☐ Municipality of Anchorage

DOCUMENTATION

Required:
☐ Original application with signature(s), 35 copies of application, plus 35 sets of:
☐ Site plan to scale depicting: building footprints; parking areas; vehicle and pedestrian circulation; lighting; grading; landscaping; signage; drainage and project location.
☐ Building plans to scale depicting: floor plans; building elevations; exterior colors and textures.
☐ Application and narrative: explaining the project; planning objectives; construction and operation schedule; final ownership
☐ Assembly Ordinance enacting zoning special limitations, if applicable.
☐ Watershed sign off form, completed

Required if indicated:
☐ Air quality impact ☐ Traffic impact analysis ☐ Economic impact analysis
☐ Soils analysis ☐ Noise impact analysis ☐ Holding capacity of the land analysis
☐ Shadow impact analysis

PUBLIC FACILITY PROJECT LANDSCAPING REVIEW STANDARDS (if applicable)
The Urban Design Commission shall consider the following criteria in reviewing public facility project landscaping under this section. Each standard must have a response in as much detail as it takes to explain how your project satisfies the standard. The burden of proof rests with you. Use additional paper if needed.:
Cost. N/A
Feasibility.

Explain how planning and design criteria are met by the proposed landscape plan:
The external impacts generated by the public facility project on adjacent areas. The landscape elements of the public facility project should complement, maintain or improve the landscape quality of adjacent neighborhoods and areas. N/A
The degree to which the landscape elements contribute to on-site use of the public facility project. The landscape elements of the public facility project should enhance safe, efficient and comfortable public use. N/A
The visual attractiveness of the landscaping and its enhancement of the architecture of the public facility project, including the integration of internal and exterior architectural themes. N/A

Application for site plan review continued

PUBLIC FACILITY STANDARDS (if applicable)

The Planning and Zoning Commission shall review a proposed site plan for consistency with the goals, policies and land use designations of the comprehensive development plan and other municipal plans adopted by the assembly, conformity to the requirements of this title, and the effects of the proposal on the area surrounding the site.

N/A

SPECIAL LIMITATION STANDARDS (if applicable)

The Planning and Zoning Commission shall review the proposed site plan governed by special limitation for consistency with the special limitations, goals, policies and land use designations of the comprehensive development plan and other municipal plans adopted by the assembly, conformity to the requirements of this title, and the effects of the proposal on the area surrounding the site. Each special limitation standard must have a response in as much detail as it takes to explain how your project satisfies the standard. The burden of proof rests with you. Use additional paper if needed.:

N/A

GENERAL SITE PLAN REVIEW STANDARDS (AMC 21.50.200) (if applicable)
The Planning and Zoning Commission shall review the proposed site plan governed by the general site plan review standards for consistency with conformity to the requirements of this title, and the effects of the proposal on the area surrounding the site. Each standard must have a response in as much detail as it takes to explain how your project satisfies the standard. The burden of proof rests with you. Use additional paper if needed.:
Explain how the proposed site plan meets the criteria for its approval established under this title.
See attached narrative, section titled Conformance with Assembly Zoning Ordinance AO 2006-046.

Explain how the proposed conditional use will not have a permanent negative impact on the items listed below substantially greater than that anticipated from permitted development:
1. Pedestrian and vehicular traffic circulation and safety. N/A
2. The demand for and availability of public services and facilities. N/A
3. Noise, air, water or other forms of environmental pollution. N/A
4. The maintenance of compatible and efficient development patterns and land use intensities. N/A

Downtown Edge at The Rail

A Condominium Development

Site Plan Application-Narrative

Table of Contents

Downtown Edge at The Rail
Condominium Development Narrative

Introduction:

The proposed Downtown Edge at The Rail project is a condominium project, located at the northwest corner of Second Avenue and Christensen Drive. The property is located on Alaska Railroad Terminal Property, and is owned and managed by the Alaska Railroad. The property is governed by Anchorage Assembly Zoning Ordinance AO 2006-46, which provides for property design guidelines and restrictions. Per that zoning ordinance, we are requesting FINAL SITE PLAN review and approval of the attached site plan and other development plans under A.M.C. 21.15.030 and 21.50.200 (Old Code). Also, we are submitting a subdivision plat of the property, which will also proceed under "Old Code", and will be heard by the Planning and Zoning Commission at the same time as this site plan.

The proposed development is for 28 units. The buildings consist of 3 basic floor plans. The site is comprised of (3) four-plex units, (1) tri-plex, (1) 5-unit building and (1) 8-plex building. Floor plans range in size from 1,300 s.f. to 1,900 s.f. of living area, and all are 2 bedroom units. There is a mix of 1 and 2 car garage units.

The property will be leased from the Alaska Railroad on a long-term basis (90 years). The current legal description for the property is a portion of Lots 1 and 2, Block 122, U.S. Survey 408. The property is also referred to as *"Alaska Railroad Reserve, Additional Terminal Reserve"*. The development lease site will be approximately 2.62 acres. Additionally, the site is being re-platted together with this site plan application. The plat will combine the two lots into one lot, and will include an easement vacation / relocation to remove and relocate the storm drain easement on the north side of the property. The individual dwellings will be condominium units, created under A.S. 34.08.170.

The construction / development of the property will be governed by A.M.C.R. 21.90 and the applicable standards of the DCM. The roads will be privately owned and maintained. As this application is governed by the Old Code, those development standards regarding parking and other development features will be applied.

The property developer and applicant is Ship Creek Development LLC. The project architect is Lumen Design. The project engineer is Triad Engineering, and the surveyor/planner is The Boutet Company.

Grading, site utilities and road construction is anticipated to be done in 2017-2018, with building construction starting in 2017.

This 28-unit development is part of an ongoing master plan effort being undertaken by the developers of this phase, the Alaska Railroad and other parties. See attached master plan.

Also, while the Old Code doesn't necessarily require Community Council presentations, the Development Team presented the project to the Downtown Community Council on 1/4/2017. Generally, the feedback was favorable to the development.

Downtown Edge at The Rail
Condominium Development Narrative

Property Overview and Planning Process:

This property is part of the Ship Creek Framework Plan, which was initially implemented in 1991 as the "Ship Creek Waterfront/Land Use Plan" (A.O. 91-88) then it was updated in 2014 as A.O. 2014-79. The actual zoning ordinance 2006-046 actually establishes the zoning in the individual areas. Part of this proposed developments lease area (Lot 2, Block 122 of U.S. Survey 408) was actually excluded from the initial zoning ordinance (that area is also referred to as "GSA Property per PLO 3532"). After the General Services Administration of the Federal Government transferred ownership to the Alaska Railroad, that portion of the land was rezoned to match the surrounding land under A.O. 2004-123. The GSA building on site will be removed as site construction commences. The existing zoning to the east and south is B-2C.

Per the zoning ordinance 2006-046, the project plan must be conceptually approved by the "Ship Creek District Review Board" before it can be presented to the Planning and Zoning Commission. That Review Board is comprised of personnel from the Alaska Railroad Corporation, the MOA Planning Staff and a representative of the P&Z Commission. That review was held December 22, 2016.

This narrative, site plans and other drawings are meant to supplement the initial Planning and Zoning application.

Utility, Road Improvements and Drainage:

The site abuts the North Right of Way of West 2nd Avenue, and the west Right of Way of Christensen Drive. Both roads are improved to full Municipal standards (30 feet wide, with curb and gutter and concrete sidewalks). To the north are the railroad tracks, and to the west is an ML&P substation. The north side of the site is steeply sloped, and the 11 units situated on the north side will be terraced into the slope.

The project drainage will be collected onsite and directed to existing storm drain lines to the north. There's an existing sewer main line available approximately 150 feet north of the property. Water main is available immediately adjacent to the property on the west side.

Because Christensen Drive was recently improved in 2000, it is desirable to minimize construction in the ROW. The utility configurations adjacent to the north and west of the property make that possible.

There will be 2 access points into the project, one from Christensen, the other from West 2nd Avenue. The road/driveway construction will be an "inverted" section, 24 and 26 feet wide (refer to grading and utility plan). This type of road construction allows for efficient drainage of the roads and driveways.

All shallow utilities (gas, electric and communications) are available immediately adjacent to the property.

Downtown Edge at The Rail
Condominium Development Narrative

Traffic, Parking and Pedestrian Circulation Considerations:

Traffic: While the new development will add additional traffic to the intersection, the traffic study performed for the project indicates that the development of 28 units will generate relatively low levels. Additionally, the proposed locations of the driveway access points exceed the minimum driveway clearances required by MOA design standards and have adequate site distance along both 2nd Avenue and Christensen Drive (refer to TIA Report by Kinney Engineering dated November 14, 2016).

Pedestrian circulation for the site is provided by a 5-foot sidewalk off both West 2nd Avenue and Christensen.

The ***parking standards*** for "old code" (21.45.080.2.d) are met, per the following calculation:
28 two bedroom units: 1.75 X 28 =49 +15% overflow (21.90 F.4) = **55 Total Required.**
79 parking provided (48 garage, 28 driveway and 3 outside stalls).

Because of the terrain drop on the north tier of the property, the units have been stepped into the slope. To keep the buildings from pushing too far over this slope, the driveways into the buildings have properties that may require formal design variances. These will be addressed when the construction plans are brought in for review. This issue has been discussed with and acknowledged by the Municipal Traffic Engineer. Refer to traffic memo dated 6/21/2016.

Geotechnical Considerations:

As required by the zoning ordinance, an evaluation of the project by the Geotechnical Advisory Commission (GAC) is required. Because of the project location within the MOA Seismic Zones 4/5 as well as the zoning ordinance requirement, an in depth Geotechnical Assessment was performed by Northern Geotechnical Engineering (NGE) this past summer, and it includes engineering recommendations for building foundation design. The project report was presented for review at the GAC's December 27th meeting. Based on feedback from the Committee at that meeting, the NGE report is in the process of being revised and will be presented to the GAC at their 1/24/17 meeting.

Landscaping:

Currently, the property is heavily wooded on the north and west sides by alders and cottonwood. There are existing large spruce trees along the north side of West 2nd Avenue on the south side of the property, some of which may be retained.

Refer to the development site plan-landscape plan for proposed landscaping.

Building Construction:

The building construction will be conventional wood style condominiums. As previously mentioned, the buildings in the north tier will be terraced down the slope, and will have engineered foundations specific to that area. See building plans for more details.

Downtown Edge at The Rail
Condominium Development Narrative

Submitted Project Plans and Documents:

- Site Plan / Landscape Plan, dated 12/14/2016 (full size and 8 ½" X 11")
- Existing site conditions map (with and without imagery) dated 11/21/2016
- Utility and grading plan, dated Nov., 2016 by Triad Engineering
- Building plans and elevations from Lumen Design, dated 12/15/2016
- Kinney Engineering Traffic Study dated 11/14/2016
- Street section acknowledgement memo from Kent Kohlhase and Stephanie Mormilo dated 12/2/2016
- Geotechnical report "Geotechnical Assessment of the site of the proposed Ship Creek Development, Anchorage, AK. by NGE/TFT dated 8/9/16
- Traffic memo regarding driveway configuration, prepared by Triad Engineering and acknowledged by Stephanie Mormilo, dated 6/7/2016.
- "The Rail" Concept Master Plan Map by KPB.

Conformance with Assembly Zoning Ordinance AO 2006-046:

Land Use (Section D.5.a): Multi Family use is an allowed PERMITTED use.
Lot and yard requirements (Sections I and J): There are no minimum lot or yard requirements.
Building height (Section K.2.): Building heights from mean sea level (m.s.l.) shall not exceed 120 feet. Conservatively, the maximum building height of these buildings will not exceed 90 feet m.s.l.
Total dwelling units (Section J): The maximum number of housing units is 400 units. As this development is the first housing project under this ordinance, the 28 units are far beneath this standard.
Site plan review (Section P): This project is subject to a Level 2 Development Review process, which triggers a review by the Ship Creek District Review Board. That review has been accomplished. This application is the final required site plan review under the ordinance.

Compliance with Ship Creek Framework Plan, Assembly Ordinance 2014-79:

This development conforms to the land use laid out in the Framework plan, in the area identified as Phase II-B. More specifically, the unit design in the plan is characterized as "residences which fit into the steep topography of the bluff, and feature attractive stepping masses and great views". The design and building orientation depicted on the Framework plan (page 67-68) are very similar in character to the proposed development.

Compliance with A.M.C. 21.15.030, Approval of site plans and conditional uses:

B. Pre-application conference was held with Municipal staff on 4/12/2016
C. Application.

1. Written documents
    a. Legal Description: *(see narrative)*
    b. Statement of planning objectives: *(see narrative)*
    c. Projected dates: *(see narrative)*
    d. Traffic and pedestrian circulation: *(see narrative)*
    e. Landscaping: *(see narrative)*
    f. Zoning map amendment: *N/A*
2. Application maps:
    2. Site conditions map: *(attached)*
    3. Plot Plan / Landscape plan: *(attached)*

D-E. N/A

F. Final Approval
1. This site plan is submitted and subject to approval as a FINAL SITE PLAN.
2. Required Information:
    a. Landscape plan: *(attached)*
    b. Preliminary grading and drainage plan: *(attached)*

Compliance with A.M.C. 21.50.200, General Standards for Site Plan:

A. Meet the criteria for its' approval: *The development of this property is governed by A.O. 2006-46, which specifies the approval criteria is approval and compliance with 21.15.130. Compliance to 21.15.130 is demonstrated above.*

B. Will not have a permanent negative impact on those items listed in this subsection substantially greater than that anticipated from permitted development: *It should be noted that this is a permitted use as outlined in the A.O. 2006-46, and that the proposed development follows the Ordinance and the Ship Creek Master Plan. However, individual responses have been prepared for each of the items.*
1. Pedestrian and vehicular traffic circulation and safety: *The development of 28 housing units will not have a negative impact on the on the traffic circulation, and the increase in traffic is minimal. The impact and design of the proposed 2 driveways is discussed in the Kinney Engineering Traffic Study.*
2. The demand and availability of public services and facilities: *Connections to all the deep underground utilities (water, sewer and storm sewer) are available adjacent to the development. There is plenty of capacity in all utilities. Shallow utilities (electric, communications and gas) are all available adjacent to the site. Refer to the utility and grading plan prepared by Triad Engineering.*
3. Noise, air, water or other forms of environmental problems: *The development of 6 buildings will create minimal environmental impacts.*
4. The maintenance of compatible and efficient development patterns and land use densities: *Again, this proposed development follows the development scenario as outlined in the adopted Ship Creek Master Plan. As such, it is maintaining the development patterns envisioned by the Municipality when it approved the plan.*

Notes:

1. BASIS OF BEARINGS ARE THE FOUND MONUMENTS, PER PLAT 78-170.
2. BASIS OF ELEVATION IS GAAB DATUM, 1972 NGS ADJUSTMENT.
3. THIS SURVEY WAS PERFORMED MAY-AUGUST, 2016.

STATE OF ALASKA 49TH Anthony P. Hoffman LS-9020 Registered Professional Land Surveyor

0 30 60 Feet

N

LANDSCAPE SCHEDULE

QTY	SYMBOL	LABEL	LATIN NAME	COMMON NAME	SIZE	FURNISHED	NOTES
EVERGREEN TREES							
16		PP	PICEA PUNGENS	COLORADO GREEN SRUCE	6' HT.	B&B	5:3 RATIO
DECIDUOUS TREES							
34		BPG	BETULA PENDULA 'GRACILIS'	CUTLEAF WEEPING BIRCH	6' HT.	B&B	5:3 RATIO
SHRUBS							
160		CL	COTONEASTER LUCIDUS	HEDGE COTONEASTER	24" HT.	POTTED	
MISCELLANEOUS							
			SCHEDULE-A SEED MIX (LAWN MIX)		EXISTING VEGETATION TO REMAIN		

LANDSCAPE NOTES:
1. ALL PLANTS ARE NURSERY GROWN UNLESS SPECIFIED OTHERWISE.
2. ALL PLANTING BEDS SHALL RECEIVE 18" TOPSOIL AND 3" DEPTH SHREDDED BARK MULCH.
3. PROVIDE 4" TOPSOIL AND SEED ALL DISTURBED AREAS WITH SCHEDULE NOTED ON PLANS.

Legend

- Aluminum Cap Monument
- Brass Cap Monument per USS 408
- 5/8" Rebar
- Underground Electric Line per Markings
- Waterline per AWWU As-builts
- Storm Drain per As-builts
- Sewer per AWWU As-builts
- Underground Communication Line per Markings
- 4' Blue Rail Fence
- Chain Link Fence
- Storm Drain Inlet
- Water Valve
- Pedestrian Trail Light
- Luminaire Street Light
- Fire Hydrant
- Sewer Manhole
- Storm Manhole
- Telecom Manhole
- Electric Manhole
- Existing Pavement
- Existing Curb & Gutter
- Proposed Storm Drain Manhole
- SD — Proposed Storm Drain
- Proposed Sewer Manhole
- SS — Proposed Sewer Line
- Proposed Fire Hydrant
- WL — Proposed Water Line
- Proposed Pavement

Vicinity Map Scale: 1" = 1 Mile

Project Location

SITE PLAN / LANDSCAPE PLAN

Located on Portions of:

The Alaska Railroad Additional Reserve, Situated in Lot 1, and Lot 2, Block 122, USS 408, and The Alaska Railroad Reserve, Located in Lot 2, USS 1170

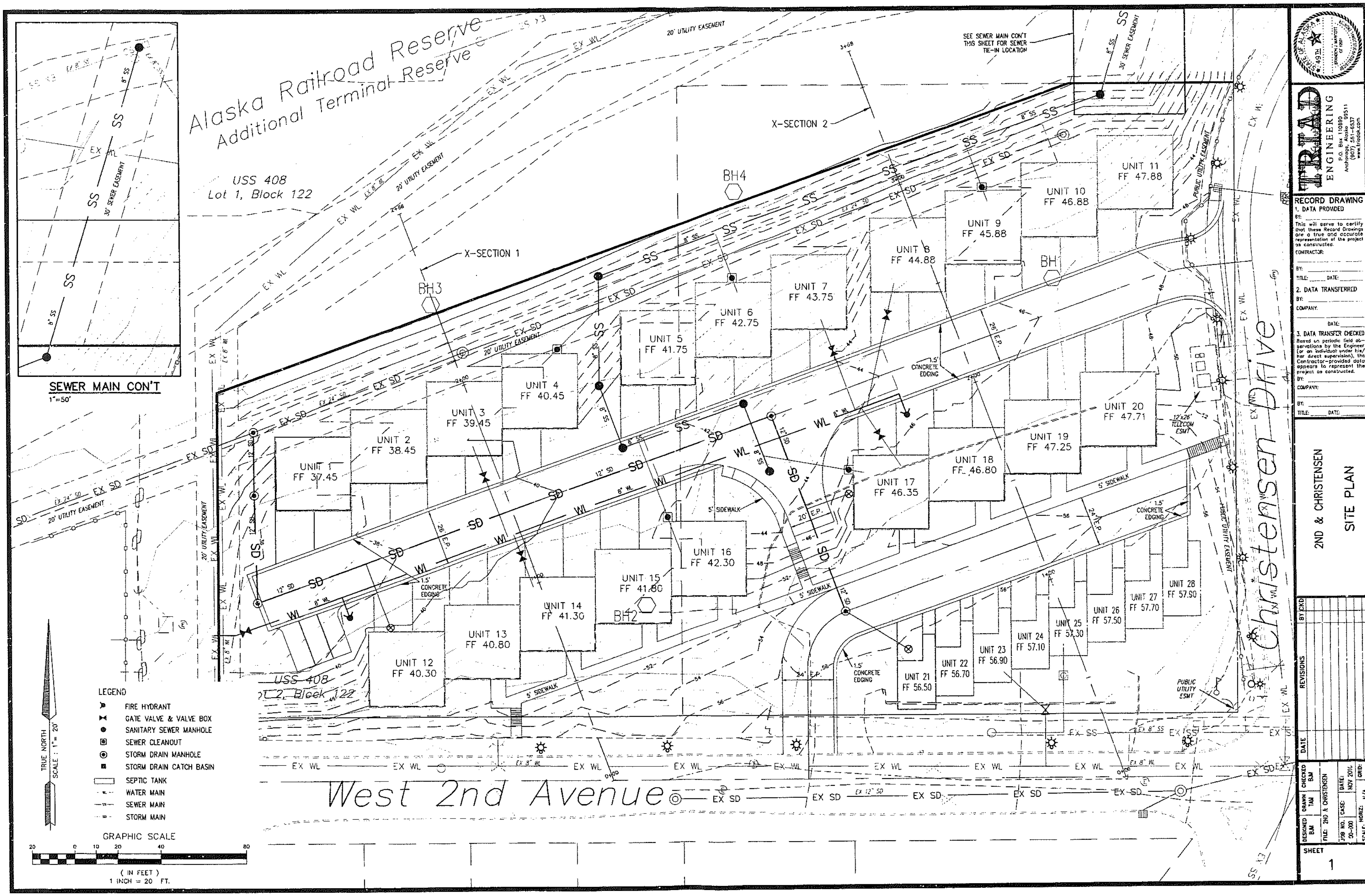
According to the official BLM Plats plat thereof, and per Plat Number 78-170, records of the Anchorage Recording District, Third Judicial District, State of Alaska.

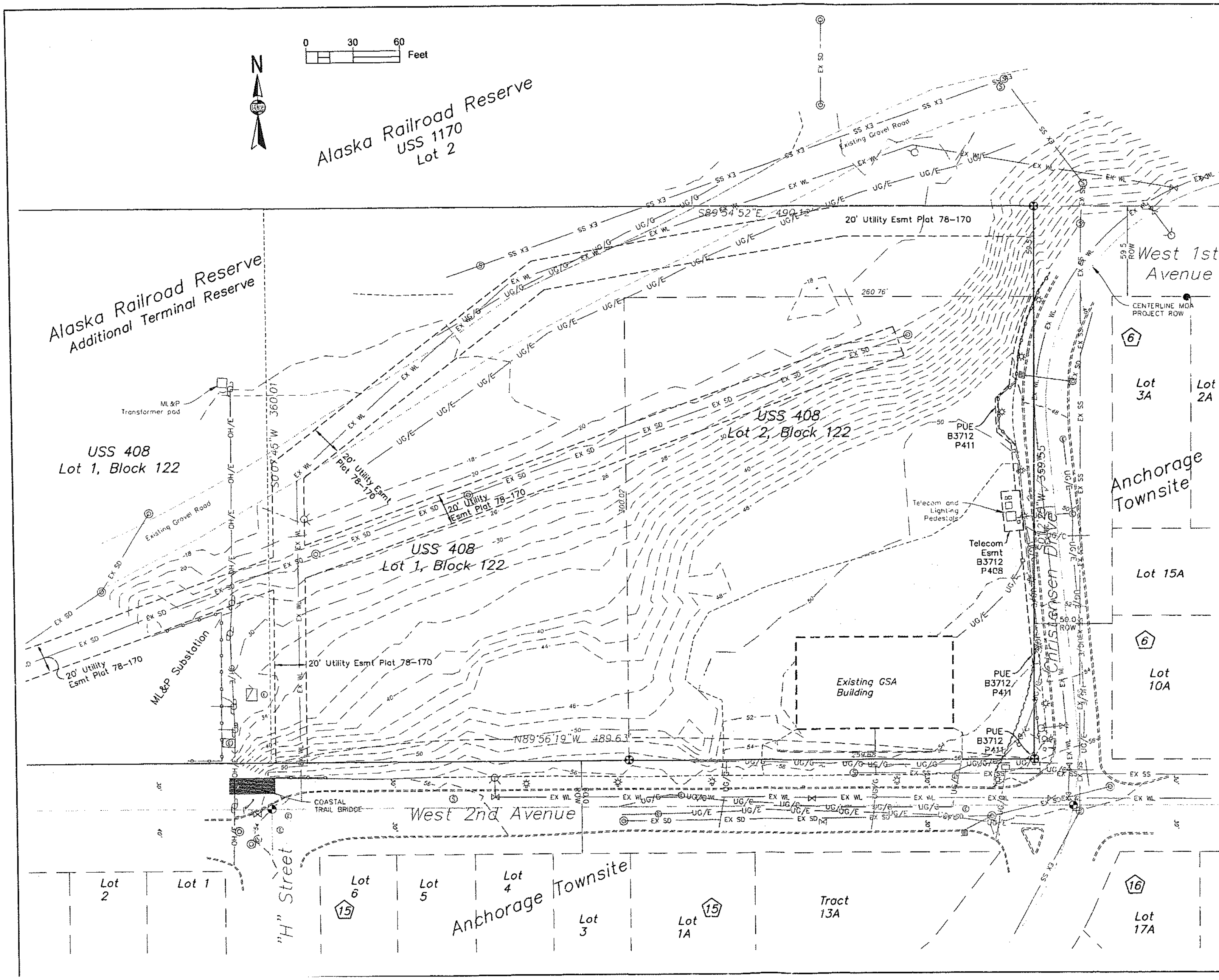
LOCATED WITHIN:

Section 11, Township 13 North, Range 3 West, S.M.

MOA Grid Map: SW1230, SW1229	
Scale: 1"=30'	
Drawn By: TH	Checked:
Job No.	
Date: 12/14/2016	
Plat No.:	

tbc
The Boutet Company, Inc.
601 E. 57th Place #102
Anchorage, AK. 99518
Ph. 907-522-6776


Alaska Railroad Reserve
Additional Terminal Reserve
USS 408
Lot 1, Block 122
USS 408
Lot 2, Block 122
West 2nd Avenue
Christensen Drive
SEWER MAIN CON'T
1"=50'
SEE SEWER MAIN CON'T THIS SHEET FOR SEWER TIE-IN LOCATION
X-SECTION 1
X-SECTION 2
BH2
BH3
BH4
BH
UNIT 1 FF 37.45
UNIT 2 FF 38.45
UNIT 3 FF 39.45
UNIT 4 FF 40.45
UNIT 5 FF 41.75
UNIT 6 FF 42.75
UNIT 7 FF 43.75
UNIT 8 FF 44.88
UNIT 9 FF 45.88
UNIT 10 FF 46.88
UNIT 11 FF 47.88
UNIT 12 FF 40.30
UNIT 13 FF 40.80
UNIT 14 FF 41.30
UNIT 15 FF 41.80
UNIT 16 FF 42.30
UNIT 17 FF 46.35
UNIT 18 FF 46.80
UNIT 19 FF 47.25
UNIT 20 FF 47.71
UNIT 21 FF 56.50
UNIT 22 FF 56.70
UNIT 23 FF 56.90
UNIT 24 FF 57.10
UNIT 25 FF 57.30
UNIT 26 FF 57.50
UNIT 27 FF 57.70
UNIT 28 FF 57.90
20' UTILITY EASEMENT
30' SEWER EASEMENT
PUBLIC UTILITY EASEMENT
PUBLIC UTILITY ESMT
12'x26' TELECOM ESMT
5' SIDEWALK
1.5' CONCRETE EDGING
26' E.P.
24' E.P.
20' E.P.
LEGEND
FIRE HYDRANT
GATE VALVE & VALVE BOX
SANITARY SEWER MANHOLE
SEWER CLEANOUT
STORM DRAIN MANHOLE
STORM DRAIN CATCH BASIN
SEPTIC TANK
WATER MAIN
SEWER MAIN
STORM MAIN
TRUE NORTH
SCALE : 1" = 20'
GRAPHIC SCALE
(IN FEET)
1 INCH = 20 FT.
TRIAD ENGINEERING
P.O. Box 110890
Anchorage, Alaska 99511
(907) 561-6537
www.triadak.com
RECORD DRAWING
1. DATA PROVIDED
This will serve to certify that these Record Drawings are a true and accurate representation of the project as constructed.
CONTRACTOR:
BY:
TITLE: DATE:
2. DATA TRANSFERRED
BY:
COMPANY:
DATE:
3. DATA TRANSFER CHECKED
Based on periodic field observations by the Engineer (or an individual under his/her direct supervision), the Contractor-provided data appears to represent the project as constructed.
BY:
COMPANY:
BY:
TITLE: DATE:
2ND & CHRISTENSEN
SITE PLAN
DATE
REVISIONS
BY CKD
DESIGNED BJM
DRAWN TAM
CHECKED BJM
FILE: 2ND & CHRISTENSEN
JOB NO. 00-000
CASE:
DATE: NOV 2016
HORIZ: N/A
VERT:
SCALE:
GRID:
SHEET
1



Legend

- Aluminum Cap Monument
- Brass Cap Monument per USS 408
- 5/8" Rebar
- UG/E — Underground Electric Line per Markings
- EX WL — Waterline per AWWU As-builts
- UG/G — Gasline per Markings
- EX SD — Storm Drain per As-builts
- EX SS — Sewer per As-builts
- UG/C — Underground Communication Line per Markings
- 4' Blue Rail Fence
- Storm Drain Inlet
- Water Valve
- Pedestrian Trail Light
- Luminaire Street Light
- Fire Hydrant
- Sewer Manhole
- Storm Manhole
- Telecom Manhole
- Electric Manhole
- Pavement
- —46— Existing Contour (2' interval)

Notes:

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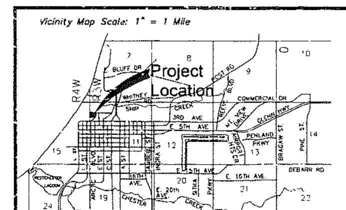
2. BASIS OF ELEVATION IS GAAB DATUM, 1972 NGS ADJUSTMENT.
3. THIS SURVEY WAS PERFORMED MAY-AUGUST, 2016.

STATE OF ALASKA
49TH
Anthony P. Hoffman
LS-9020
Registered Professional Land Surveyor



Existing Conditions Map

Located on Portions of:

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According to the official BLM Plats plat thereof, and per Plat Number 78-170, records of the Anchorage Recording District, Third Judicial District, State of Alaska.

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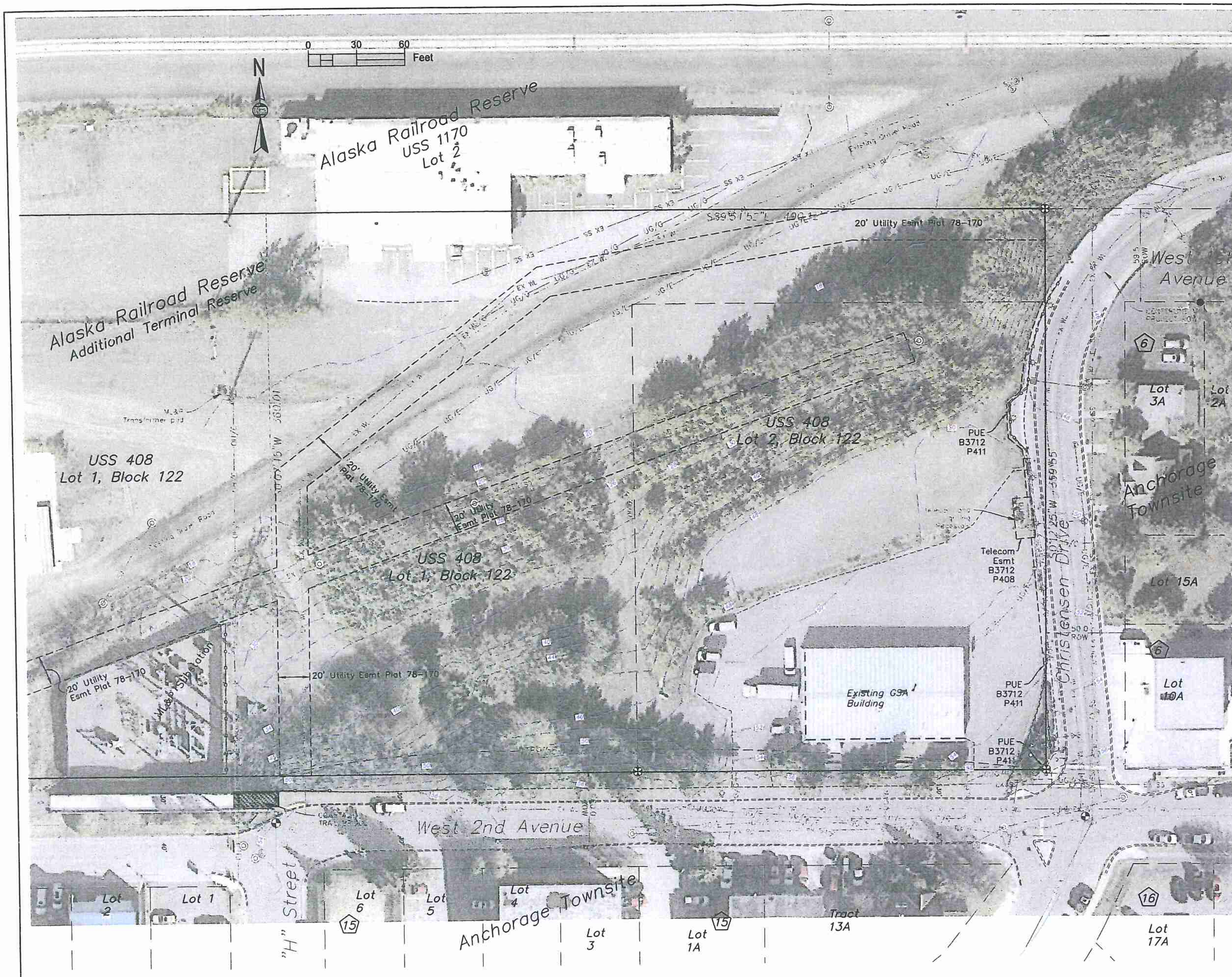
Scale: 1"=30'	
Drawn By: TH	Checked:
Job No.	
Date: 11/21/2016	
Plat No.:	
SHEET: 1 of 1	

The Boutet Company, Inc.
601 E. 57th Place #102
Anchorage, AK 99518
Ph. 907-522-6776
Fx. 907-522-6779
License No. AECC957



Legend

	Aluminum Cap Monument
	Brass Cap Monument per USS 408
	5/8" Rebar
UG/E	Underground Electric Line per Markings
EX WL	Waterline per AWWU As-builts
UG/G	Gasline per Markings
EX SD	Storm Drain per As-builts
EX SS	Sewer per As-builts
UG/C	Underground Communication Line per Markings
	4' Blue Rail Fence
	Storm Drain Inlet
	Water Valve
	Pedestrian Trail Light
	Luminaire Street Light
	Fire Hydrant
	Sewer Manhole
	Storm Manhole
	Telecom Manhole
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	Pavement
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Registered Professional Land Surveyor

Vicinity Map Scale: 1" = 1 Mile

Existing Conditions Map

Located on Portions of:

The Alaska Railroad Additional Reserve, Situated in Lot 1, and Lot 2, Block 122, USS 408, and The Alaska Railroad Reserve, Located in Lot 2, USS 1170

According to the official BLM Plats plat thereof, and per Plat Number 78-170, records of the Anchorage Recording District, Third Judicial District, State of Alaska.

LOCATED WITHIN:
Section 11, Township 13 North, Range 3 West, S.M.

MOA Grid Map: SW1230, SW1229	
Scale: 1"=30'	
Drawn By: TH	Checked:
Job No.	
Date: 11/21/2016	
Plat No.:	
SHEET: 1 of 1	

The Boutet Company, Inc.
601 E. 57th Place #102
Anchorage, AK 99518
Ph. 907-522-6776
Fx. 907-522-6779
License No. AECC957

Authorization Certificate

Date: December 14, 2016

Current Project Legal: The Alaska Railroad Additional Reserve, Situated in Lot 1, and Lot 2, Block 122, USS 408, and The Alaska Railroad Reserve, Located in Lot 2, USS 1170

Proposed Legal: Same

Type of Authorization: Preliminary Plat and Site Plan Applications

Statement:

I hereby authorize Tony Hoffman of The Boutet Company Inc. to represent me in the Municipality of Anchorage Platting and Site Plan Applications of the above described property.

Thank you,

Ship Creek Development:

The Rail
at ship creek

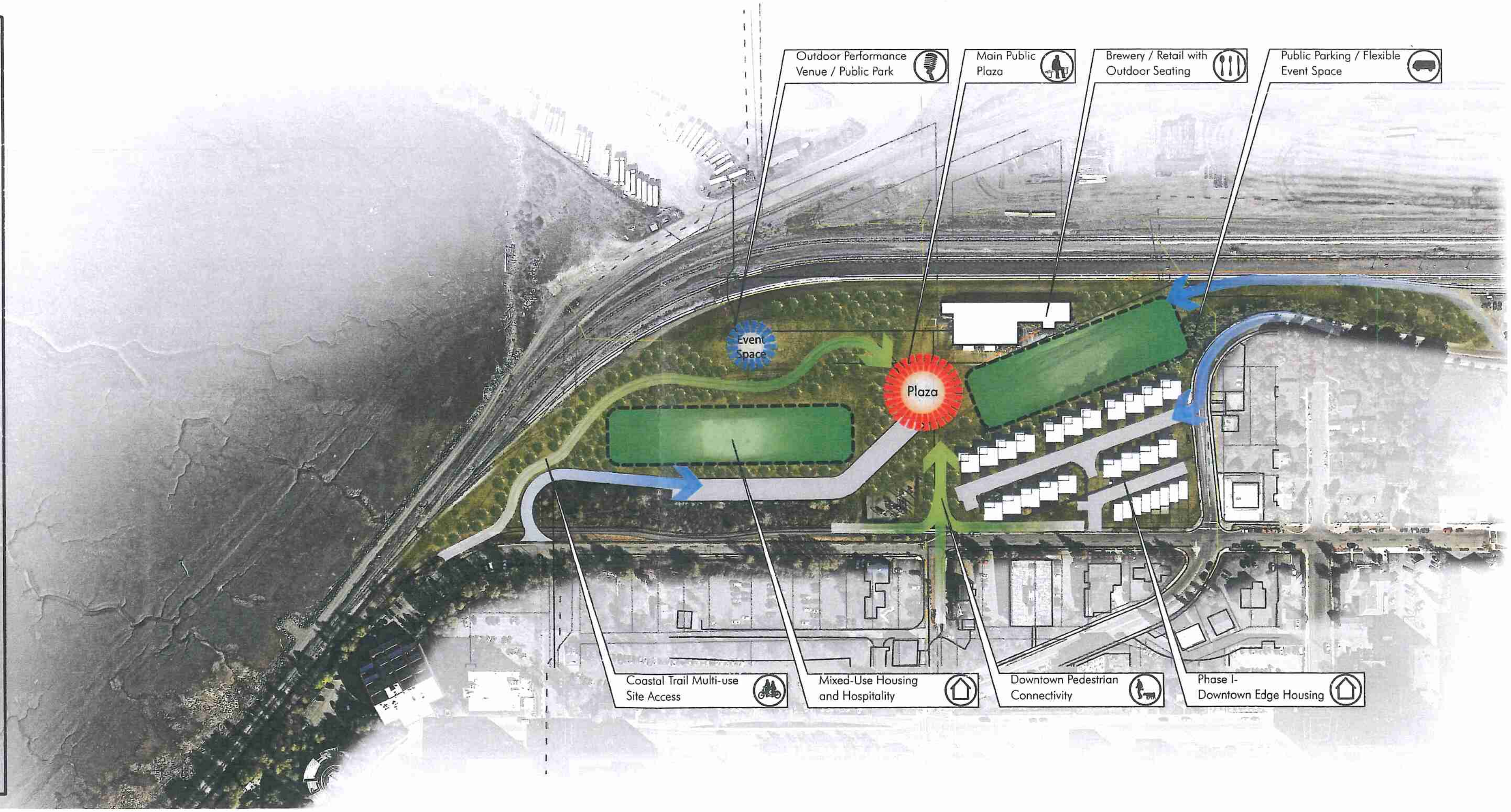
Concept Master Plan

HBC

Swell, LLC

the petersen group BUILDING & DEVELOPMENT

KPB ARCHITECTS



NORTHERN GEOTECHNICAL ENGINEERING, INC. / TERRA FIRMA TESTING

Laboratory Testing   Geotechnical Engineering   Instrumentation   Construction Monitoring Services   Thermal Analysis

August 9, 2016   NGE-TFT Project # 4385-16(G)

John McGrew
9831 Main Tree Drive
Anchorage, Alaska

RE: GEOTECHNICAL ASSESMENT OF THE SITE OF THE PROPOSED SHIP CREEK DEVELOPMENT, ANCHORAGE, AK.

John,

We (Northern Geotechnical Engineering, Inc. *d.b.a.* Terra Firma Testing) have completed a geotechnical engineering assessment of the aforementioned project. We have provided details of our findings along with our conclusions and engineering recommendations in the following report.

The project site has the potential for both slope failure and large lateral movements (cyclical movements of the order of two feet) under strong seismic motions. The purpose of this report is to help design building foundations that will remain intact enough to minimize the risk to human life of the occupants during a strong seismic event. It is not economically feasible at this site to design a foundation that will not move during a large seismic event. Therefore, our approach is to design foundations that will move as a whole during a large seismic event, and as such, we have recommended a thickened edge concrete structural slab foundation. *Again, it is important to note that this foundation is designed to help reduce the potential for catastrophic collapse of the building and loss of life during a strong seismic event, it is not designed to prevent movement or damage to the building. After such an event, the building may be displaced from its original location and/or be un-inhabitable.*

In addition, the project site has a significant amount of fill material which has been placed over the past several decades. We cannot be certain of the vertical and horizontal extents of this fill across the project site, nor can we be certain of the level of compaction effort (if any) that was applied to the fill during placement. These factors have led us to be more conservative with our design recommendations.

We greatly appreciate the opportunity to provide you with our professional service. Please contact us directly with any questions or comments you may have regarding the information that we present in this report, or if you have any other questions, comments, and/or requests.

Sincerely,
Northern Geotechnical Engineering, Inc. *d.b.a.* Terra Firma Testing,

STATE OF ALASKA
49TH
9 Aug 2016
Keith F. Mobley
CE - 5066
REGISTERED PROFESSIONAL ENGINEER

Andrew L. Fortt
Project Engineer

Keith F. Mobley
President

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1.0 INTRODUCTION

In this report, we (Northern Geotechnical Engineering, Inc. *d.b.a.* Terra Firma Testing) present the findings of our geotechnical assessment that we conducted at the proposed Ship Creek Development; hereafter referred to as "the project site". We provided our professional service in accordance with our service fee proposal (#16-078G) which we submitted to you on April 7, 2016. You authorized our proposed scope by signed fee proposal on April 20, 2016.

We were contracted to characterize the subsurface conditions across the project site in an effort to provide geotechnical design criteria and engineering recommendations for the proposed improvements.

In this report, we provide a summary of our subsurface exploration and laboratory testing programs, as well as provide our conclusions regarding the suitability of the project site for the proposed improvements. We also provide design and construction recommendations for the proposed improvements.

2.0 SITE AND PROJECT DESCRIPTIONS

As we detail in Figure 1 of this report, the project site is located in the ARRC Additional Terminal Reserve area in Anchorage, Alaska. The project site consists of two adjacent parcels, which are:

1. 701 West 2nd Ave; and
2. the southeast portion of 801 West 2nd Avenue.

The proposed improvements to the project site include the demolition of the existing structures and the construction of a series of residential, multi-unit dwellings, and associated utilities, parking areas, and driveways. A conceptual site plan was not available at the time we issued this report.

We believe (based upon our field exploration, previous geotechnical reports and a study of the available aerial photography) that there has been a significant amount of fill placed at the project site at various intervals since at least 1960. Based upon the information available to us at the time of this report, we cannot be certain of the horizontal and vertical extents of this fill. Appendix A of this report contains aerial photographs of the project site taken over a timespan of several decades. As can be seen from the photograph in Appendix A-1, fill placement was occurring during May of 1960. By July of 1970 (Appendix A-2) the original slope had been re-graded and a pad and building (General Services Administration (GSA) building) constructed in the southeast corner of the lot. Sometime between 2000 (a report entitled "*GSA Parking, Slope Stability Study, Anchorage, AK* by Shannon & Wilson was produced) and 2002 (see image in Appendix A-3 and A-4) the parking lot for the GSA building was increased in area and the slope

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to the north was re-graded. The project site appears to be relatively unchanged since 2002 (see image taken in May 2015 in Appendix A-5).

Currently, from the GSA building parking area, the project site slopes downwards to the north and west (approximately 4H:1V), with a vertical relief of approximately 30 feet. The slope is currently vegetated with small trees and brush.

At the toe of the slope is the former tidal flats area of Ship Creek that flows east to west, north of the project site. Following the 1964 Good Friday Earthquake, the toe of this slope was reinforced with a compacted sand and gravel buttress, with the intent to improve stability of the slope. Currently, this area is devoid of vegetation and is being used as a vehicle parking lot.

As we discuss in detail later in this report, the project site has a high to very high ground failure susceptibility (MOA Seismic Hazard Zones 4 to 5). While the project site did not slide during the 1964 earthquake, the areas surrounding the project site did slide (L Street Slide and 4th Avenue Slide). Therefore we have customized our geotechnical recommendations to help protect the proposed improvements from catastrophic failure during a large seismic event.

3.0 PREVIOUS EFFORTS

Shannon & Wilson, Inc. (S&W) prepared a geotechnical report entitled "*Preliminary Geotechnical Study, Ship Creek Development Plan, Additional Terminal Reserve, Anchorage, Alaska*" in September of 2015. The S&W report discusses and summarizes the different subsurface exploration activities that they (and others) have conducted in the vicinity of the project site at various dates in the past. Two S&W borings (drilled in 2000 for their geotechnical report entitled "*GSA Parking, Slope Stability Study, Anchorage, AK*") are located on the project site. S&W's report provides some useful information, however their report was intended to be used in the conceptual-level design stages and does not provide the complete information necessary for a geotechnical engineering design for the project site.

4.0 CURRENT SUBSURFACE EXPLORATION

We coordinated and directed subsurface explorations at the project site on May 5, 2016. We contracted Discovery Drilling (DD), who in turn mobilized a truck-mounted CME-75 drill rig and two-man drill crew to the project site to perform the necessary drilling, sampling, and borehole backfill activities. DD advanced a total of five boreholes at the project site, designated B1 through B5, using 3.25-inch I.D. (6.625-inch O.D.) hollow-stem augers with center drill rods. We have plotted the approximate location of the five boreholes on Figure 2 of this report. The boreholes were advanced to depths of between 32 feet to 52 feet below the existing ground surface (bgs). A representative from our firm was present on-site during the entire drilling program to direct the drilling activities, which included: determining the final location of each borehole; logging the geology of each borehole; collecting and preparing representative soil samples; and directing borehole backfill activities.

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Under our direction, DD performed a Modified Penetration Test (MPT) at regular intervals during the drilling of each borehole. A MPT can be used to assess the consistency of a soil interval and to collect representative soil samples. A MPT is performed by driving a 3.0-inch O.D. (2.4-inch I.D.) split-spoon sampler at least 18 inches past the bottom of the advancing augers with blows from a 340-lb drop-hammer, free-falling 30 inches onto an anvil attached to the top of the drill rod stem. Our field representative recorded the hammer blows required to drive the modified split-spoon sampler the entire length of each sample interval, or until sampler refusal was encountered. We have provided the field blow count data for each sample interval (in six-inch increments) on the graphical borehole logs in Appendix B of this report.

We corrected the field blow count data for all five boreholes for standard confining pressure, drill rod length, and drop-hammer operation procedure to estimate a standard $(N_1)_{60}$ value for each sample interval. $(N_1)_{60}$ values are a measure of the relative density (compactness) and consistency (stiffness) of cohesionless or cohesive soils, respectively. Our estimate of the $(N_1)_{60}$ values is based on the drop-hammer blows required to drive the spilt-spoon sampler the final 12-inches of an 18-inch MPT. We have provided our estimated $(N_1)_{60}$ values for each sample interval on the graphical borehole logs in Appendix B of this report. The automatic drop-hammer that DD used for this project is not standard, so we applied a correction factor of 1.1 to the $(N_1)_{60}$ values to account for the efficiency of the automatic drop-hammer that DD used for the project. We have provided a graphical plot of the field blow count corrections for confining pressure and drill rod length in Figure 3 of this report.

Our field representative photographed each split-spoon sample that they collected during the exploration program. A photograph of each split-spoon sample that we collected during our subsurface exploration program is provided in Appendix B of this report. We sealed each split-spoon sample that was collected during our subsurface exploration program inside of air-tight bags, to help preserve the moisture content of each sample, and then submitted each sample to our laboratory for further identification and analysis.

Under our direction, DD performed thin-walled Shelby tube sampling at specified intervals during the drilling of each borehole. Shelby tube sampling methods are used to collect undisturbed samples of soft, fine-grained (cohesive) soils in an effort to recover intact samples which are representative of the in-situ soil density and water content; two factors which are essential for evaluating engineering properties such as the strength, compressibility, permeability, and density of fine-grained soils. DD collected each Shelby tube sample by advancing a 3.0-inch O.D. seamless steel tube (constructed from either 16-gauge or 18-gauge steel) past the bottom of the advancing augers by applying constant downward pressure directly to the drill rod stem using the vertical hydraulic feed system of the drill rig. We recorded the average hydraulic feed pressure (in psi) required to advance each Shelby tube sampler 24 inches (in six-inch increments).

DD allowed each Shelby tube sample to rest (in-place) for at least five minutes prior to sampler retrieval, which increases the potential for complete sample recovery by allowing the soil sample to adhere to the inside of the Shelby tube sampler. Following the rest period, DD manually

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rotated the drill rod stem (using a large pipe wrench) approximately 180 to 360 degrees in an attempt to shear the end of the soil sample from the in-situ soils and relieve any suction pressures, thus reducing the potential for the soil sample to be pulled from the Shelby tube sampler upon sampler retrieval. Our field representative sealed, labeled, stored, and transported each Shelby tube sample from the project site to our laboratory in a manner consistent with the standard practices outlined in ASTM D1587-08.

Once the exploration activities were complete, we directed DD to backfill the annulus of each exploration with its respective drill cuttings

5.0 LABORATORY TESTING

We collected a total of 56 soil samples from the five boreholes that DD advanced at the project site and submitted all of the soil samples to our laboratory for further identification and geotechnical analysis. We tested select soil samples in general accordance with the respective ASTM standard test methods including:

- moisture content analysis (ASTM D-2216);
- grain size sieve and hydrometer analysis (ASTM D-6913 & D-422);
- organic content (ASTM D2974);
- Atterberg limits (ASTM D-4318);
- Determination of fines content (a.k.a. P200 – ASTM D-1140);
- Consolidated Undrained (CU) triaxial compressive strength (ASTM D4767)

It is important to note that ASTM test method D-6913 requires that any soil sample specimen which is to be submitted for gradational analysis (by ASTM D-422 or other methods) must satisfy a minimum mass requirement based on the maximum particle size of the sample specimen. Split-spoon sampling techniques (standard or modified), as well as other small-diameter soil sampling techniques (e.g., macro-core, etc.), typically recover anywhere from approximately 1 to 10 pounds of sample specimen. The amount of sample specimen recovered can be influenced by (amongst other variables) the soil gradation, soil density, sample interval, sampler tooling, and soil moisture content. As a result, samples of coarse-grained soils (with individual soil particles greater than approximately 0.75 inches in diameter) collected with small-diameter sampling methods (e.g., split-spoons, macro-core, etc.) may not meet the minimum mass requirement specified by Table 2 of ASTM D-6913. This may result in inaccurate gradational and frost classification results. The use of small-diameter sampling devices in coarse-grained soils (e.g., sand and gravel) can result in the collection of unrepresentative samples due to: the exclusion of oversized particles (larger than the opening of the sampler) from the sample; and the mechanical breakdown/degradation of coarse-grained particles by the sampling process (producing an unrepresentative increase in smaller-diameter particles in the sample). Both of these sampling biases can skew laboratory test results towards the fine-grained end of the gradational spectrum.

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The laboratory test results, along with the observations we made during our subsurface exploration efforts, aid in our evaluation of the subsurface conditions at the project site and help us to assess the suitability of the subsurface materials located at the project site to support the proposed improvements. The results of our geotechnical laboratory analyses are provided on the graphical exploration logs contained in Appendix B of this report and on the laboratory data sheets contained in Appendix C of this report.

6.0 DESCRIPTION OF SUBSURFACE CONDITIONS

We compiled our field observations with the results from our laboratory analyses to produce graphical logs of each subsurface exploration (Appendix B). The graphical exploration logs depict the subsurface conditions that we identified at each exploration location and help us to interpret/extrapolate the subsurface conditions for areas adjacent to, and immediately surrounding, each exploration location across the project site.

We positioned the boreholes in an effort to sample subsurface conditions across the entire project site. Borehole B1 is located at the uppermost elevation of approximately +50 feet (reference elevation taken from Figure 2). Borehole B2 is located to the southwest of borehole B1 at an elevation of approximately +45 feet. Boreholes B3, B4 and B5 are all located along the toe of the existing slope, at elevations ranging from +18 feet to +20 feet.

6.1 General Subsurface Profile

In general, our subsurface exploration identified two predominant material types at the project site. The first, a sand/gravel fill material, was encountered directly below the ground surface and was of varying thickness. The second predominant material type was a silt/clay (locally known as the Bootlegger Cove Formation) that extended to depths of at least 52 feet bgs. Previous explorations in the general area (S&W's 2015 report) has demonstrated that the silt/clay is underlain by a sand and gravel material, at an elevation of approximately -120 to -130 feet, that is likely to be glacial till.

At the top of the slope, we encountered approximately 25 feet of loose to medium dense sand/gravel fill material at borehole B1, before encountering the underlying native silt layer. This silt layer ranged from soft to medium stiff in density. In borehole B2 we encountered approximately 8 feet of loose sand/gravel fill material before encountering soft silt.

At the toe of the slope, we encountered approximately eight feet of loose to medium dense sand/gravel fill material before encountering the soft to medium dense underlying native silt. In boreholes B4 and B5 we observed a thin layer (approximately 1-2 feet in thickness) of peat between the sand/gravel and underlying silt.

The near surface materials (sand/gravel) were moderately frost susceptible (Frost classification F1 to F3).

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6.2 Groundwater

Groundwater depth varied across the project site. At the top of the slope, we were not able to determine the exact location of groundwater in borehole B1, but we observed groundwater at a depth of approximately 6.5 feet bgs in borehole B2.

At the toe of the slope, we observed groundwater between 6 feet bgs to 18 feet bgs.

6.3 Frozen Soils

We did not encounter any frozen soil (seasonal or permafrost) at the project site at the time of our subsurface exploration, and we do not expect permafrost to exist at any depth across the project site.

7.0 SEISMIC GROUND RESPONSE ANALYSIS

We used a computer program known as ProShake (from EduPro Civil Systems, Inc,) to perform a seismic response analysis for the soil column above the bedrock at the project site.

Table 1: Proshake Analysis Soil Profile

LAYER DEPTH BGS (ft)	MATERIAL TYPE	UNIT WEIGHT (pcf)	G_{MAX} (ksf)	V_S (fps)
0-10	Sand & Gravel	120	3613	984
10-50	Soft Bootlegger Clay	110	368	328
50-80	Medium Stiff Bootlegger Clay	110	575	410
80-200	Stiff Bootlegger Clay	110	828	492
200-700	Glacial Till	150	72263	3937
700	Bedrock	165	141315	5249

We used borehole logs from the project site to create a soil profile for the analysis (as shown in Table 1). The analysis requires values for unit weight and shear wave velocity for each soil layer. We estimated shear wave velocities using the correlation equations of Pitilakis et al. [1999] and the corrected blow count data listed on the logs contained in Appendix B. Unit weights for each material type are estimated based upon the material descriptions shown on the boring logs contained in Appendix B.

We used the ground motion record for the 2016 Iniskin Earthquake from the Glen Alps recording station [we obtained the data from the Center for Engineering Strong Motion Data (CESMD)] as the bedrock acceleration input motion and then scaled the motion to a maximum acceleration of 0.63 g (2475 year return interval at the project site) and 0.41 g (⅔ of the 2475 year return interval).

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We have provided plots of the input motions, surface displacement time history, and peak accelerations throughout the soil column in Figure 4 of this report.

The results from the analysis suggest that the ground surface accelerations during large seismic events at the project site are relatively insensitive to the bedrock acceleration. The soft silt/clay layers tend to attenuate any bedrock accelerations and it is unlikely that any ground surface accelerations will exceed 0.2 g. However, the ground surface displacements calculated by the program are, of the order of two feet during large seismic events (such as the scaled Iniskin earthquake in the this analysis).

8.0 SLOPE STABILITY ANALYSIS

We conducted a slope stability analysis to evaluate the existing slope stability with the proposed improvements.

8.1 SLOPE/W (GeoStudio 2012)

SLOPE/W is one component in a complete suite of geotechnical modeling software known as GeoStudio (produced by Geo-Slope International).

There are various methods available in SLOPE/W for calculating the factor of safety for a modeled slope. We used the Morgenstern-Price method in our slope stability analysis as it generally results in a lower (more conservative) calculated factor of safety than other analysis methods. The Morgenstern-Price method allows for various user-specified interslice force functions, including both shear and normal interslice forces, and satisfies both moment and force equilibrium.

SLOPE/W (by default) utilizes the half-sine interslice force function for the Morgenstern-Price method. The half-sine function tends to concentrate the interslice shear forces towards the middle of the sliding mass and diminishes the interslice shear forces in the crest and toe areas of the slope model.

SLOPE/W uses the concept of regions to define the model geometry. In basic terms, this means that a line (or boundary) is drawn around a soil unit or stratigraphic layer to indicate a distinctive soil profile and soil properties. For this project, we modeled the soil properties using the Mohr-Coulomb model.

SLOPE/W allows for the application of surcharge loads to simulate a pressure applied over a portion of the soil surface (e.g., to model a structure on the ground surface). The magnitude of the surcharge load applied is computed by multiplying the unit weight of the surcharge material by the vertical distance between the surcharge load and the ground surface.

We simulated Seismic loads using a pseudo-static approach where a seismic coefficient (k) is defined to generate a destabilizing horizontal force. The seismic loading is equal to the seismic

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coefficient (k) times the weight of the assumed failure wedge. We assumed the shear strength of the soil to be unaltered by seismic forces.

8.2 Model Configuration

We have presented a topographic map of the project site (provided by the client) in Figure 2 of this report which details the approximate location and orientation of the subsurface profiles that we modeled as a part of our slope stability analysis. We selected two cross sections and estimated the soil parameters based on our field explorations and laboratory testing results. We have presented both cross section profiles in Figures 5 and 6 of this report. The proposed structure locations are estimated and subject to change. We can re-conduct all analyses once the building layout is finalized.

The project site is located at the boundary of MOA Seismic Hazard Zones 4 and 5. We, therefore, used a seismic coefficient of 0.2g, as recommended by the MOA for Seismic Hazard Zones 4 and 5. Our Proshake analysis shows this value to be slightly conservative, but a reasonable assumption.

8.3 Analysis Results

In an effort to assess the slope stability with the proposed improvements at the project site, we ran analyses for four different loading conditions:

1) cross section A-A’ under static conditions;
2) cross section B-B’ under static conditions;
3) cross section A-A’ under a pseudo-static seismic load of 0.2g; and
4) cross section B-B’ under a pseudo-static seismic load of 0.2g.

We have provided graphical plots of all analyses in Figures 7 to 10 of this report. Our modeling and analysis efforts suggest that:

- The existing slope (at cross section A-A’) appears to be stable under static conditions with a minimum factor of safety of 2.5 for the soil parameters used and the slope angle as shown on Figure 7;
- The existing slope (at cross section B-B’) appears to be stable under static conditions with a minimum factor of safety of 1.7 for the soil parameters used and the slope angle as shown on Figure 8;
- The existing slope (at cross section A-A’) appears to be stable under a pseudo-static seismic load of 0.2g, for the soil parameters used with a minimum factor of safety of 1.1 (Figure 9);
- The existing slope (at cross section B-B’) appears to be stable under a pseudo-static seismic load of 0.2g, for the soil parameters used with a minimum factor of safety of 1.1 (Figure 10).

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9.0 ENGINEERING CONCLUSIONS

9.1 General Site Conclusions

Based on the findings of our field efforts, laboratory testing and computer modeling, it is our conclusion that the current subsurface materials which we observed across the project site are generally suitable to support the proposed improvements; provided that our concerns and recommendations that we present in this report are addressed by the design and construction processes.

The three primary concerns for the project site that should be considered during all stages of development are:

- Fill material – There is an unknown quantity of fill material at the site that has been placed over an unknown timeframe. We cannot be certain of the extent (both horizontal and vertical) of this fill. Nor, do we know the level of compactive effort (if any) that was applied to the fill during placement. The soil bearing capacities that we provide for the project site (as given in Section 10.2.1) have been adjusted accordingly to compensate for this uncertainty.
- Ground failure susceptibility – Although our slope stability analysis yielded a factor of safety greater than, or equal to 1.1, in our opinion, the project site has a moderate to high susceptibility for slope failure during a large seismic event, due to the soft Bootlegger Cove Silt/Clay present at depth (moisture content greater than the liquid limit in areas), the vertical relief across the project site and the land slide history of the surrounding areas. The project site will also likely experience large amplitude cyclical lateral movements during large seismic events. During these large cyclical movements, some lateral spreading may occur. Our recommendations for a foundation design are not intended to stop any structures from sliding or moving during such an event (such a design would not be economically feasible). Instead our design is intended to hold the structure together during any movements, such that the structure moves as a whole, thus limiting structural failure and catastrophic collapse.
- Peat/organic layer – We observed a 1 to 2 feet thick peat layer at approximately 5 to 6 feet bgs at the toe of the existing slope. Any peat located within the footprint of any pavement, foundations and/or gravity-fed utilities will need to be removed to its horizontal and vertical extents prior to construction.

9.2 Earthworks

In general, the primary earthworks planned for this project will likely consist of: 1) underground utility installation; 2) fill pad and site grading; and 3) pavement section construction. All earthworks should be completed with quality control inspection, including: bottom-of-hole inspections; fill gradation classification; and in-situ compaction testing. A bottom-of-hole inspection should be conducted by a qualified geotechnical engineer, geologist, or special

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inspector following site excavation activities (and before any underground utility construction begins) in order to visually confirm the findings of this report and provide recommendations for any non-conforming conditions encountered during the excavation activities.

Any peat/organic soil which is located within the footprint of the proposed foundations and/or gravity-fed utility alignment will need to be removed to its horizontal and vertical extent prior to construction.

9.3 Slope Stability

The computer analyses we conducted (as we describe in Section 7.0 and 8.0 of this report) generally supports our professional opinion, that while the project site did not undergo large transitional (sliding) movements in the 1964 Good Friday earthquake, the project site may experience large displacements during future large earthquakes. The magnitude of the movement will be a function of the earthquake magnitude and location, site-specific soil conditions, and groundwater conditions. If the site does not 'slide' then we expect horizontal ground movements (due to the soft silt layers) during a large seismic event to be of the order of two feet.

9.4 Foundations

As we discuss in Section 9.3 of this report and due to the risk for movement of the project site during a large seismic event, the most suitable foundation construction approach will be a shallow foundation utilizing a thickened edge reinforced concrete structural slab. Thickened edge reinforced concrete structural slabs are better suited for tolerating large motions than strip/spread footings.

Again, it should be noted that, it will not be economically feasible to design a foundation system for the project site that will not move during a large seismic event. Instead, the thickened edge reinforced concrete structural slab foundation is intended to move as a whole in order to prevent catastrophic failure of the building and loss of life. After such an event the building may be displaced from its original location, and/or be un-inhabitable. We provide more detailed recommendations for shallow foundation design in Section 10.2 of this report.

9.5 Underground Utilities

In general, the soils in which deep, gravity-fed utility trenches (6 to 10 feet bgs) are to be constructed consist of gravel/sand and/or silt. Buried utilities can be founded directly onto the native soils or properly placed and compacted structural fill. As mentioned above, the peat/organic soils that we observed at depths of approximately 5 to 6 feet bgs are not suitable for supporting gravity-fed utilities and, if encountered should be removed to the vertical and horizontal extent of the utilities. We provide more detailed recommendations for underground utility design in Section 10.4 of this report.

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9.6 Pavement

The existing near surface materials that we encountered during our exploration program are moderately frost susceptible (MOA Frost Classifications F1 to F3). The frost classification, combined with the relatively shallow groundwater table, leads us to expect a moderate risk of ice lens development at the site. Therefore, an appropriately designed pavement section will be required to help reduce the potential for pavement damage and extend the life of the pavement surface. We provide pavement section design recommendations in Section 10.5 of this report.

9.7 Settlements

Settlements for shallow foundations should be within tolerable limits, provided that they are placed directly onto the undisturbed sand/gravel material (or properly placed structural fill located directly above the undisturbed sand/gravel material We anticipate a total settlement for any thickened edge concrete structural slab foundation placed on either the undisturbed sand/gravel material and/or or structural fill placed above the undisturbed sand/gravel material (as we discuss in Section 10.1 of this report) to be less than three-quarters (3/4) of an inch, with differential settlements comprising about one-half (1/2) of the total anticipated settlement. Settlement amounts could increase substantially if the structural fill material used to bring any foundation pads to grade is not properly compacted. Most of the settlements should occur as the building loads are applied, such that additional long-term settlements should be relatively small and within tolerable limits.

Settlements under driveways and parking areas are expected to be vary more than under any buildings, especially where utility trenches are located. Proper earthwork is necessary to help reduce the settlement potential. The settlement potential can be reduced by performing all utility excavation and backfill efforts as early in the construction schedule as possible and placing any pavement as last in the construction schedule as possible.

9.8 Seismic Design Parameters

We have assumed that the International Building Code (IBC) 2012 will be used for the design of the proposed structure. Per IBC 2012, the site classification should be determined based on the average soil strength in the top 100 feet of the soil column. In our professional opinion, the seismic site classification for the project site is *E*. However, as is typical for geotechnical evaluation for project of this magnitude, our boreholes were advanced to a maximum depth of 52 feet bgs. Therefore, we used our local knowledge of this portion of the Anchorage Bowl (in assuming the strength of the materials present in the lower 48 feet) to come to a conclusion on the classification. Because of the assumption, we have calculated parameters for both site class D and also site class E, as depending upon the design of the buildings (see below), one site class will lead to a more conservative design than the other.

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We utilized the United States Geological Survey (USGS) Seismic Design Maps tool (http://earthquake.usgs.gov/designmaps/us/application.php) to calculate the seismic design parameters for the project site for both site class D and also E, which are:

Site Class D: $F_a = 1.000$ ($S_s = 1.500$) and $Fv = 1.500$ ($S_1 = 0.676$)

Site Class E: $F_a = 0.900$ ($S_s = 1.500$) and $F_v = 2.400$ ($S_1 = 0.676$)

A copy of the USGS Design Maps reports for both site class D and E for the project site is contained in Appendix D of this report. Appendix D also contains a figure (D-1) which compares the design response spectrum for both site class D and E. Should the fundamental frequency of the building be lower than 0.75 seconds, then the parameters for site class D should be used. Should the fundamental frequency of the building be greater than 0.75 seconds, then the parameters for site class E should be used. This approach will lead to the most conservative design (regarding seismic site classification).

Based on our findings, we expect there to be a low potential for soil liquefaction at the project site due to the subsurface materials present.

10.0 DESIGN RECOMMENDATIONS

We have presented our design recommendations in the general order that the project site will most likely be developed. Our design recommendations can be used in parts (as needed) for the final design configuration.

10.1 Earthworks

Our recommendations assume that any shallow foundations (i.e., poured-concrete footings) will be founded either directly onto the undisturbed sand/gravel material or compacted structural fill pads constructed directly above the undisturbed sand/gravel material. If the foundations are to be founded upon the silt material, a geofabric should be placed above the silt for separation, and two feet of gravel placed above that. The first foot of gravel should be compacted to 90% of the modified Proctor density, after that the gravel should be compacted to 95% of the modified Proctor density. Any other structural fill materials used on-site should be compacted to a minimum of 95 % of the modified Proctor density.

Any material removed during the initial site grading and excavation activities, which does not contain any organic/deleterious material, and has relatively low silt content (less than 15 percent passing the #200 sieve), can be re-used on-site as structural fill. Proper placement and compaction techniques need to be applied during the backfill process (see Section 11.1 of this report for more details). Additional laboratory testing may be required to verify the frost susceptibility of any excavated soil for use in shallow fill applications.

All earthworks should be completed with quality control inspection, including: bottom-of-hole inspections; fill gradation classification; and in-situ compacting testing. A bottom-of-hole

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inspection should be conducted by a qualified geotechnical engineer, geologist, or special inspector following site excavation activities (and before any foundation construction begins) in order to visually confirm the findings of this report and provide recommendations for any non-conforming conditions encountered during the excavation activities.

10.1.1 Shallow Foundations

As we mention in Section 9.4 of this report, we recommend a thickened edge reinforced concrete structural slab foundation to support any structures at the project site. The foundation should be capable of tolerating a two-foot cantilever load and two feet of lateral movement. Again, any peat/organic soil which is located within the footprint of the proposed foundations will need to be removed to its horizontal and vertical extent prior to construction.

10.1.2 Soil Bearing Capacity

Thickened edge concrete structural slab foundations placed on either the undisturbed sand/gravel material or on structural fill pads (constructed directly above the undisturbed sand/gravel material) may be designed for an allowable soil bearing capacity of 1,500 pounds per square foot (psf). The soil bearing capacity may be increased by one-third (1/3) to accommodate short-term wind and/or seismic loads.

10.1.3 Thickened Edge Foundations and Floor Slabs

Thickened edge concrete structural slab foundations and/or floor slabs can be founded directly onto the undisturbed sand/gravel material or properly placed structural fill located directly above the undisturbed sand/gravel material. As described in Section 10.1, if the foundation is to be placed on silt, then a geofabric and two feet of structural fill should be added.

Our recommended insulation and footing configurations for various shallow foundation and floor slab combinations is presented in Figure 11 of this report. For the project site, we recommend using configurations B or C (Figure 11) for a heated shallow foundation. Insulation may be placed beneath of the floor slab. However, no insulation should be placed under the thickened edge of any perimeter footings, as this can promote freezing of the foundation soils by preventing adequate heat transfer from the interior of the building to the foundation soils. Alternatively, insulation should be placed along the exterior of the thickened edge concrete structural slab to prevent freezing (and the associated frost heaving) of the foundation soils along the perimeter of the foundation.

As we mention in Section 10.1 of this report, the upper structural fill material (at or above the footing grade) used to construct the structural pad for a heated building should be relatively free draining (sands and gravels) with less than 15% of the fill material passing through a #200 sieve. Furthermore, the top six inches of the structural pad located beneath the slabs should be free draining, with less than 3% passing the #200 sieve. This "blanket" will serve as a capillary break to help maintain a dry slab.

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Slabs constructed directly on the undisturbed sand/gravel or on properly constructed granular fill pads (located directly above the undisturbed sand/gravel), as we described above, may be designed using a modulus of subgrade reaction of k_1=150 pci (k_1 is the value for a 1-ft x 1-ft rigid plate). For this project, the following equations can be used (with standard English units) to calculate the appropriate modulus of subgrade reaction for slabs bearing on the undisturbed sand/gravel or on properly placed granular structural fill located directly above the undisturbed sand/gravel:

$$k_{(B\,x\,B)} = k_1 \left(\frac{B+1}{2B}\right)^2 \quad (1)$$

Where:

B = the slab width of a square slab in feet
k_1 = the modulus of subgrade reaction for a 1ft × 1ft rigid plate in pci
$k_{(B\,x\,B)}$ = the modulus of subgrade reaction for a square slab of width B in pci

The following equation (2) can be used for a rectangular slab having the dimensions $B \times L$ (in feet) with similar bearing soils as the equation above (1).

$$k_{(B\,x\,L)} = \frac{k_{(B\,x\,B)}\left(1+0.5\frac{B}{L}\right)}{1.5} \quad (2)$$

Where:

$k_{(B\,x\,B)}$ = the modulus of subgrade reaction for a $B \times B$ square slab
$k_{(B\,x\,L)}$ = the modulus of subgrade reaction for $B \times L$ rectangular slab
B = the least horizontal dimension of a rectangular slab
L = the larger horizontal dimension of a rectangular slab

10.1.4 Cold (Unheated) Shallow Foundations

It is difficult to predict the depth of frost penetration and extent of ice lens formation at any given site. Therefore, we do not recommend the construction of cold (unheated) shallow foundations as the formation of ice lenses beneath of a foundation can result in deformation to the overlying foundation.

Cold (unheated) shallow foundations should be placed on granular structural pads constructed of NFS fill material with a minimum thickness of five feet (NFS material should have less than 6% of the material passing a #200 sieve). Insulation may be incorporated into the foundation design to help protect the foundation soils from freezing. Insulation may be used in lieu of some of the NFS backfill. In terms of insulating properties, one inch of rigid board insulation can be considered equivalent to one foot of NFS fill. Our recommended insulation and footing configurations for cold shallow foundations are provided in Figure 11 of this report (configuration A).

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10.1.5 Lateral Loads for Foundation and Retaining Walls

Retaining walls (such as perimeter foundation stem walls for buildings with basements or crawl spaces) must be designed to resist lateral earth pressures. The magnitude of the pressure exerted on a retaining wall is dependent upon several factors, including:

1) whether the wall is allowed to deflect after placement of backfill;
2) the type of backfill used;
3) compaction effort; and
4) wall drainage provisions.

An active-earth pressure condition will prevail (under static loading) if a retaining wall is allowed to deflect or rotate a minimum of 0.001 times by the wall height. An at-rest pressure condition will prevail if a retaining wall is restrained at the top and cannot move at least 0.001 times the wall height. Lateral forces exerted by wind or seismic activity may be resisted by passive-earth pressures against the sides of the foundation footings, exterior walls (below grade), and grade beams.

In order to prevent water accumulation against the outside of any foundation or retaining wall, the wall must have a perimeter drainage system connected to an outlet that will not freeze closed at any time of the year. The top of the drainage piping must be located below the top of the footing for the foundation and/or retaining wall. Backfill used against the wall (and extending a minimum of one foot beyond the wall) must be free-draining with less than three percent fines. The top one-foot of backfill against the outside of a foundation and/or retaining wall should consist of relatively impermeable (fine-grained) material and be tightly compacted such that surface water is directed away from the foundation and/or retaining wall. A permeable geotextile fabric may be useful to prevent mixing of the impermeable (fine-grained) overburden and underlying free-draining (coarse-grained) backfill. Furthermore, the finished surface should slope away from any foundation and/or retaining wall with a minimum grade of 2 %, such that surface water is directed away from the foundation and/or retaining wall.

Seismic loading on foundation and/or retaining walls generally increases the lateral pressures on the wall and decreases the passive resistance. For foundation systems where the building foundation is continuous, the differential lateral movement between the soil and foundation is very small, and as such, essentially no excess lateral loading on the foundation wall is experienced. Foundation walls with a differential in backfill heights of over six feet (basements, crawl spaces, etc.) will experience seismic lateral loading from the inertial effects of seismic waves passing through the foundation.

The lateral soil pressures can be represented by equivalent fluid pressures. The pressure distribution is a function of wall restraint, seismic loading, and drainage conditions. Figure 12 presents the distribution diagrams for various loading conditions (for retaining walls less than eight feet high). Table 2 presents the unit weights to be used with Figure 12 for this project.

Any foundation stem walls that are not designed to carry lateral loads should be backfilled on both sides simultaneously to prevent differential lateral loading of the foundation stem wall. We developed the unit weights provided in Table 2 of this report assuming that structural fill (containing less than ten percent fines) is used as backfill, and that the fill is compacted to at least 90 percent of the Modified Proctor density.

Table 2: Equivalent Fluid Specific Weight for Lateral Loading Design

Equations only valid for units of pcf (t_1-t_8) and ft (H and H_1).

LOADING CONDITION	DRAINED EQUIVALENT FLUID SPECIFIC WEIGHT		UN-DRAINED EQUIVALENT FLUID SPECIFIC WEIGHT	
	SPECIFIC WEIGHT (pcf)	SYMBOL USED IN FIGURE 12	SPECIFIC WEIGHT (pcf)	SYMBOL USED IN FIGURE 12
ACTIVE	40	t_1	28	t_2
AT-REST	55	t_3	38	t_4
PASSIVE	300	t_5	225	t_6
SEISMIC	16	t_7	9	t_8

Lateral forces may also be resisted by friction between the concrete foundations and the underlying soil. The frictional resistance may be calculated using a coefficient of friction of 0.4 between the concrete and soil.

For restrained walls higher than eight feet, the methods used to calculate the unit weights in Table 2 become non-conservative. The calculations and methodology for determining the pressure loads and distribution for walls greater than eight feet in height are presented in Appendix E. As an example, we have included the calculation for a nine feet high restrained wall (Appendix E and Figure 13). We can recalculate the pressure distribution after the restrained retaining wall height is determined.

10.2 Insulation

Any subsurface insulation should consist of extruded polystyrene such as DOW Styrofoam™ Highload or UC Industries Foamular. Any subsurface insulation used under pavement sections or structural slabs should be closed cell, board stock with a minimum compressive strength of 60 psi at five percent deflection. Subsurface insulation around not subject to structural or lvehicular loading, should have a minimum compressive strength of 25 psi at five percent deflection. The insulation should not absorb more than two percent water per ASTM Test Method C-272. The thermal conductivity (k) of the insulation should not exceed 0.25 BTU-in/hr-ft^2-°F when tested at 75 °F.

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10.3 Underground Utilities

In general, the soils in which deep utility trenches (6 to 10 feet bgs) are to be constructed are composed of sand/gravel or silt. Any gravity-fed utility trenches extending into the sand/gravel or silt should be a minimum of three feet wide at the bottom with the utility piping located in the center of the trenches. Any peat/organic soil which is located within the footprint of the proposed gravity-fed utility alignment will need to be removed to its horizontal and vertical extent prior to construction. If the utilities are to be founded upon the silt material, a geofabric should be placed above the silt for separation, and two feet of gravel placed above that. The first foot of structural fill should be compacted to 90% of the modified Proctor density, after that the structural fill should be compacted to 95% of the modified Proctor density. Structural fill should be used to bring the gravity-fed utilities to the proper installation grade. Utilities that are not sensitive to settlement may be placed in the existing sand/gravel material.

Underground utilities which are susceptible to damage from freezing need to be frost-protected by sufficient amounts of backfill, insulation, and/or active freeze protection systems (e.g., heat tape, thaw wire, etc.); or some combination of the above. Any utilities which are susceptible to damage from freezing that are planned to be constructed less than eight feet below the planned finished grade should contain some level of additional frost-protection (e.g., insulation, active freeze protection systems, or a combination of both).

Any insulation used should conform to the specifications detailed in Section 10.3 of this report and should extend a minimum of two feet (and a maximum of four feet) perpendicular to either side of the proposed utility alignment. The thickness of the insulation used will be a function of the burial depth. In general one inch of insulation is equal to approximately 12 inches of compacted NFS backfill. Underground utilities which are susceptible to damage from freezing should not be constructed within four feet of the planned finished grade (regardless of insulation measures or active freeze-protection systems).

10.4 Pavement Section

Construction of the pavement section will be guided in part by the amount of cut/fill needed to achieve the final grade. The existing near surface materials are moderately frost susceptible (MOA frost classification F1-F3). This will require an appropriately engineered pavement section in order to help reduce the potential for future pavement damage and prolong the life of the proposed parking areas. In addition, all peat/organic soil should be removed to the vertical and horizontal extents of any pavement sections, if any fill is required it should be structural fill with a frost classification of F2 or better and placed according to Section 10.1 of this report. Confirmation testing of the subgrade soils along the proposed pavement section should be conducted after the completion of utility installation in order to confirm the frost classification of the subgrade soils. We present two recommended pavement sections in Tables 3 and 4 of this report. Table 3 provides an appropriate pavement section if a curb, gutter, and storm drain

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system is present on-site. Table 4 provides an appropriate pavement section if those systems are not on-site.

Table 3: Suitable Pavement Section Construction with Curb, Gutter and Storm Drain Present On-site

Section Thickness	Material
2 inches min.	Asphalt (concrete pavement thickness will be a function of reinforcement)
2 inches max.	NFS leveling course (RAP or "D-1")
12 inches	Type II A
12 inches	Type II
N/A	F2 or better Structural Fill
N/A	Existing frost susceptible soils (F1 or F3)

The leveling course, Type IIA, and Type II used should conform to the MOA specifications we provide in Figure 14 of this letter. The Type II material should not be placed within eight inches of any leveling course surface, as it may affect the long-term smoothness of the asphalt surface. As the Type II material settles/consolidates (from vehicle traffic, etc.), larger particles (3-8 inches in diameter) can protrude into the overlying leveling course and produce a lumpy or dimpled asphalt surface. Therefore, a layer (at least eight inches in thickness) of Type IIA material (which has a maximum particle size of three inches) should always be used to separate the leveling course from underlying coarse-grained (e.g., Type II) materials.

Table 4: Suitable Pavement Section Construction without Curb, Gutter and Storm Drain Present On-site

Section Thickness	Material
2 inches min.	Asphalt (concrete pavement thickness will be a function of reinforcement)
2 inches max.	NFS leveling course (RAP or "D-1")
12 inches	Type II A
18 inches	Type II
N/A	F2 or better Structural Fill
N/A	Existing frost susceptible soils (F1 or F3)

Any leveling course used should be NFS in order to maintain a low potential for ice lens development within the leveling course. It is our experience that the "D1" leveling course material currently available in the Anchorage area may not be NFS following compaction, because compaction with a vibratory compactor may increase the frost susceptibility of the

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leveling course by increasing the percentage of fine-grained material (due to degradation of the soil particles from the impact of the compaction equipment). As such, the leveling course thickness should be kept to two inches or less to reduce the potential for ice lens formation in the leveling course. All of these materials should be placed in thin lifts and each lift should be compacted to a minimum of 95 % of the modified Proctor density. As an alternative to "D1", recycled asphalt pavement (RAP) can be used. The residual oil in the RAP greatly reduces the frost susceptibility.

10.5 Surface Drainage

After the property is brought to grade it should be relatively flat, such that storm water will tend to accumulate and flow off the project site slowly. Water accumulation will have a detrimental effect on foundations, retaining structures, and pavement sections. Provisions should be included in the design to collect runoff and divert it away from any foundations, retaining structures, and pavement sections. The ground surface surrounding the proposed developments should be graded such that surface runoff is channeled away from foundations, retaining walls, and pavement sections. The soils on the surface should be tightly compacted to help reduce surface runoff infiltration. Roof, parking lot, and driveway drainage should be directed away from foundations. If storm sewer is available, tight-line connections from roof drain collectors should be made.

11.0 CONSTRUCTION RECOMMENDATIONS

We have presented our construction recommendations in the general order that the project site will most likely be developed. Our construction recommendations are intended to aid the construction contractor(s) during the construction process.

11.1 Earthwork

The first lift of fill material placed directly upon the underlying silt can be compacted to 90 percent of the modified Proctor density (unlikely to achieve 95 percent without disturbing underlying silt), as determined by ASTM D-1557. Subsequently, any and all fill material used should be placed at 95 percent of the modified Proctor density, unless specifically stated otherwise in other sections of this report. The thickness of individual lifts will be determined based on the equipment used, the soil type, and existing soil moisture content. Typically, fill material will need to be placed in lifts of less than one-foot in thickness. All earthworks should be completed with quality control inspection.

In our professional experience, structural fill should have less than approximately 15 percent passing the #200 sieve for ease of placement. Soils with higher silt contents can be used within the foundation footprint. However, the effort required to achieve proper compaction of silt-rich soils may be more costly than purchasing better grade materials. The time of year, existing moisture content, rainfall, air temperature, and fill temperature can all have an impact on the effort required to adequately compact silt-rich material.

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Any excavated fill or on-site soils (which are free of organic material and have relatively low silt contents) which are stockpiled on-site (for later use as structural backfill) should be protected from additional moisture inputs (precipitation, etc.) through the use of plastic tarps, etc. Additional moisture inputs can have detrimental effects on the effort needed to achieve proper compaction rates.

11.2 Heated Shallow Foundations

Care should be taken during foundation excavation activities to limit the disturbance of the bottom of any foundation excavations. The bottom of any foundation excavation should be moisture conditioned and proof-rolled as necessary to return the exposed soils to their original in-situ density.

In general, the soils in which the proposed foundation pads are to be constructed consist primarily sand/gravel material. As such, any surface water (*e.g.*, from precipitation, snowmelt, etc.) that enters into foundation excavations will tend to dissipate relatively quickly. Excess water will have a negative impact on any backfill and compaction efforts. Therefore, if surface water does accumulate in any open foundation excavations it can be controlled by excavating a shallow drainage trench around the perimeter of the excavation. The drainage trench will collect surface water and direct it to a sump area, which should be located outside of the foundation footprint. The excess water can then be pumped from the sump area and be discharged at an appropriate location away from the excavation and any other existing foundations.

It is imperative that shallow building foundations for heated structures remain in a thawed state for the entire construction period; even when dealing with soils that have little to no frost susceptibility. Foundation soils that are allowed to freeze during the initial construction (before the building is enclosed and heated) may be compromised by the development of ice lenses. Upon thawing, which may take several weeks or months, potential differential settlements could distort the structure resulting in damaged foundations, cracked sheetrock, skewed door frames, and broken windows. If construction extends into the winter months, temporary enclosures should be constructed which completely enclose warm foundations and heat should be applied to the enclosure to prevent freezing of the soils located beneath any warm foundation and/or floor slab.

11.3 Unheated Shallow Foundations

The frost susceptibility of the foundation soils range from F1 to F3. Therefore, the existing frost susceptible soils are unsuitable to support any cold (unheated) shallow foundations without freeze protection, as they may experience ice lens development and/or thaw-weakening, which could result in damages to the proposed foundations. As we mention in Section 10.2.3 of this report, cold foundations should be placed on a five-foot thick structural pad constructed of NFS fill. The NFS structural pad thickness may be reduced by using insulation at a rate of one inch of insulation to one foot of NFS material.

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11.4 Insulation

The satisfactory performance of any subsurface insulation is in part controlled by the details of construction including: 1) the care taken to ensure that the board stock lies flat on a smooth, level surface; and 2) the adjoining ends of the insulation are closely butted together. Any vertical joints should be staggered where more than one layer of insulation is used.

11.5 Underground Utilities

We expect that utility trench wall stability in the sand/gravel material will be poor, especially where utility trenches extend below the groundwater table. The contractor should be responsible for trench safety and regulation compliance. If groundwater is encountered during utility trench excavation then dewatering efforts may be required to facilitate proper utility installation and trench backfill.

All piping should be bedded per the manufacturer's recommendations, with the bedding material compacted to provide pipe support. Above the bedding materials, the backfill should be similar to, and compacted to the approximate density of, the surrounding soils.

11.6 Pavement

All of the earthwork within any areas to be paved should be completed as early in the construction schedule as possible, and the pavement placed as late in the construction schedule as possible. This will give the subgrade soils time to settle, compress, and stabilize prior to placement of the pavement. Any structural fill used should be placed in thin lifts (less than one foot in thickness) and each lift should be compacted to a minimum of 95 % of the modified Proctor density. Prior to paving, any surface fill material should be re-leveled and re-compacted. All backfill and paving materials should be inspected and tested for material specification compliance and compaction.

Underground utility piping should be installed prior to construction of any pavement sections such that trenching is done through the subgrade soils only. This will help ensure that a uniform pavement section is maintained, which will reduce the potential for differential settlements along underground utility trench alignments.

The minimum thickness for any asphalt pavement surfaces is two inches. The minimum thickness of any concrete pavement surfaces will be a function of the reinforcement required. All applicable ACI and IBC standards should be followed.

11.7 Winter Construction

Proper placement and compaction of structural fill is not possible when fill material is frozen, and as such, frozen fill material should never be used for structural support unless it has been subsequently thawed and compacted to 95 % of the modified Proctor density (throughout its vertical extent). Furthermore, subgrade soils (fill or native) need to be completely thawed prior

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to the placement and compaction of additional lifts of thawed fill material. In our professional experience, ambient soil temperatures need to be above 37 °F in order to achieve efficient compaction. It is extremely difficult to achieve compaction levels equal to 95 percent of the modified Proctor density in fill material that is between 32 °F to 37 °F. We discuss the risks associated with winter foundation construction in more detail in Sections 11.2 of this report

12.0 THE OBSERVATIONAL METHOD

A comprehensive geoprofessional service (e.g., geotechnical, geological, civil, and/or environmental engineering, etc.) should consist of an interdependent, two-part process comprised of:

Part I - pre-construction site assessment, engineering, and design; and

Part II - continuous construction oversight and design support.

This process, commonly referred to in the geoprofessional industry as "The Observational Method", was developed to reduce the costs required to complete a construction project, while simultaneously reducing the overall risk associated with the design and construction of the project.

In geotechnical engineering, Part I of the Observational Method (OM) begins with a geotechnical assessment of the site, which typically consists of some combination of literature research, site reconnaissance, subsurface exploration, laboratory testing, and geotechnical engineering. These efforts are usually documented in a formal report (e.g., this report) that summarizes the findings of the geotechnical assessment, and presents provisional geotechnical engineering recommendations for design and construction. Geotechnical assessment reports (and the findings and recommendations contained within) are considered provisional due to the fact that their contents are typically based primarily on limited subsurface information for a site. Most conventional geotechnical exploration programs only physically characterize a very small percentage of a given site, as it is typically cost prohibitive to conduct extensive (i.e. high density/frequency) exploration programs. As an alternative, geoprofessionals use the subsurface information available for a site to extrapolate subsurface conditions between exploration locations and develop appropriate provisional recommendations based on the inferred site conditions. As a result, the geoprofessional of record cannot be certain that the provisional recommendations will be wholly applicable to the site, as subsurface conditions other than those identified during the geotechnical assessment may exist at the site which could present obstacles and/or increased risk to the proposed design and construction.

Part II of the OM is employed by geoprofessionals to help reduce the risk associated with unidentified and/or unexpected subsurface conditions. Geoprofessionals accomplish Part II of the OM by providing construction oversight (e.g., construction observation, inspection, and testing). Part II of the OM is a valuable service, as the geoprofessional of record is available if unexpected conditions are encountered during the construction process (e.g., during excavation,

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fill placement, etc.) to make timely assessments of the unexpected conditions and modify their design and construction recommendations accordingly; thus reducing considerable cost resulting from potential construction delays and reducing the risk of future problems resulting from inappropriate design and construction practices.

Oftentimes, a client may be persuaded to use an alternative geoprofessional firm to conduct Part II of the OM for a given project; as some geoprofessional firms offer the same services at discounted prices in order to help them obtain the overall construction materials engineering and testing (CoMET) commission. The geoprofessional industry as a whole recommends against this practice. An alternative geoprofessional firm cannot provide the same level of service as the geoprofessional of record. The geoprofessional of record has (amongst other things) a unique familiarity with the project including; an intimate understanding of the subsurface conditions, the proposed design, and the client's unique concerns and needs, as well as other factors that could impact the successful completion of a construction project. An alternative geoprofessional firm is not aware of the inferences made and the judgment applied by the geoprofessional of record in developing the provisional recommendations, and may overlook opportunities to provide extra value during Part II of the geoprofessional service.

Clients that prevent the geoprofessional of record from performing a complete service can be held solely liable for any complications stemming from engineering omissions as a result of unidentified conditions. The geoprofessional of record may not be liable for any resulting complications that occur, as the geoprofessional of record was not able to complete their services. Furthermore, the replacement geoprofessional firm may also be found to have no liability for the same reasons.

We are available at any time to discuss the OM in more detail, or to provide you with an estimate for any additional construction observation and testing services required.

13.0 CLOSURE

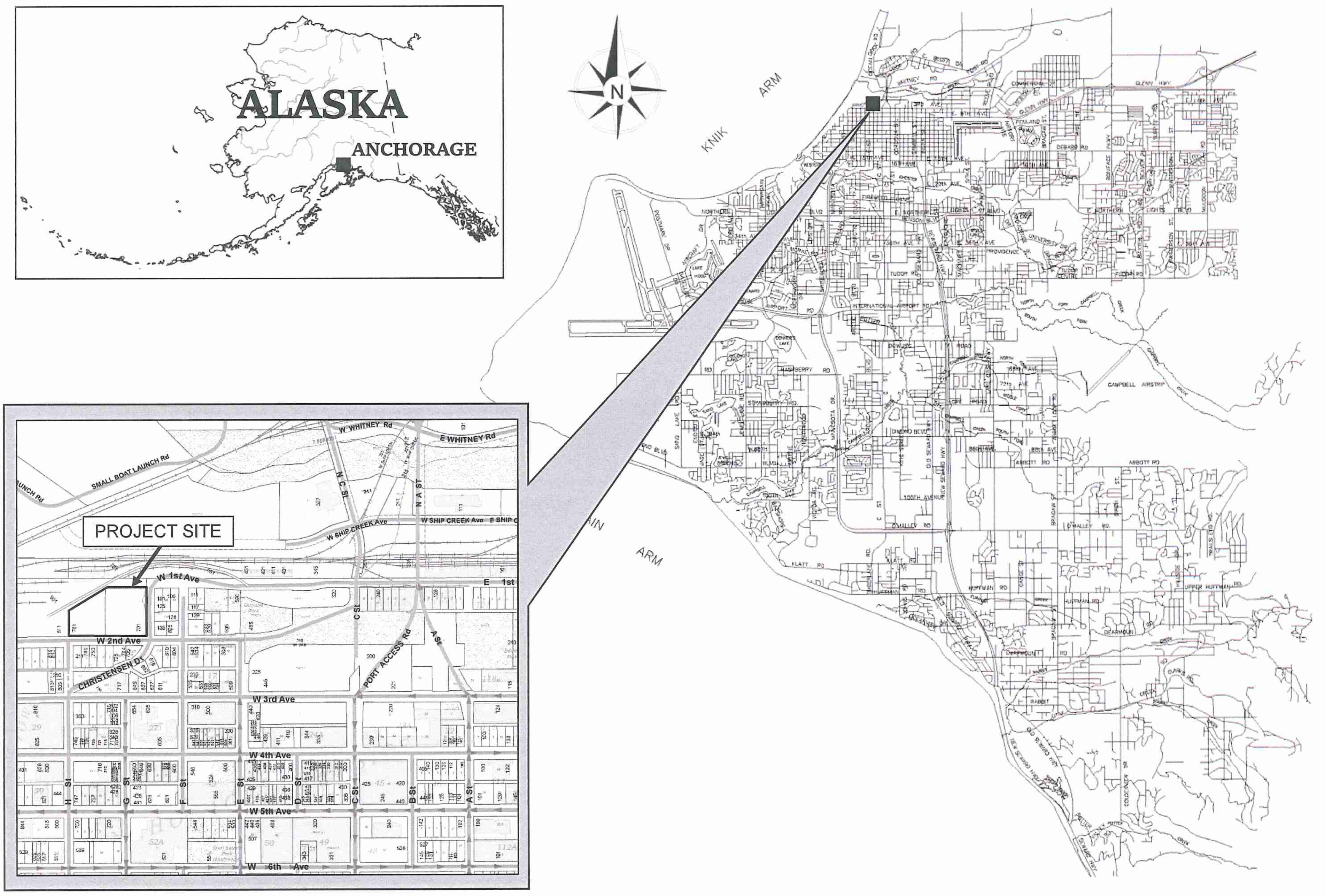
We (Northern Geotechnical Engineering, Inc. d.b.a. Terra Firma Testing) prepared this report exclusively for the use of John McGrew and his consultants/contractor/etc. for use in the design and construction of the proposed improvements. We should be notified if significant changes are to occur in the nature, design, or location of the proposed improvements in order that we may review our conclusions and recommendations that we present in this report and, if necessary, modify them to satisfy the proposed changes.

Due to the natural variability of earth materials, variations in the subsurface conditions across the project site may exist other than those we identified during the course of our geotechnical assessment. Therefore, we recommend that a qualified geotechnical engineer, geologist, and/or special inspector be on-site during construction activities to provide corrective recommendations for any unexpected conditions revealed during construction (see our discussion of the Observational Method in Section 12.0 of this report for more detail). Furthermore, the construction budget should allow for any unanticipated conditions that may be encountered during construction activities.

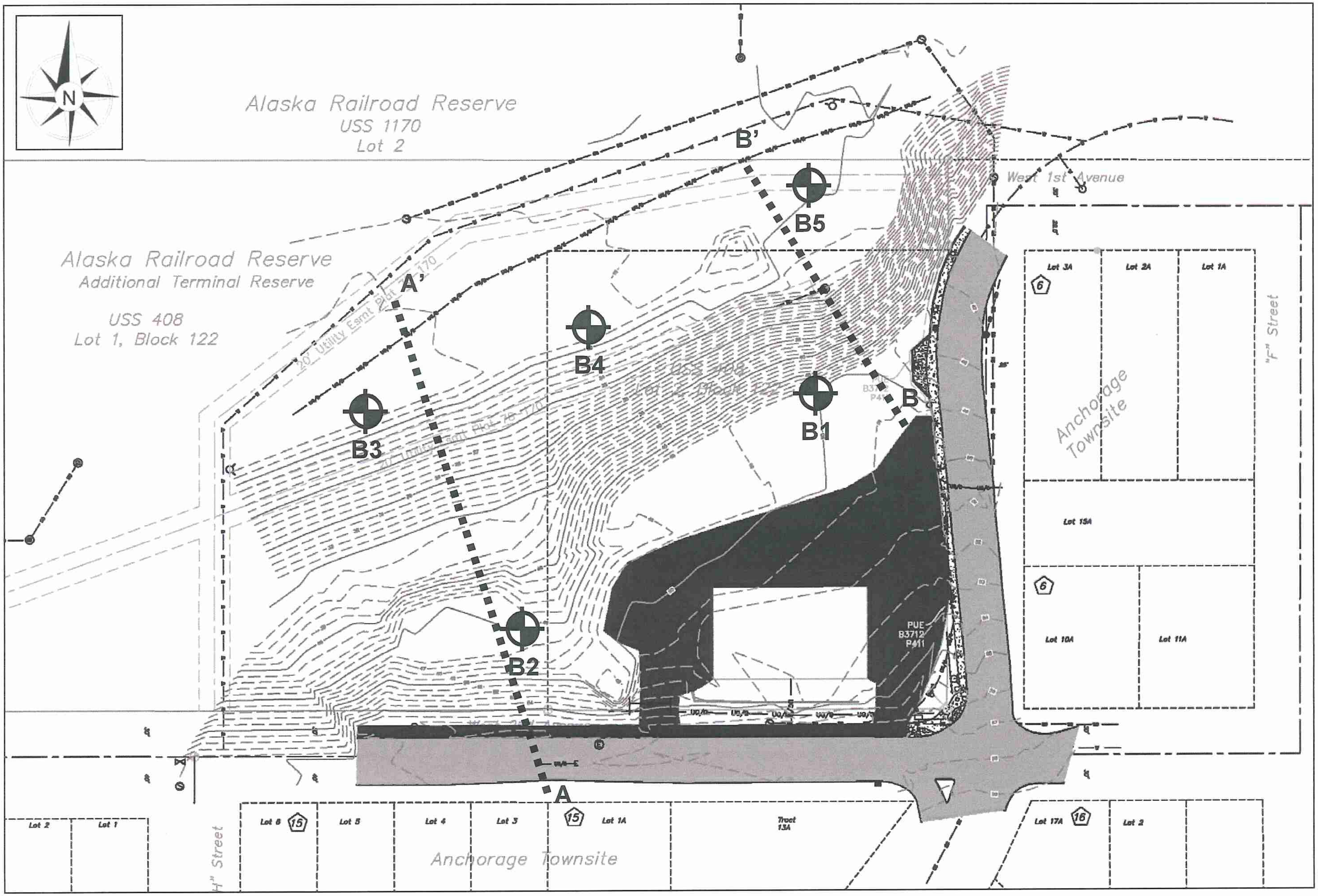
We conducted this evaluation following the standard of care expected of professionals undertaking similar work in the State of Alaska under similar conditions. No warranty, expressed or implied, is made.

11301 Olive Lane Anchorage, Alaska 99515 · Phone: (907) 344-5934 · Fax: (907) 344-5993 · Website: www.nge-tft.com

REPORT FIGURES

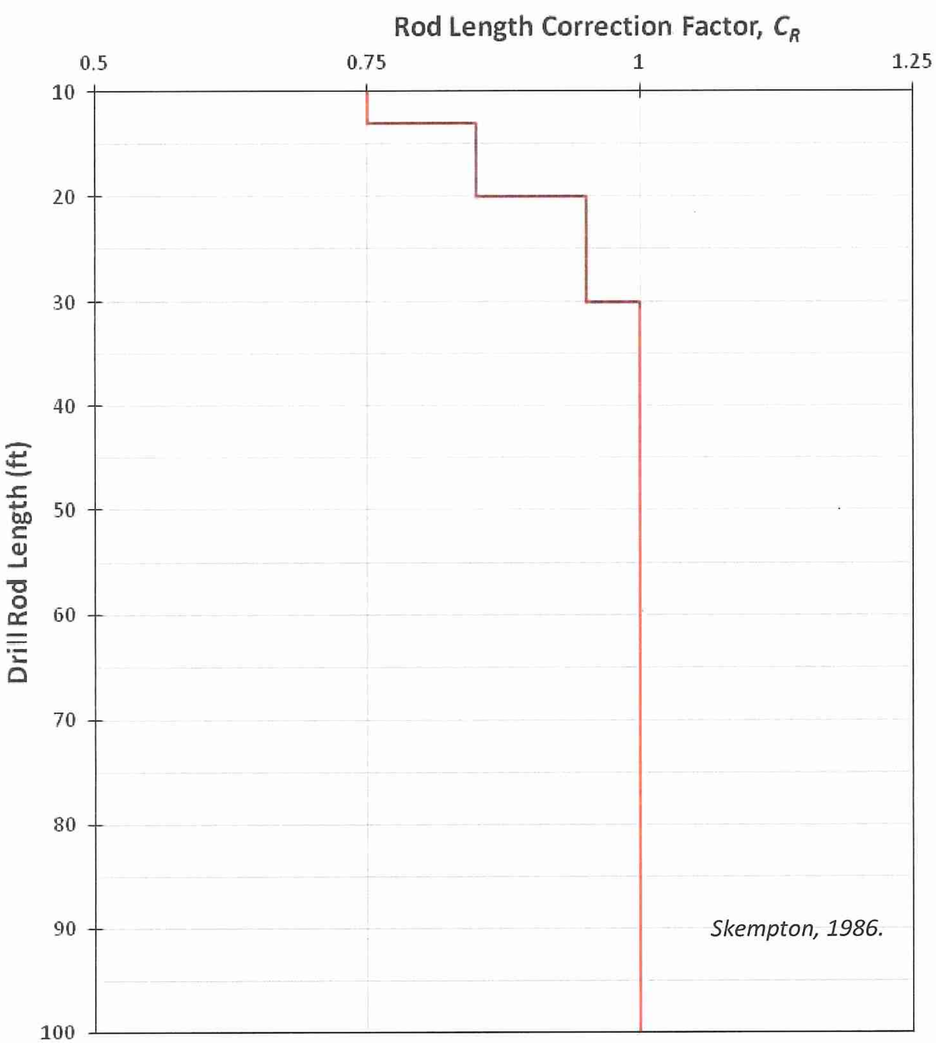


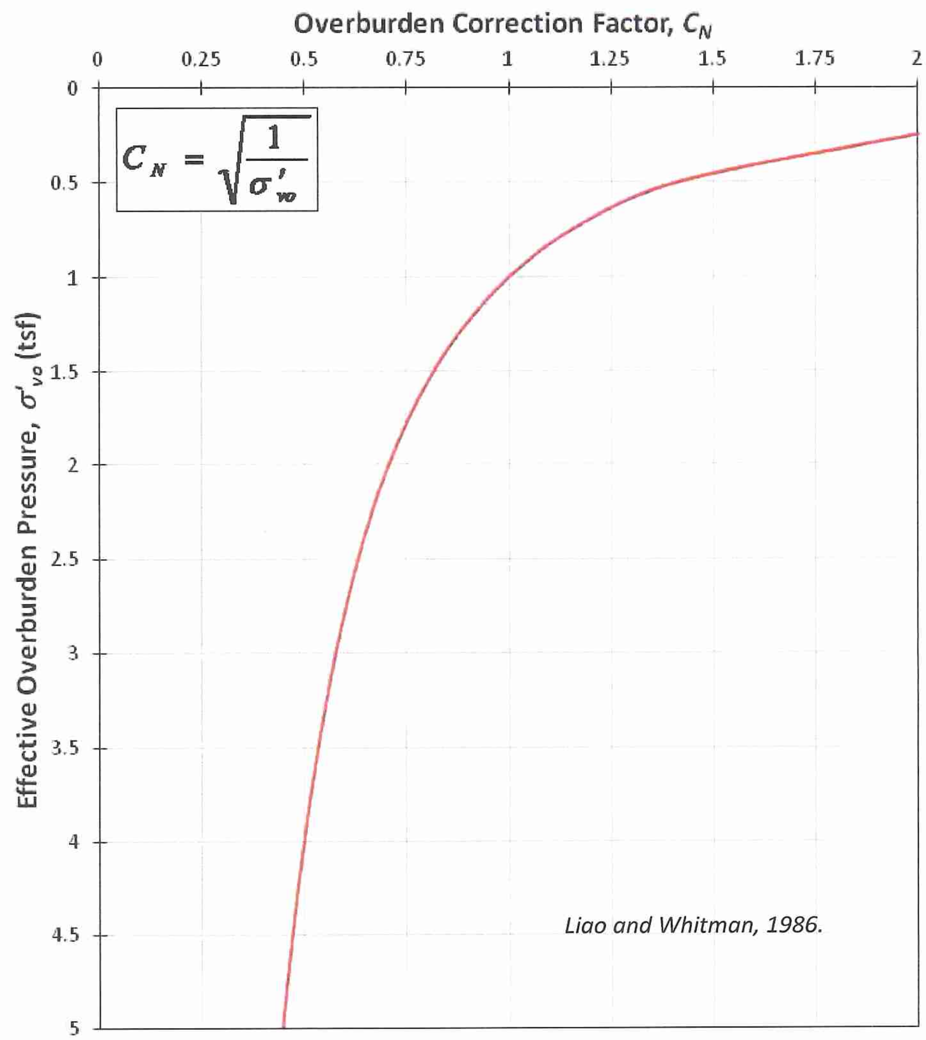
NORTHERN GEOTECHNICAL ENGINEERING, INC. TERRA FIRMA TESTING	FIGURE TITLE: PROJECT SITE LOCATION MAP
	PROJECT NAME: SHIP CREEK DEVELOPMENT — PROJECT ID: 4385-16
	PROJECT LOCATION: ANCHORAGE, AK — FIGURE NUMBER: 1



NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: EXPLORATION LOCATION MAP		
PROJECT NAME: SHIP CREEK DEVELOPMENT		PROJECT ID: 4385-16
PROJECT LOCATION: ANCHORAGE, AK		FIGURE NUMBER: 2



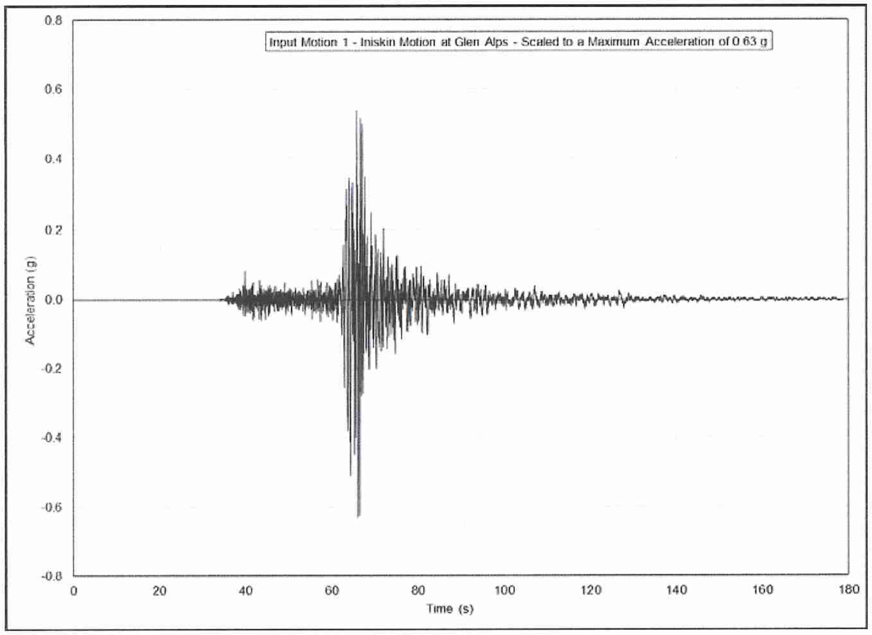


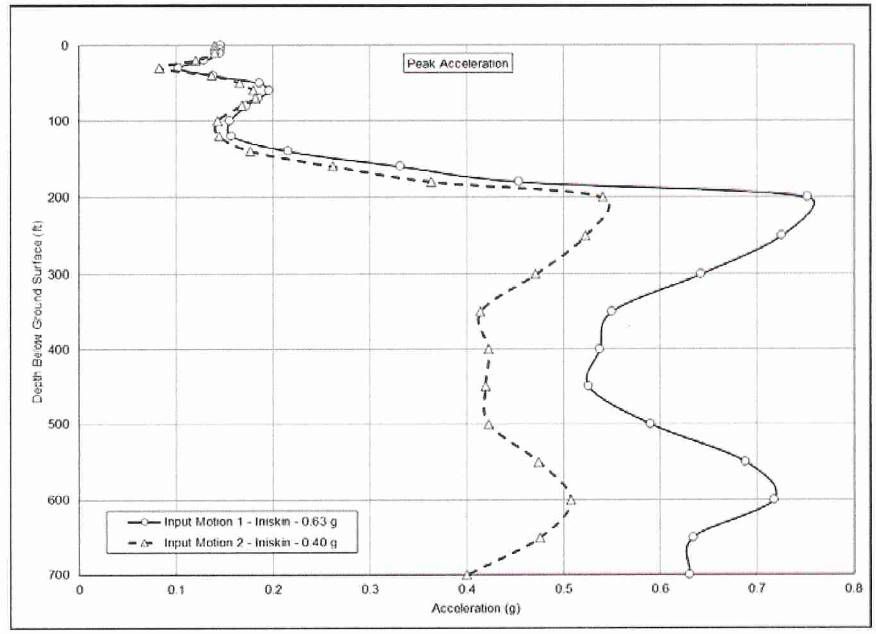
Notes:

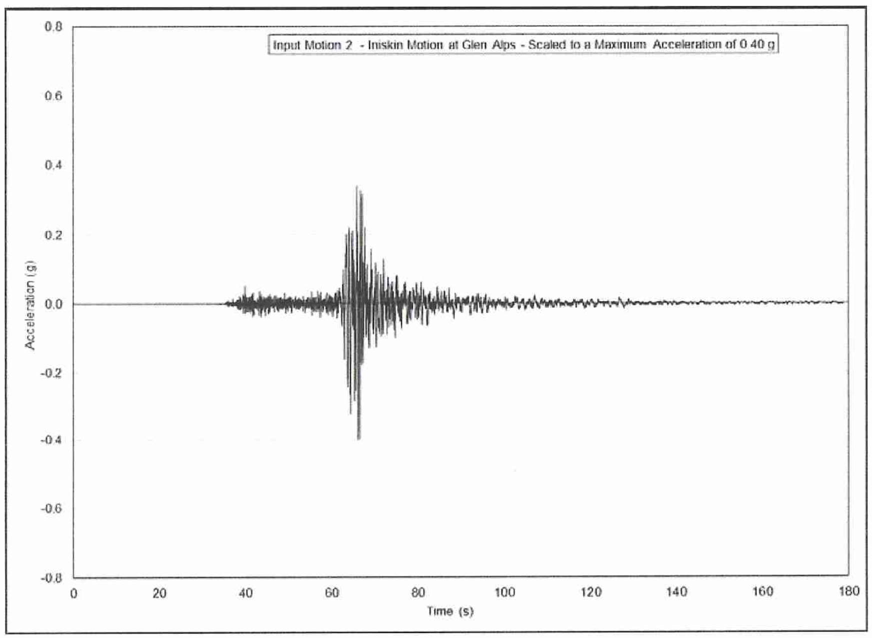
- Overburden correction factor is used only for cohesionless soils
- C_N is the ratio of the measured blow count to what the blow count would be at an overburden pressure of 1 ton/ft^2
- σ'_{vo} is the effective oberburden pressure at the point of measurement (ton/ft^2)

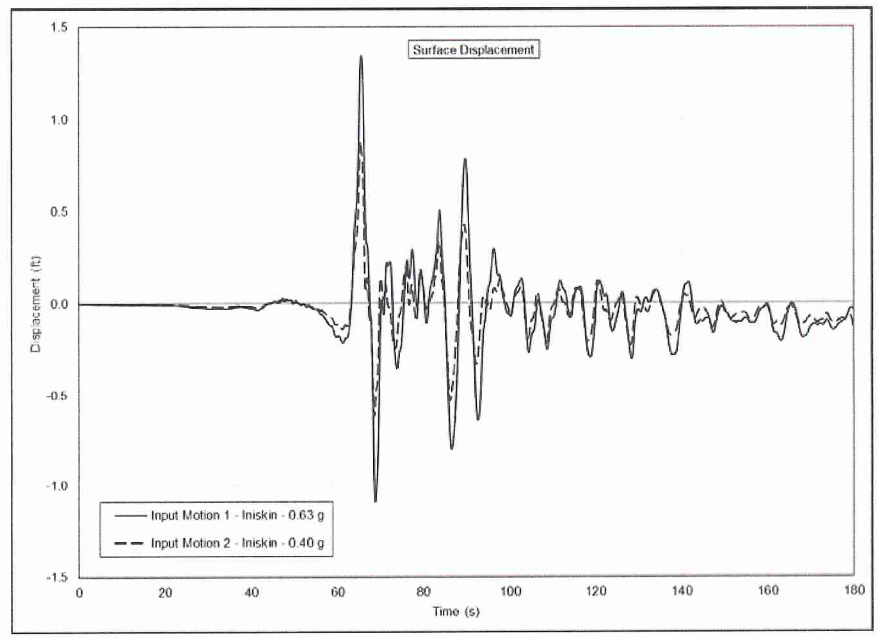
NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE:	BLOW COUNT CORRECTIONS		
PROJECT NAME:	SHIP CREEK DEVELOPMENT	PROJECT ID:	4385-16
PROJECT LOCATION:	ANCHORAGE, AK	FIGURE NUMBER:	3



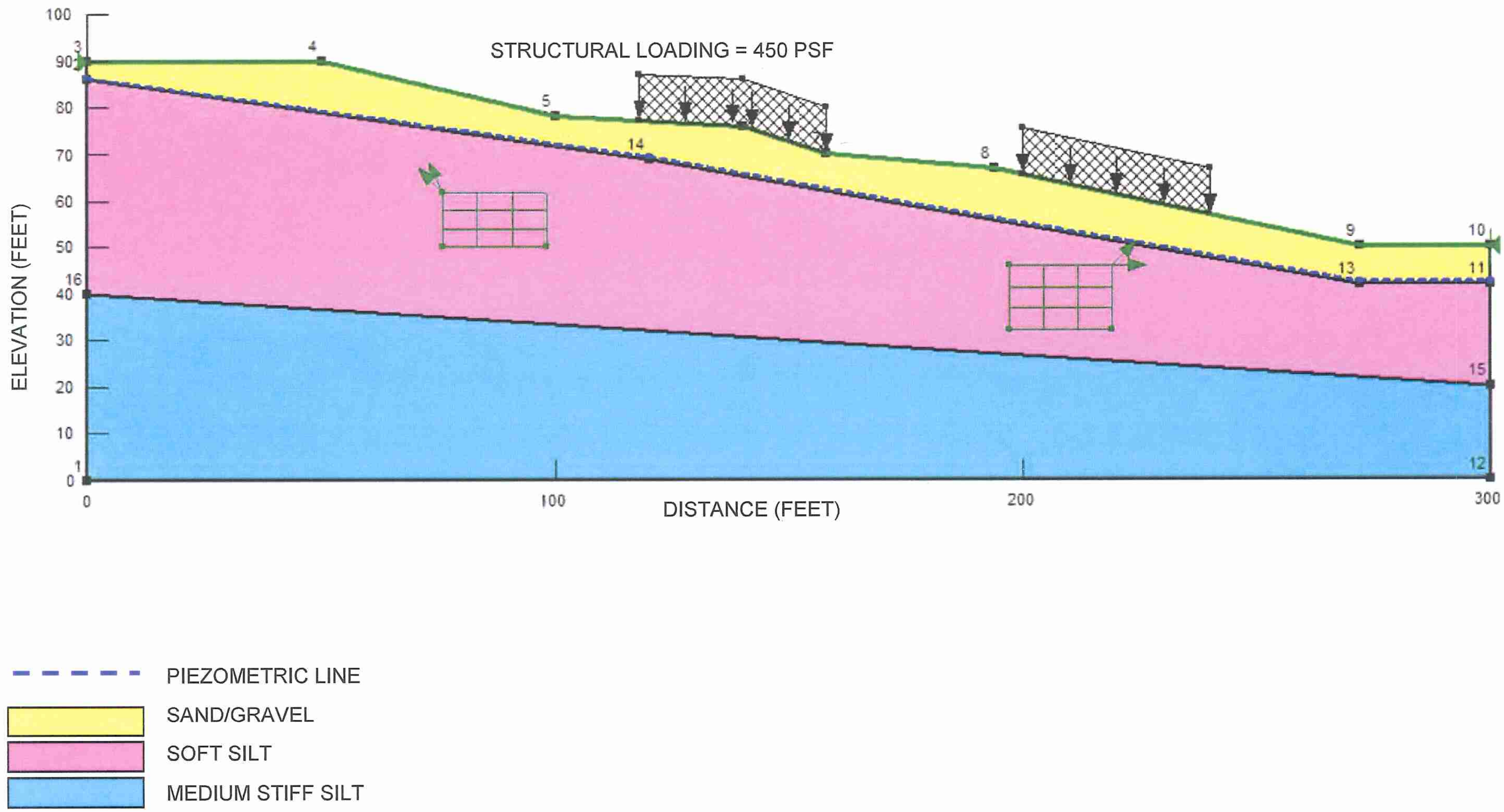






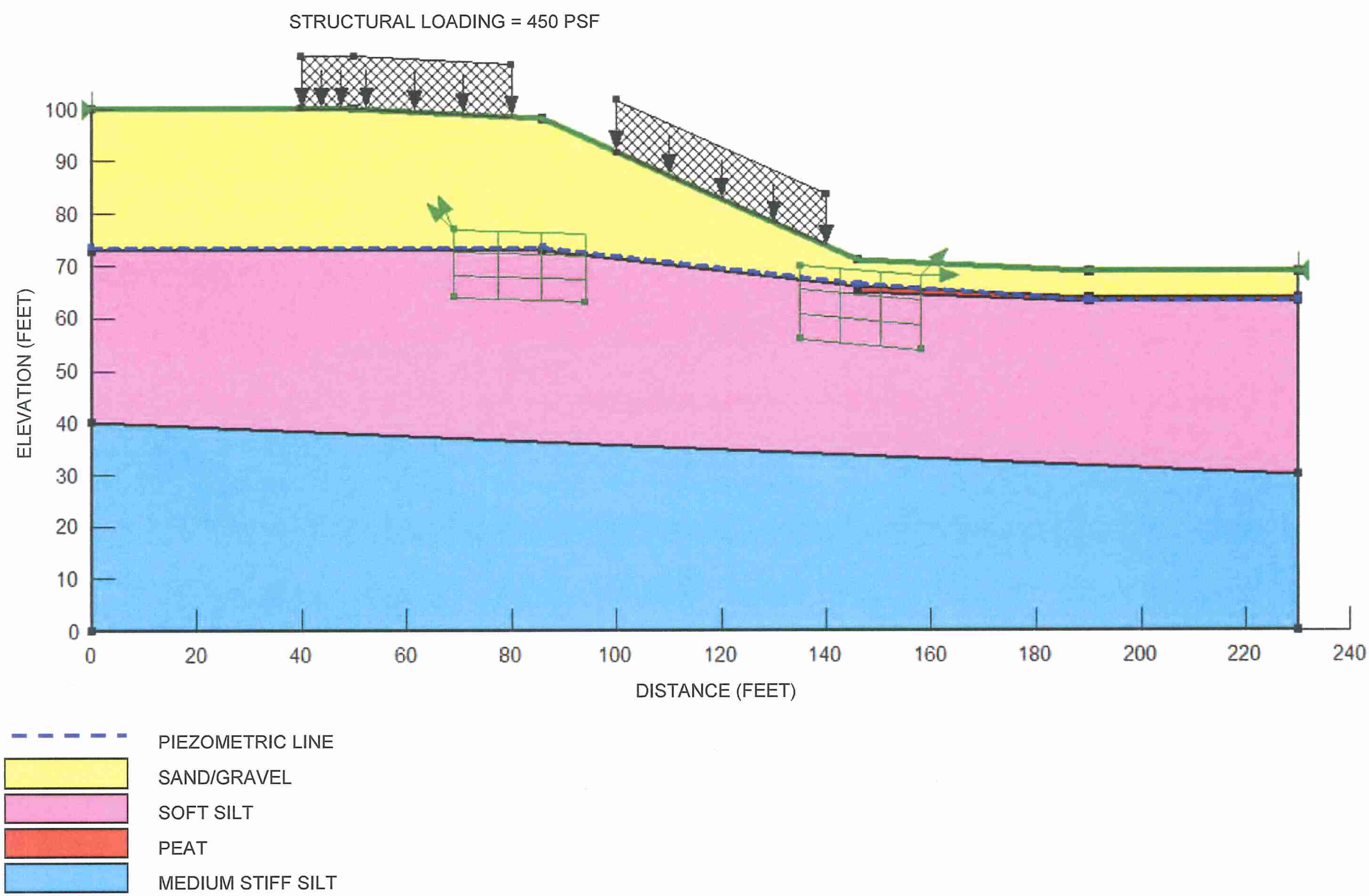
NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: PROSHAKE ANALYSIS RESULTS	
PROJECT NAME: SHIP CREEK DEVELOPMENT	PROJECT ID: 4385-16
PROJECT LOCATION: ANCHORAGE, AK	FIGURE NUMBER: 4



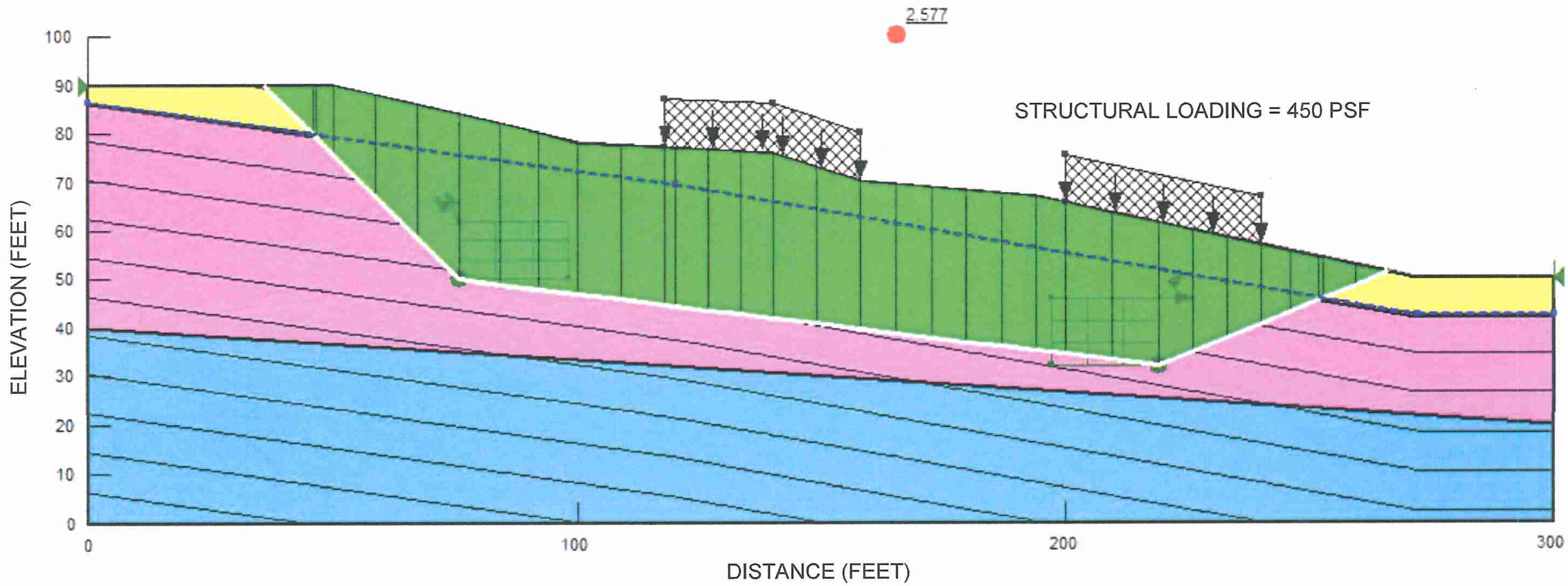
NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: A-A' SLOPE CROSS SECTION PROFILE	
PROJECT NAME: SHIP CREEK DEVELOPMENT	PROJECT ID: 4385-16
PROJECT LOCATION: ANCHORAGE, AK	FIGURE NUMBER: 5



NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: B-B' SLOPE CROSS SECTION PROFILE	
PROJECT NAME: SHIP CREEK DEVELOPMENT	PROJECT ID: 4385-16
PROJECT LOCATION: ANCHORAGE, AK	FIGURE NUMBER: 6



PIEZOMETRIC LINE

SAND/GRAVEL: UNIT WEIGHT=125 PCF; COHESION=0 PSF; PHI=32°

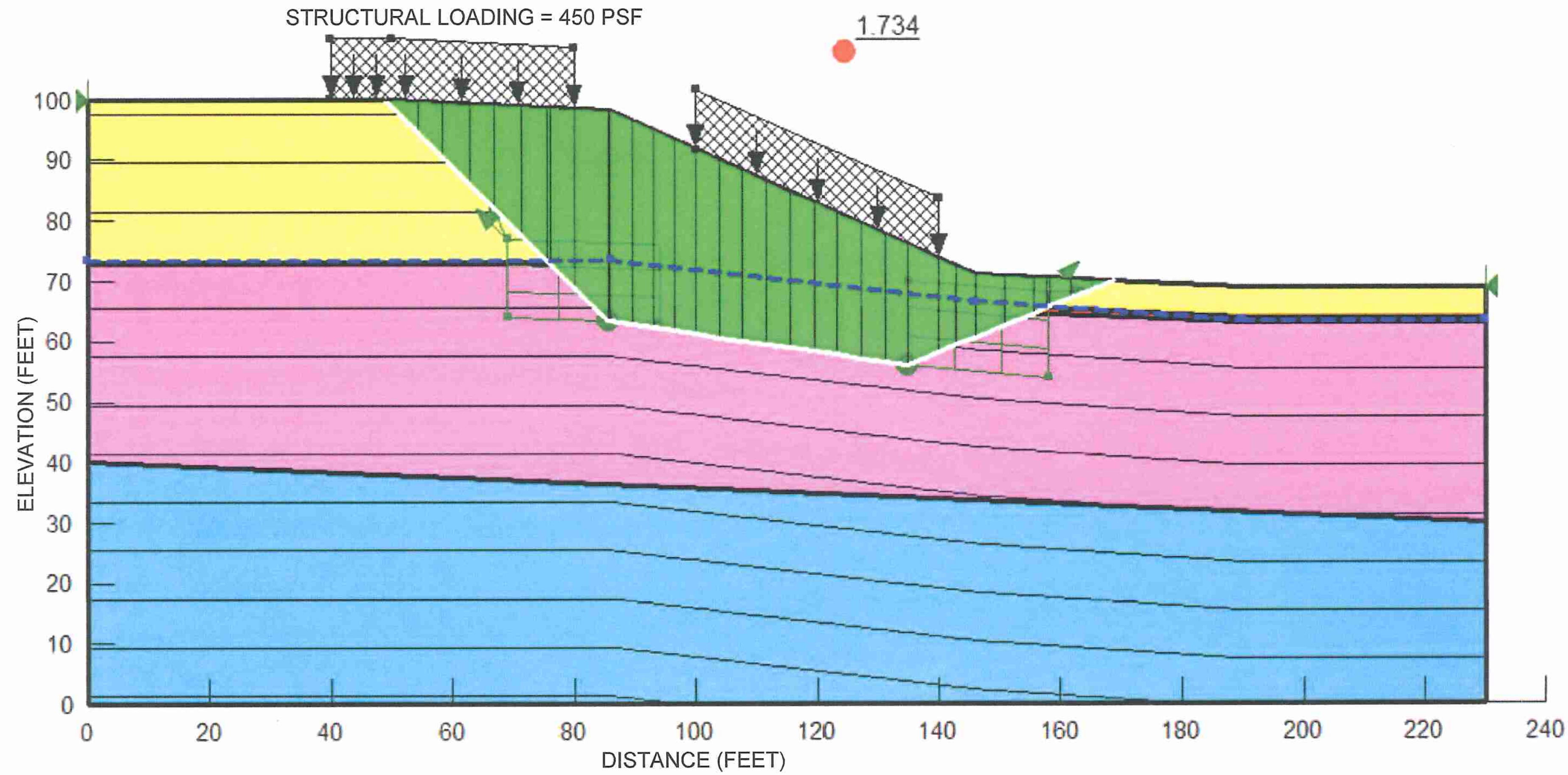
SOFT SILT: UNIT WEIGHT=110 PCF; COHESION=800 PSF; PHI=8°

MEDIUM STIFF SILT: UNIT WEIGHT=110 PCF; COHESION=1000 PSF; PHI=0°

* SILT FRICTION ANGLE TENDS TO BE RELATIVELY LOW AT A LOW STRAIN RATE.

NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: A-A' STATIC SLOPE STABILITY ANALYSIS	
PROJECT NAME: SHIP CREEK DEVELOPMENT	PROJECT ID: 4385-16
PROJECT LOCATION: ANCHORAGE, AK	FIGURE NUMBER: 7



PIEZOMETRIC LINE

SAND/GRAVEL: UNIT WEIGHT=125 PCF; COHESION=0 PSF; PHI=32˚

SOFT SILT: UNIT WEIGHT=110 PCF; COHESION=800 PSF; PHI=8˚

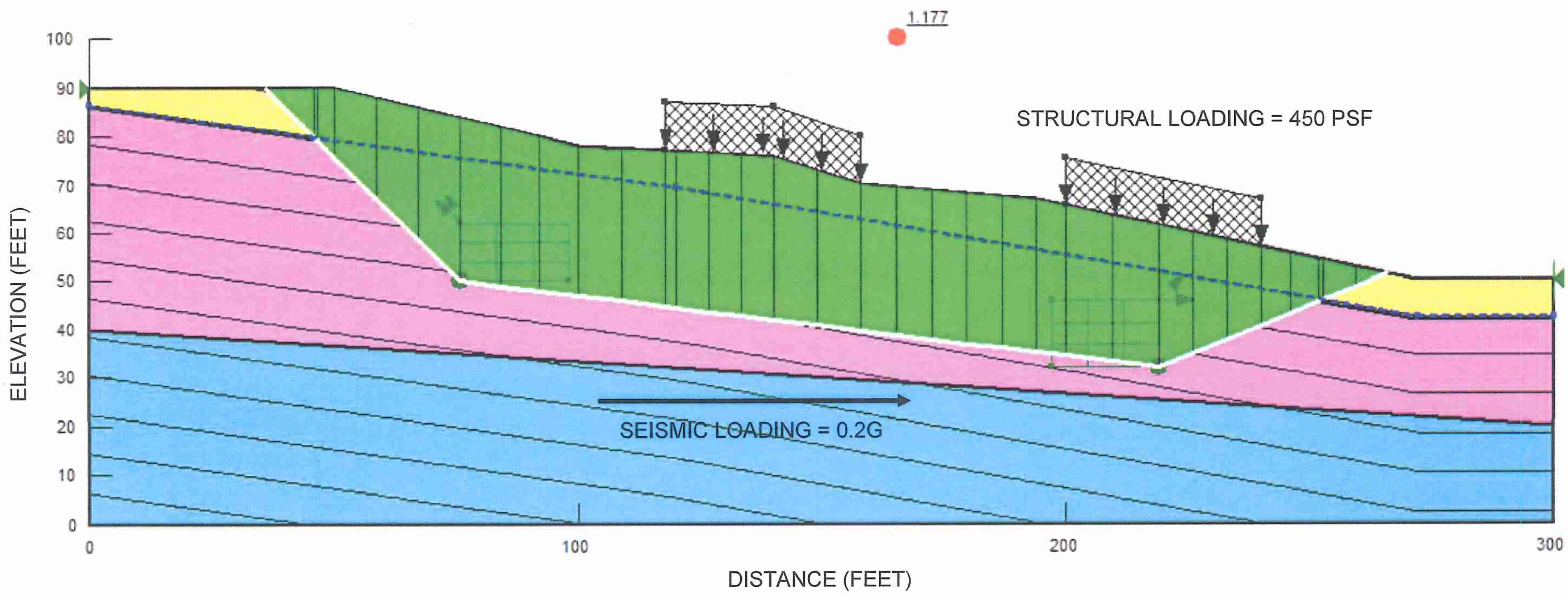
PEAT: UNIT WEIGHT=90 PCF; COHESION=0 PSF; PHI=28˚

MEDIUM STIFF SILT: UNIT WEIGHT=110 PCF; COHESION=1000 PSF; PHI=0˚

* SILT COHESION TENDS TO BE RELATIVELY LOW AT A LOW STRAIN RATE.

NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: B-B' STATIC SLOPE STABILITY ANALYSIS	
PROJECT NAME: SHIP CREEK DEVELOPMENT	PROJECT ID: 4385-16
PROJECT LOCATION: ANCHORAGE, AK	FIGURE NUMBER: 8



PIEZOMETRIC LINE

SAND/GRAVEL: UNIT WEIGHT=125 PCF; COHESION=0 PSF; PHI=32°

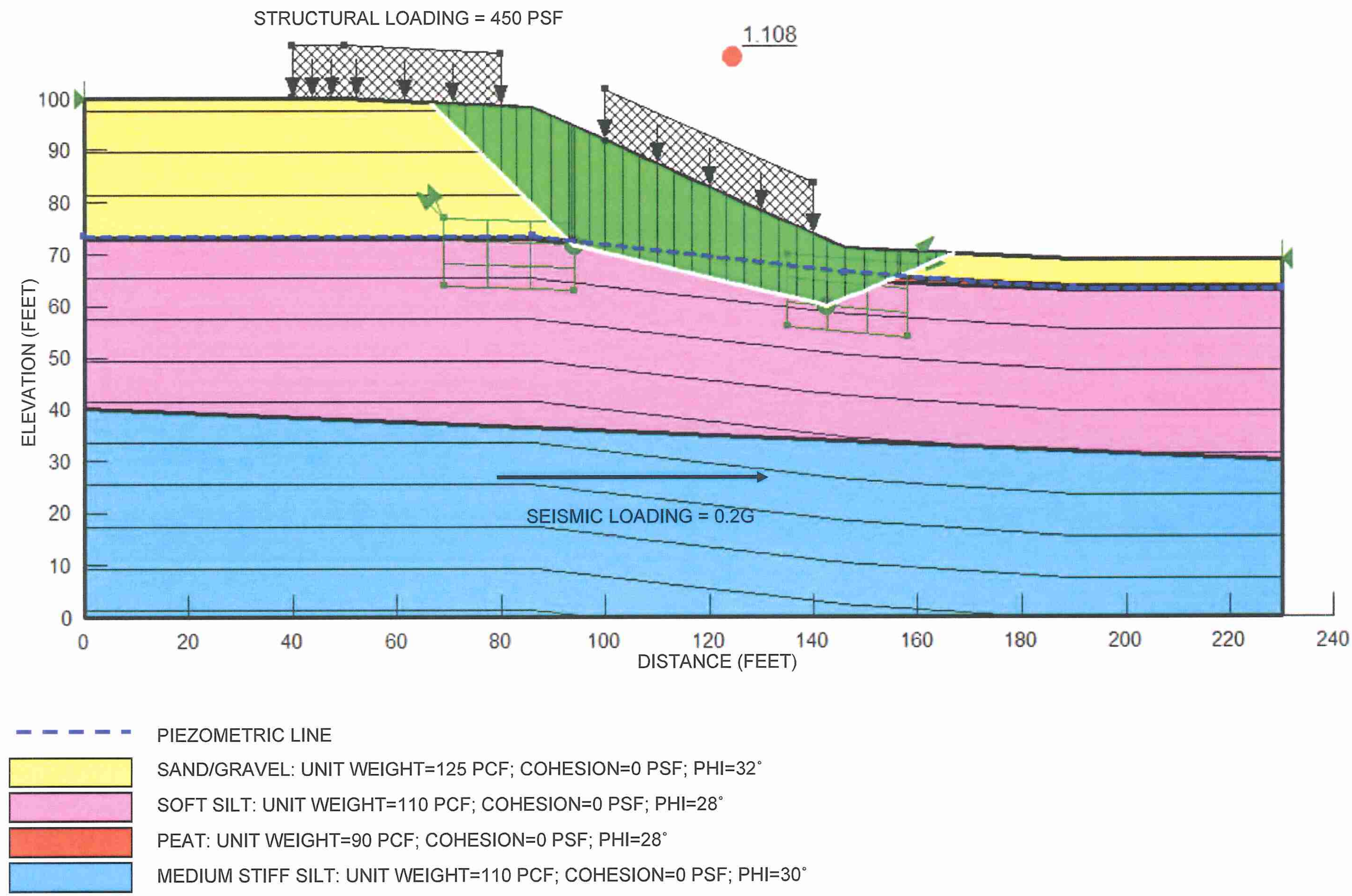
SOFT SILT: UNIT WEIGHT=110 PCF; COHESION=0 PSF; PHI=28°

MEDIUM STIFF SILT: UNIT WEIGHT=110 PCF; COHESION=0 PSF; PHI=30°

* SILT COHESION TENDS TO BE RELATIVELY LOW AT A HIGH STRAIN RATE.

NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

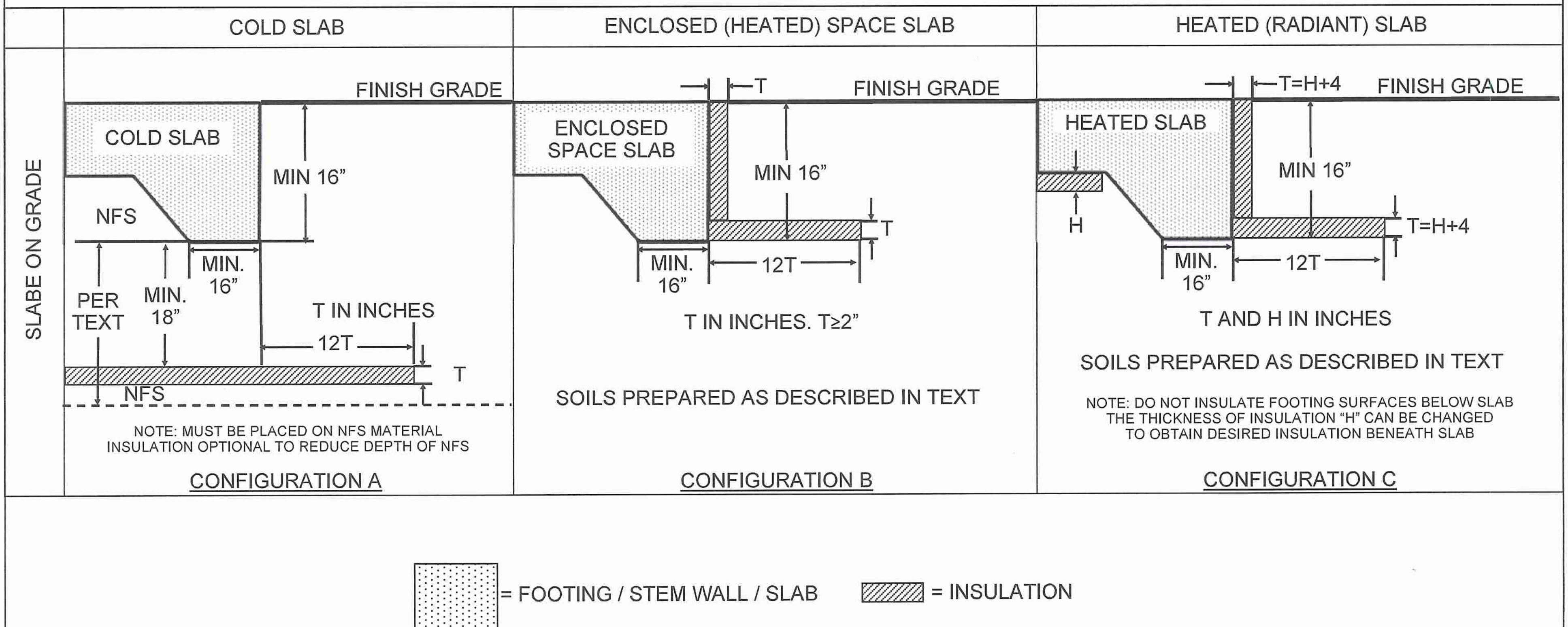
FIGURE TITLE: A-A' PSEUDO-STATIC SLOPE STABILITY ANALYSIS			
PROJECT NAME: SHIP CREEK DEVELOPMENT		PROJECT ID: 4385-16	
PROJECT LOCATION: ANCHORAGE, AK		FIGURE NUMBER: 9	



* SILT COHESION TENDS TO BE RELATIVELY LOW AT A HIGH STRAIN RATE.

NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: B-B' PSEUDO-STATIC SLOPE STABILITY ANALYSIS	
PROJECT NAME: SHIP CREEK DEVELOPMENT	PROJECT ID: 4385-16
PROJECT LOCATION: ANCHORAGE, AK	FIGURE NUMBER: 10

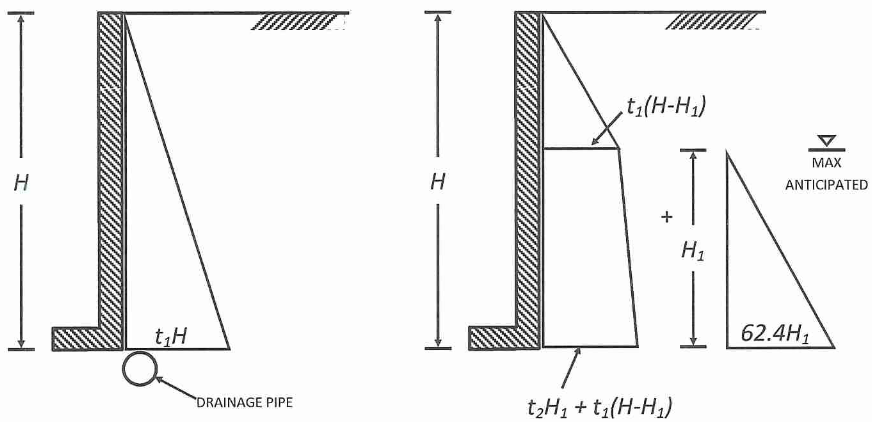


CONFIGURATIONS NOT TO SCALE

NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: FOUNDATION INSULATION CONFIGURATIONS	
PROJECT NAME: SHIP CREEK DEVELOPMENT	PROJECT ID: 4385-16
PROJECT LOCATION: ANCHORAGE, AK	FIGURE NUMBER: 11

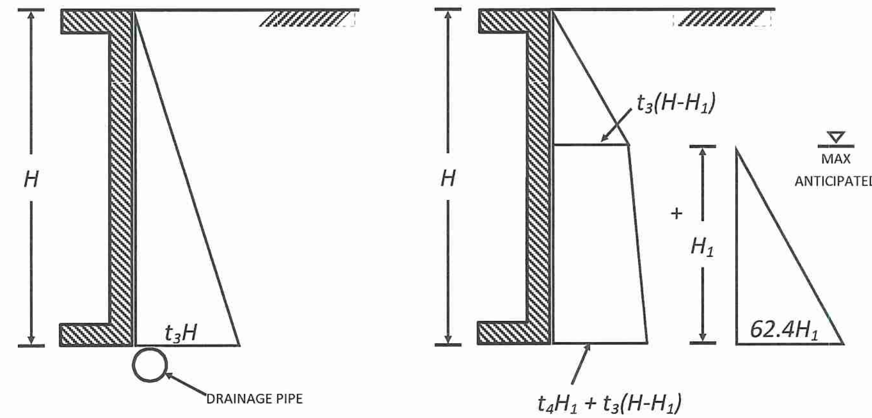
ACTIVE PRESSURE CONDITION



WITH DRAINAGE

WITHOUT DRAINAGE

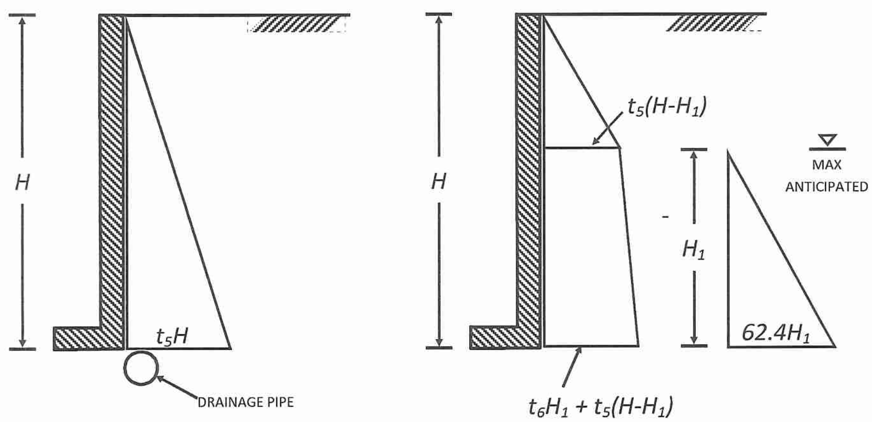
AT-REST PRESSURE CONDITION



WITH DRAINAGE

WITHOUT DRAINAGE

PASSIVE PRESSURE CONDITION

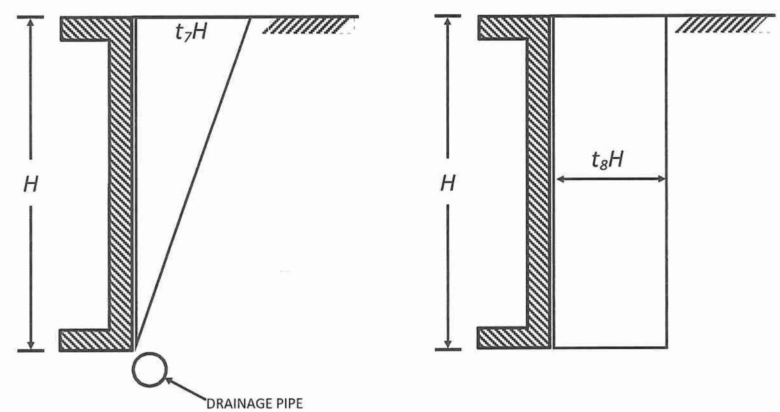


WITH DRAINAGE

WITHOUT DRAINAGE

NOTE: WALLS CAN BE EITHER FREE OR RESTRAINED AT THE TOP FOR THE PASSIVE PRESSURE CONDITION. EQUATIONS ARE VALID FOR UNITS OF $t_{1\text{-}8}$ (PCF) AND H-H_1(FT).

SEISMIC



ACTIVE

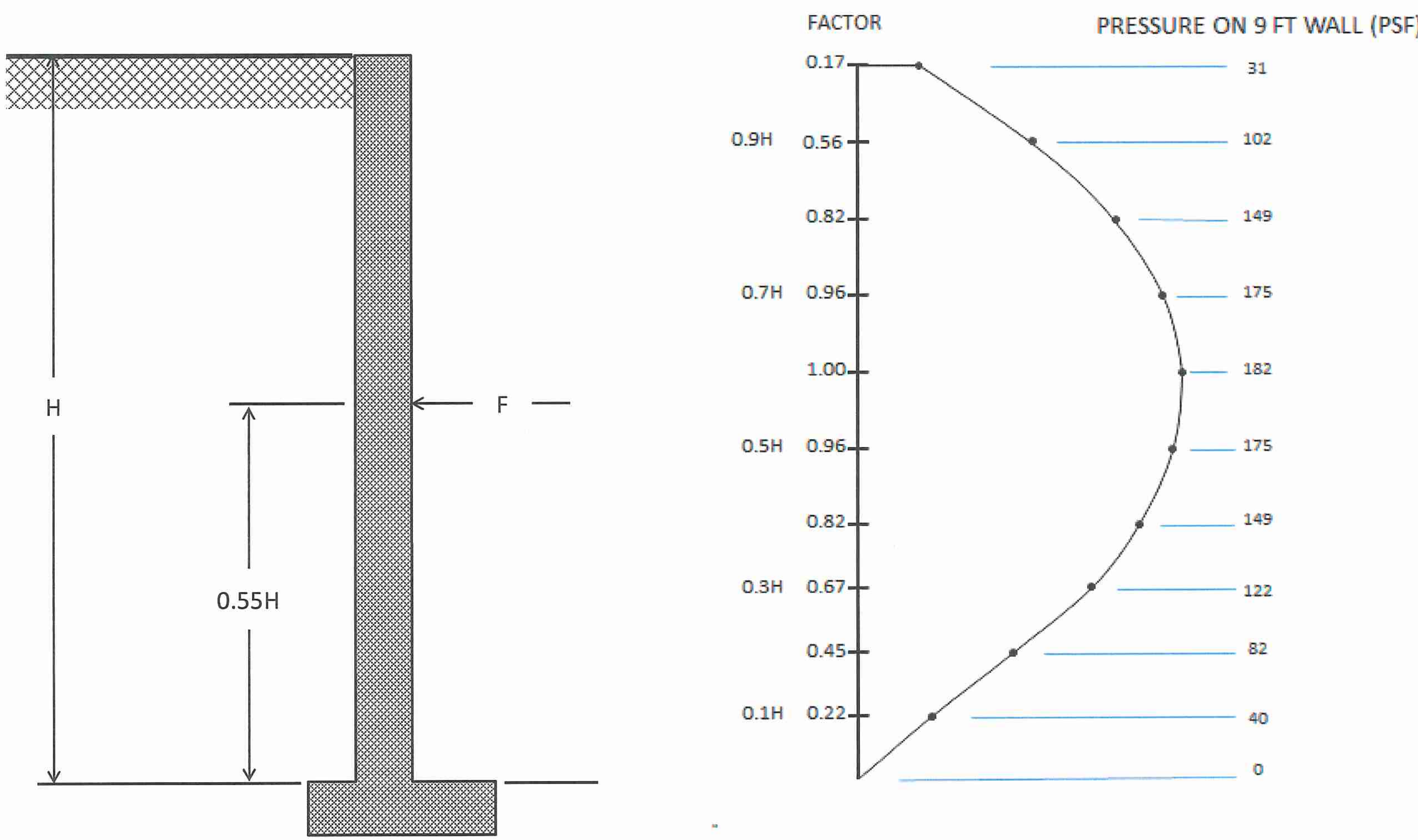
AT-REST

NOTE: SEISMIC LOADS ARE VALID FOR WALLS RETAINING LESS THAN 8 FEET VERTICAL OF EARTH. THE SEISMIC LOAD IS ADDED TO ACTIVE & AT-REST CONDITIONS AND IS SUBTRACTED FROM PASSIVE CONDITIONS.

NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: LATERAL RETAINING WALL PRESSURES	
PROJECT NAME: SHIP CREEK DEVELOPMENT	PROJECT ID: 4385-16
PROJECT LOCATION: ANCHORAGE, AK	FIGURE NUMBER: 12

RESTRAINED WALL ONLY



RESULTANT F: 9' WALL 1038 PLF (9 FT SOIL RETAINED)

NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: RESTRAINED WALL SEISMIC PRESSURE DISTRIBUTION	
PROJECT NAME: SHIP CREEK DEVELOPMENT	PROJECT ID: 4389-16
PROJECT LOCATION: ANCHORAGE, AK	FIGURE NUMBER: 13

LEVELING COURSE

SIEVE SIZE	% BY MASS PASSING
1"	100
3/4"	70-100
3/8"	50-80
#4	35-65
#8	20-50
#50	8-28
#200	2-6
0.02	0-3

TYPE II

SIEVE SIZE	% BY MASS PASSING
8"	100
3"	70-100
1-1/2"	55-100
3/4"	45-85
#4	20-60
#10	12-50
#40	4-30
#200	*2-6
0.02	0-3

*IN ADDITION TO THE GRADING LIMITS LISTED ABOVE, THE FRACTION OF MATERIAL PASSING THE #200 SIEVE SHALL NOT BE GREATER THAN FIFTEEN PERCENT (15 %) OF THAT FRACTION PASSING THE #4 SIEVE.

TYPE II - A

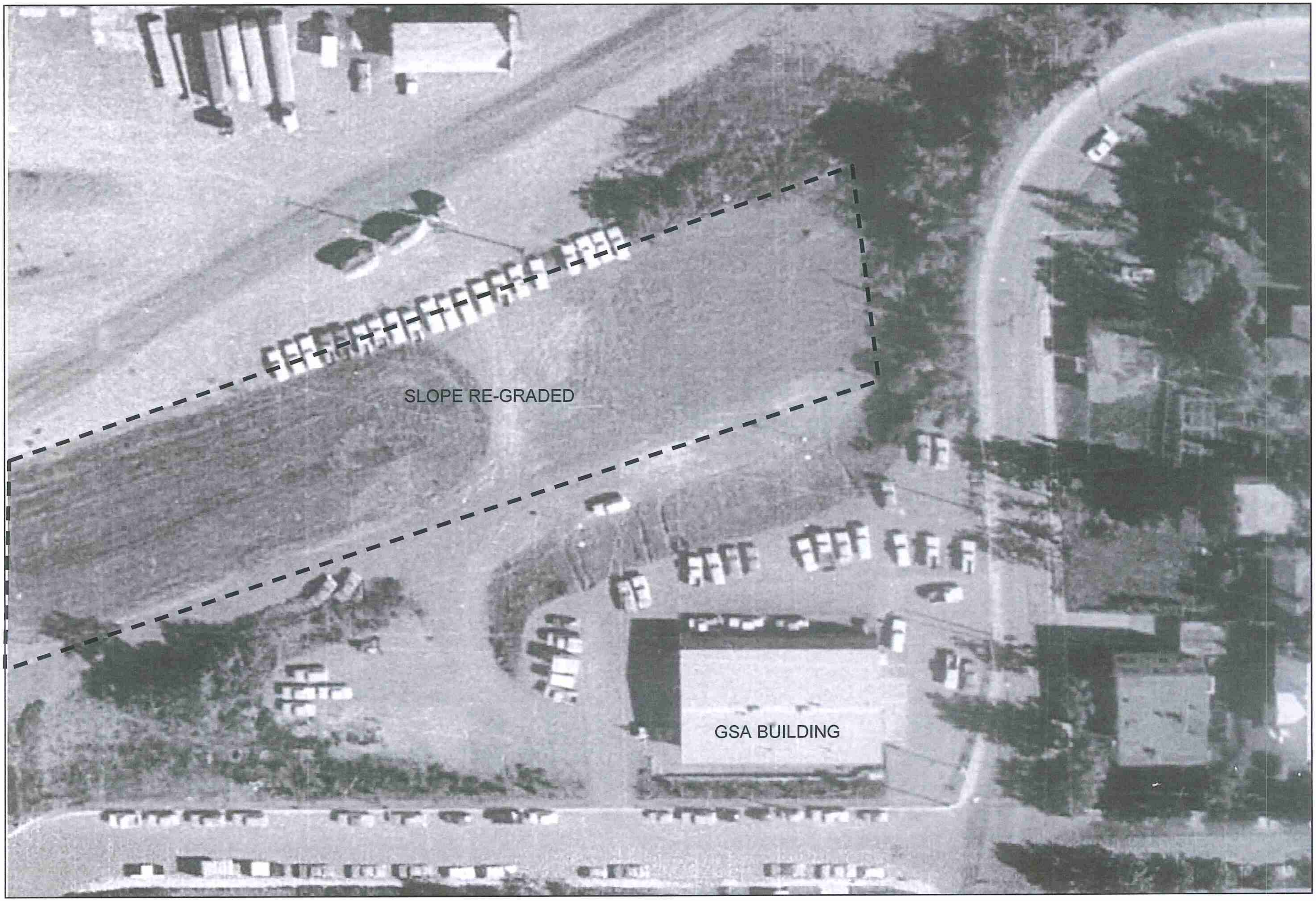
SIEVE SIZE	% BY MASS PASSING
3"	100
3/4"	50-100
#4	25-60
#10	15-50
#40	4-30
#200	*2-6
0.02	0-3

*IN ADDITION TO THE GRADING LIMITS LISTED ABOVE, THE FRACTION OF MATERIAL PASSING THE #200 SIEVE SHALL NOT BE GREATER THAN TWENTY PERCENT (20 %) OF THAT FRACTION PASSING THE #4 SIEVE.

APPENDIX A

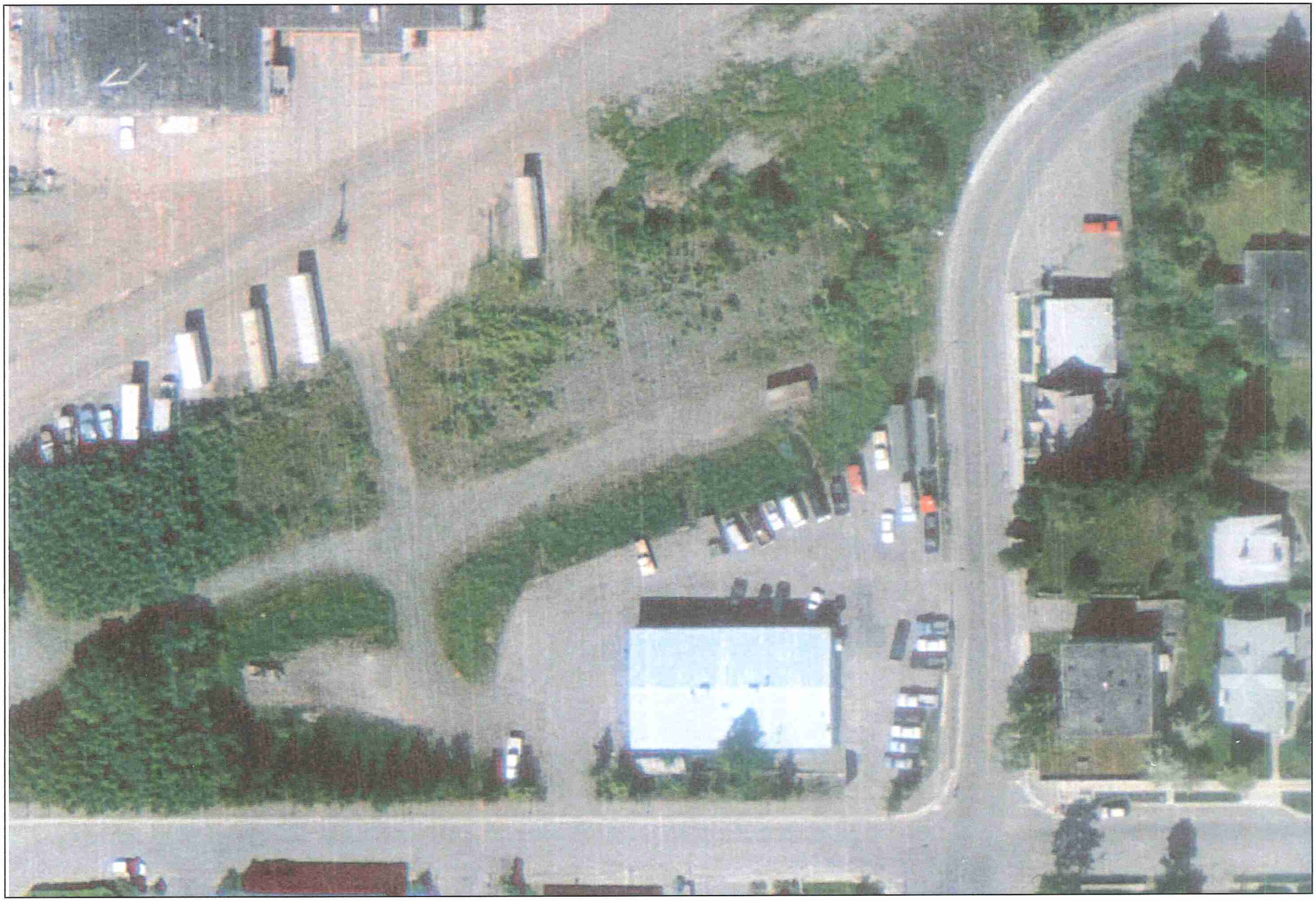
AERIAL PHOTOGRAPHS OF PROJECT SITE

NORTHERN GEOTECHNICAL ENGINEERING, INC. TERRA FIRMA TESTING	APPENDIX TITLE: AERIAL PHOTOGRAPHS OF PROJECT SITE— MAY 5, 1960	
	PROJECT NAME: SHIP CREEK DEVELOPMENT	PROJECT ID: 4385-16
	PROJECT LOCATION: ANCHORAGE, AK	APPENDIX NUMBER: A-1



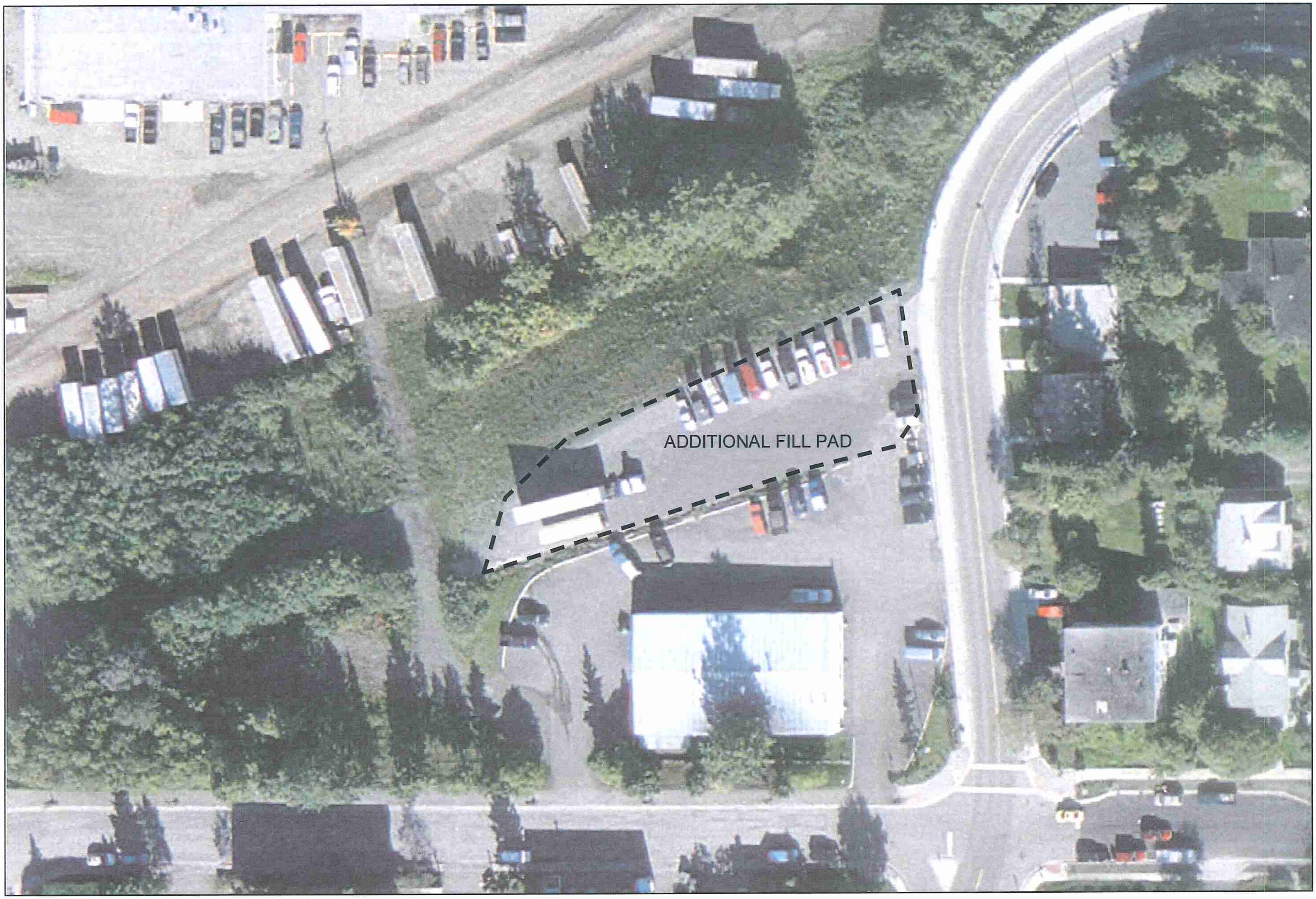
NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

APPENDIX TITLE: AERIAL PHOTOGRAPHS OF PROJECT SITE— JULY 7, 1970	
PROJECT NAME: SHIP CREEK DEVELOPMENT	PROJECT ID: 4385-16
PROJECT LOCATION: ANCHORAGE, AK	APPENDIX NUMBER: A-2



Northern Geotechnical Engineering, Inc.
Terra Firma Testing

APPENDIX TITLE: AERIAL PHOTOGRAPHS OF PROJECT SITE— JUNE 3, 1990	
PROJECT NAME: SHIP CREEK DEVELOPMENT	PROJECT ID: 4385-16
PROJECT LOCATION: ANCHORAGE, AK	APPENDIX NUMBER: A-3



NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

APPENDIX TITLE: AERIAL PHOTOGRAPHS OF PROJECT SITE— SEPTEMBER 10, 2002	
PROJECT NAME: SHIP CREEK DEVELOPMENT	PROJECT ID: 4385-16
PROJECT LOCATION: ANCHORAGE, AK	APPENDIX NUMBER: A-4

NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

APPENDIX TITLE: AERIAL PHOTOGRAPHS OF PROJECT SITE— MAY 1, 2015	
PROJECT NAME: SHIP CREEK DEVELOPMENT	PROJECT ID: 4385-16
PROJECT LOCATION: ANCHORAGE, AK	APPENDIX NUMBER: A-5

APPENDIX B

GEOTECHNICAL BOREHOLE LOGS

Northern Geotechnical Engineering Inc.
d.b.a. Terra Firma Testing
11301 Olive Lane
Anchorage, AK 99515
Telephone: 907-344-5934
Fax: 907-344-5993

EXPLORATION B-1

NGE-TFT PROJECT NAME: Ship Creek Condo Development
NGE-TFT PROJECT NUMBER: 4385-16
PROJECT LOCATION: Anchorage, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: CME-75 w/ 340lb autohammer
EXPLORATION METHOD: Hollow Stem Auger w/ center drill rods
SAMPLING METHOD: Modified Split-spoon/Thin-walled Shelby Tube
LOGGED BY: E. Boatwright
DATE STARTED: 5/6/2016
DATE COMPLETED: 5/6/2016
EXPLORATION LOCATION: See report figure 2
GROUND ELEVATION: Approx. 50 ft above mean sea level
▽ **GROUNDWATER (ATD):** N/E
▼ **GROUNDWATER (AD):** N/A
EXPLORATION COMPLETION: Backfilled with cuttings.
WEATHER CONDITIONS: Overcast

DEPTH (ft)	GRAPHIC LOG	FROZEN SOILS	MATERIAL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	SHELBY DOWN PRESSURE (psi)	LAB RESULTS	REMARKS/NOTES
0			**GRAVEL** (GP-GM), loose to dense, gray to dark brown, dry to damp, FILL		S1	N/A	N/A	N/A	N/A	MC = 1.0%	
					S2	12	3 7 11	30	N/A	MC = 4.7% 44.2% gravel, 44.0% sand, 11.8% silt P0.02 = 8.4% FC = F1	
5					S3	14	3 3 4	9	N/A	MC = 7.3%	Increased sand content at approx. 5' bgs.
					S4	12	3 5 4	10	N/A	MC = 7.2% 31.0% gravel, 56.3% sand, 12.7% silt	
10					S5	12	4 6 3	8	N/A	MC = 8.2%	
15					S6a S6b	16	2 7 12	16	N/A	MC = 3.7% (S6a) MC = 11.1% (S6b) 26.4% gravel, 47.7% sand, 25.9% silt P0.02 = 20.1% FC = F3	Increased silt content at approx. 15' bgs.
20											

(Continued Next Page)

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EXPLORATION METHOD: Hollow Stem Auger w/ center drill rods
SAMPLING METHOD: Modified Split-spoon/Thin-walled Shelby Tube
LOGGED BY: E. Boatwright
DATE STARTED: 5/6/2016
DATE COMPLETED: 5/6/2016
EXPLORATION LOCATION: See report figure 2
GROUND ELEVATION: Approx. 50 ft above mean sea level
GROUNDWATER (ATD): N/E
GROUNDWATER (AD): N/A
EXPLORATION COMPLETION: Backfilled with cuttings.
WEATHER CONDITIONS: Overcast

DEPTH (ft)	GRAPHIC LOG	FROZEN SOILS	MATERIAL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	SHELBY DOWN PRESSURE (psi)	LAB RESULTS	REMARKS/NOTES
20			**GRAVEL** (GP-GM), loose to dense, gray to dark brown, dry to damp, FILL *(continued)*		S7	14	3 18 19	31	N/A	MC = 3.5%	Decreased sand and silt content at approx. 20' bgs.
25			**SILT** (ML), with sand, soft to medium stiff, brownish gray to blueish gray, lensed, moist to wet, low-medium plasticity, occasional sand lenses		S8	14	2 2 3	5	N/A	MC = 14.0% P200 = 67.8%	
30					S9	16	2 3 3	6	N/A	MC = 20.4% LL = 28 PL = 24 PI = 4	
35					S10	18	2 1 1	2	N/A	MC = 20.8%	
40											

(Continued Next Page)

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LOGGED BY: E. Boatwright
DATE STARTED: 5/6/2016
DATE COMPLETED: 5/6/2016
EXPLORATION LOCATION: See report figure 2
GROUND ELEVATION: Approx. 50 ft above mean sea level
▽ GROUNDWATER (ATD): N/E
▼ GROUNDWATER (AD): N/A
EXPLORATION COMPLETION: Backfilled with cuttings.
WEATHER CONDITIONS: Overcast

DEPTH (ft)	GRAPHIC LOG	FROZEN SOILS	MATERIAL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	SHELBY DOWN PRESSURE (psi)	LAB RESULTS	REMARKS/NOTES
40			**SILT** (ML), with sand, soft to medium stiff, brownish gray to blueish gray, lensed, moist to wet, low-medium plasticity, occasional sand lenses *(continued)*		S11	18	1 1 1	2	N/A	MC = 37.1% P200 = 45.2%	
45					S12	24 N/A N/A	N/A	N/A	400 450 450 500		PP @ 45.83' bgs = 0.5 tsf TV @ 45.83' bgs = 0.4 tsf. PP @ 46.66' bgs = 0.25 tsf TV @ 46.66' bgs = 0.45 tsf.
50					S13	24 N/A N/A	N/A	N/A	400 450 500 500	MC = 29.2% 0.0% gravel, 1.3% sand, 98.7% silt P0.02 = 88.3% LL = 38 PL = 23 PI = 13 FC = F4 P0.002 = 55.4% SG = 2.694	PP @ 51.66' bgs = 0.5 tsf TV @ 51.66' bgs = 0.45 tsf. PP @ 51.91' bgs = 0.125 tsf TV @ 51.91' bgs = 0.3 tsf.

Bottom of borehole at 52.0 feet bgs.

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Fax: 907-344-5993

PHOTO APPENDIX

CLIENT John McGrew

PROJECT NAME Ship Creek Condo Development

PROJECT NUMBER 4385-16

PROJECT LOCATION Anchorage, AK

Exploration B1 Sample S2
Sample Interval 2.5'-4' bgs

Exploration B1 Sample S3
Sample Interval 5'-6.5' bgs

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PHOTO APPENDIX

CLIENT John McGrew

PROJECT NAME Ship Creek Condo Development

PROJECT NUMBER 4385-16

PROJECT LOCATION Anchorage, AK

Exploration B1 Sample S5
Sample Interval 10'-11.5' bgs

Exploration B1 Sample S6
Sample Interval 15'-16.5' bgs

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PHOTO APPENDIX

CLIENT John McGrew

PROJECT NAME Ship Creek Condo Development

PROJECT NUMBER 4385-16

PROJECT LOCATION Anchorage, AK

Exploration B1 Sample S7
Sample Interval 20'-21.5' bgs

Exploration B1 Sample S8
Sample Interval 25'-26.5' bgs

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PHOTO APPENDIX

CLIENT John McGrew | **PROJECT NAME** Ship Creek Condo Development

PROJECT NUMBER 4385-16 | **PROJECT LOCATION** Anchorage, AK

Exploration B1 Sample S9
Sample Interval 30'-31.5' bgs

Exploration B1 Sample S11
Sample Interval 40'-41.5' bgs

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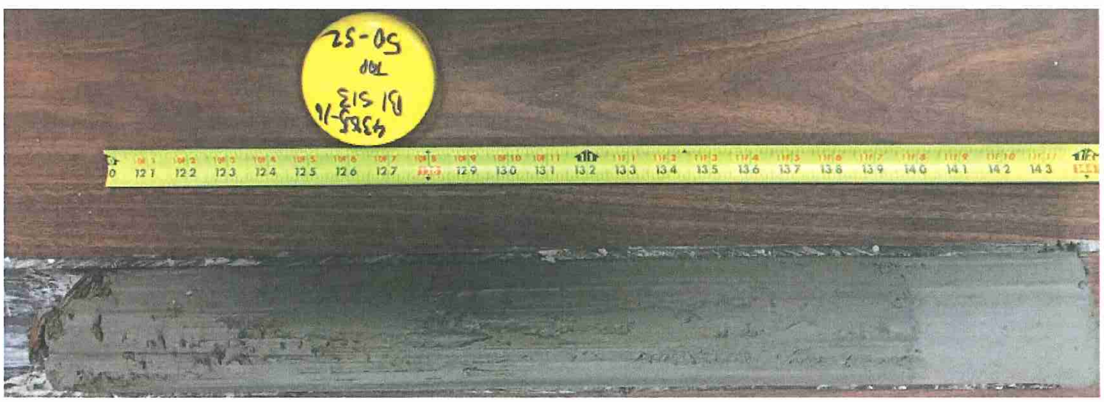
PHOTO APPENDIX

CLIENT John McGrew **PROJECT NAME** Ship Creek Condo Development

PROJECT NUMBER 4385-16 **PROJECT LOCATION** Anchorage, AK

Exploration B1 Sample S12
Sample Interval 45'-47' bgs
Extracted Shelby Tube



Exploration B1 Sample S13
Sample Interval 50'-52' bgs
Extracted Shelby Tube

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EXPLORATION B-2

NGE-TFT PROJECT NAME: Ship Creek Condo Development
NGE-TFT PROJECT NUMBER: 4385-16
PROJECT LOCATION: Anchorage, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: CME-75 w/ 340lb autohammer
EXPLORATION METHOD: Hollow Stem Auger w/ center drill rods
SAMPLING METHOD: Modified Split-spoon/Thin-walled Shelby Tube
LOGGED BY: E. Boatwright
DATE STARTED: 5/6/2016
DATE COMPLETED: 5/6/2016
EXPLORATION LOCATION: See report figure 2
GROUND ELEVATION: Approx. 45 ft above mean sea level
▽ GROUNDWATER (ATD): Approx. 6.5 ft bgs
▼ GROUNDWATER (AD): N/A
EXPLORATION COMPLETION: Backfilled with cuttings.
WEATHER CONDITIONS: Overcast

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE NUMBER	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	SHELBY DOWN PRESSURE (psi)	LAB RESULTS	REMARKS/NOTES
0	**GRAVEL WITH SAND** (GP-GM), some organics, loose, dark brown to brown, damp to moist, FILL	S1	N/A	N/A	N/A	N/A	MC = 3.1%	
		S2	12	3 3 3	10	N/A	MC = 16.0% 36.9% gravel, 40.3% sand, 22.8% silt OC = 3.0%	
5	**SILT WITH SAND** (SM), loose, dark brown, moist	S3	14	2 3 2	6	N/A	MC = 13.6% 14.7% gravel, 56.2% sand, 29.1% silt P0.02 = 16.4% FC = F3	Increased silt content at approx. 4.7' bgs. Increased sand and silt content at approx. 5.3' bgs.
	▽ **SILT TO LEAN CLAY** (ML/CL), with sand, soft, gray, lensed, moist to wet, low plasticity, occasional sand lenses	S4	16	1 2 2	3	N/A	MC = 33.2% LL = 28 PL = 27 PI = 1	
10		S5b S5a	16	1 3 5	7	N/A	MC = 17.5% (S5b)	
15		S6	18	1 1 1	2	N/A	MC = 28.2% P200 = 97.4%	
20								

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EXPLORATION B-2

NGE-TFT PROJECT NAME: Ship Creek Condo Development
NGE-TFT PROJECT NUMBER: 4385-16
PROJECT LOCATION: Anchorage, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: CME-75 w/ 340lb autohammer
EXPLORATION METHOD: Hollow Stem Auger w/ center drill rods
SAMPLING METHOD: Modified Split-spoon/Thin-walled Shelby Tube
LOGGED BY: E. Boatwright
DATE STARTED: 5/6/2016
DATE COMPLETED: 5/6/2016
EXPLORATION LOCATION: See report figure 2
GROUND ELEVATION: Approx. 45 ft above mean sea level
▽ GROUNDWATER (ATD): Approx. 6.5 ft bgs
▼ GROUNDWATER (AD): N/A
EXPLORATION COMPLETION: Backfilled with cuttings.
WEATHER CONDITIONS: Overcast

DEPTH (ft)	GRAPHIC LOG	FROZEN SOILS	MATERIAL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	SHELBY DOWN PRESSURE (psi)	LAB RESULTS	REMARKS/NOTES
20			**SILT TO LEAN CLAY** (ML/CL), with sand, soft, gray, lensed, moist to wet, low plasticity, occasional sand lenses *(continued)*		S7	18	1 1 1	2	N/A	MC = 31.6%	
25					S8	18	1 1 1	2	N/A	MC = 36.6%	
30					S9	18	1 2 2	4	N/A	MC = 29.2%	
35					S10	18	1 2 2	4	N/A	MC = 30.2%	
40											

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EXPLORATION B-2

NGE-TFT PROJECT NAME: Ship Creek Condo Development
NGE-TFT PROJECT NUMBER: 4385-16
PROJECT LOCATION: Anchorage, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: CME-75 w/ 340lb autohammer
EXPLORATION METHOD: Hollow Stem Auger w/ center drill rods
SAMPLING METHOD: Modified Split-spoon/Thin-walled Shelby Tube
LOGGED BY: E. Boatwright
DATE STARTED: 5/6/2016
DATE COMPLETED: 5/6/2016
EXPLORATION LOCATION: See report figure 2
GROUND ELEVATION: Approx. 45 ft above mean sea level
▽ GROUNDWATER (ATD): Approx. 6.5 ft bgs
▼ GROUNDWATER (AD): N/A
EXPLORATION COMPLETION: Backfilled with cuttings.
WEATHER CONDITIONS: Overcast

DEPTH (ft)	GRAPHIC LOG	FROZEN SOILS	MATERIAL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	SHELBY DOWN PRESSURE (psi)	LAB RESULTS	REMARKS/NOTES
40			**SILT TO LEAN CLAY** (ML/CL), with sand, soft, gray, lensed, moist to wet, low plasticity, occasional sand lenses *(continued)*		S11	18	1 2 2	4	N/A	MC = 26.1% P200 = 99.6%	
45					S12	24	N/A	N/A	250 350 400 450	MC = 28.9% 0.0% gravel, 0.1% sand, 99.9% silt P0.02 = 95.0% LL = 34 PL = 28 PI = 6 FC = F4 P0.002 = 53.1% SG = 2.681	PP @ 46.25' bgs = 0.25 tsf TV @ 46.25' bgs = 0.325 tsf. PP @ 46.75' bgs = 0.25 tsf TV @ 46.75' bgs = 0.35 tsf.
50					S13	24	N/A	N/A	250 350 400 400	MC = 28.5% 0.0% gravel, 0.2% sand, 99.8% silt P0.02 = 84.7% LL = 29 PL = 21 PI = 8 FC = F4 P0.002 = 36.4% SG = 2.633	Very thin sand layer at approx. 50.33' bgs. Very thin sand layer at approx. 50.58' bgs. PP @ 51.58' bgs = 0.25 tsf. PP @ 51.83' bgs = 0.75 tsf TV @ 51.83' bgs = 0.3 tsf.
			Bottom of borehole at 52.0 feet bgs.								

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CLIENT John McGrew

PROJECT NAME Ship Creek Condo Development

PROJECT NUMBER 4385-16

PROJECT LOCATION Anchorage, AK

Exporation B-2 Sample S2
Sample Interval 2.5'-4' bgs

Exporation B-2 Sample S3
Sample Interval 5'-6.5' bgs

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PHOTO APPENDIX

CLIENT John McGrew
PROJECT NAME Ship Creek Condo Development
PROJECT NUMBER 4385-16
PROJECT LOCATION Anchorage, AK

Exporation B-2 Sample S4
Sample Interval 7.5'-9' bgs

Exporation B-2 Sample S5
Sample Interval 10'-11.5' bgs

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PHOTO APPENDIX

CLIENT John McGrew **PROJECT NAME** Ship Creek Condo Development

PROJECT NUMBER 4385-16 **PROJECT LOCATION** Anchorage, AK

Exporation B-2 Sample S6
Sample Interval 15'-16.5' bgs

Exporation B-2 Sample S7
Sample Interval 20'-21.5' bgs

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PHOTO APPENDIX

CLIENT John McGrew

PROJECT NAME Ship Creek Condo Development

PROJECT NUMBER 4385-16

PROJECT LOCATION Anchorage, AK

Exporation B-2 Sample S8
Sample Interval 25'-26.5' bgs

Exporation B-2 Sample S9
Sample Interval 30'-31.5' bgs

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CLIENT John McGrew

PROJECT NAME Ship Creek Condo Development

PROJECT NUMBER 4385-16

PROJECT LOCATION Anchorage, AK

Exporation B-2 Sample S10
Sample Interval 35'-36.5' bgs

Exploration B2 Sample S12
Sample Interval 45'-47' bgs
Extracted Shelby Tube

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PHOTO APPENDIX

CLIENT John McGrew

PROJECT NAME Ship Creek Condo Development

PROJECT NUMBER 4385-16

PROJECT LOCATION Anchorage, AK

Exploration B2 Sample S13
Sample Interval 50'-52' bgs
Extracted Shelby Tube

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EXPLORATION B-3

NGE-TFT PROJECT NAME: Ship Creek Condo Development
NGE-TFT PROJECT NUMBER: 4385-16
PROJECT LOCATION: Anchorage, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: CME-75 w/ 340lb autohammer
EXPLORATION METHOD: Hollow Stem Auger w/ center drill rods
SAMPLING METHOD: Modified Split-spoon/Thin-walled Shelby Tube
LOGGED BY: E. Boatwright
DATE STARTED: 5/5/2016
DATE COMPLETED: 5/5/2016
EXPLORATION LOCATION: See report figure 2
GROUND ELEVATION: Approx. 18 ft above mean sea level
∇ GROUNDWATER (ATD): Approx. 18.0 ft bgs
▼ GROUNDWATER (AD): N/A
EXPLORATION COMPLETION: Backfilled with cuttings.
WEATHER CONDITIONS: Overcast

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE NUMBER	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	SHELBY DOWN PRESSURE (psi)	LAB RESULTS	REMARKS/NOTES
0	**SILTY GRAVEL** (GM), some organics, medium dense, brown, moist, FILL	S1	N/A	N/A	N/A	N/A	MC = 7.0% 37.5% gravel, 39.9% sand, 22.6% silt OC = 3.9%	
	SILT WITH SAND AND GRAVEL (SM), medium stiff, brown, moist to wet	S2	15	1 3 4	12	N/A	MC = 17.1%	
5		S3	14	3 3 4	6	N/A	MC = 27.3% 13.1% gravel, 43.7% sand, 43.2% silt P0.02 = 28.8% FC = F3	
	SILT (ML), trace organics, very soft to soft, brown to blueish gray, moist to wet, low plasticity, occasional sand lenses	S4	10	3 3 4	6	N/A	MC = 22.3% OC = 6.2%	
10		S5	10	2 3 4	6	N/A	MC = 25.0%	
15		S6	9	1 1 2	3	N/A	MC = 30.6% LL = 28 PL = 28 PI = 0	
	∇							
20		S7	15	1 1 1	2	N/A	MC = 36.2%	

(Continued Next Page)

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EXPLORATION B-3

NGE-TFT PROJECT NAME: Ship Creek Condo Development
NGE-TFT PROJECT NUMBER: 4385-16
PROJECT LOCATION: Anchorage, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: CME-75 w/ 340lb autohammer
EXPLORATION METHOD: Hollow Stem Auger w/ center drill rods
SAMPLING METHOD: Modified Split-spoon/Thin-walled Shelby Tube
LOGGED BY: E. Boatwright
DATE STARTED: 5/5/2016
DATE COMPLETED: 5/5/2016
EXPLORATION LOCATION: See report figure 2
GROUND ELEVATION: Approx. 18 ft above mean sea level
▽ GROUNDWATER (ATD): Approx. 18.0 ft bgs
▼ GROUNDWATER (AD): N/A
EXPLORATION COMPLETION: Backfilled with cuttings.
WEATHER CONDITIONS: Overcast

DEPTH (ft)	GRAPHIC LOG	FROZEN SOILS	MATERIAL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	SHELBY DOWN PRESSURE (psi)	LAB RESULTS	REMARKS/NOTES
			SILT (ML), trace organics, very soft to soft, brown to blueish gray, moist to wet, low plasticity, occasional sand lenses *(continued)*								
25					S8	24	N/A	N/A	150 350 300 300		PP @ 25.66' bgs = 0.6 tsf TV @ 25.66' bgs = 0.375 tsf.
30					S9	18	1 1 1	2	N/A	MC = 26.2% P200 = 97.4%	
35					S10	24	N/A	N/A	300 400 450 500	MC = 31.0% 0.0% gravel, 2.4% sand, 97.6% silt P0.02 = 94.0% LL = 37 PL = 29 PI = 8 FC = F4 P0.002 = 52.1% SG = 2.671	PP @ 36' bgs = 0.75 tsf. PP @ 36.5' bgs = 0.5 tsf TV @ 36.5' bgs = 0.4 tsf.

Bottom of borehole at 37.0 feet bgs.

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PHOTO APPENDIX

CLIENT John McGrew

PROJECT NAME Ship Creek Condo Development

PROJECT NUMBER 4385-16

PROJECT LOCATION Anchorage, AK

Exploration B-3 Sample S3
Sample Interval 5'-6.5' bgs

Exploration B-3 Sample S4
Sample Interval 7.5'-9' bgs

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PHOTO APPENDIX

CLIENT John McGrew

PROJECT NAME Ship Creek Condo Development

PROJECT NUMBER 4385-16

PROJECT LOCATION Anchorage, AK

Exploration B-3 Sample S5
Sample Interval 10'-11.5' bgs

Exploration B-3 Sample S6
Sample Interval 15'-16.5' bgs

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PHOTO APPENDIX

CLIENT John McGrew

PROJECT NAME Ship Creek Condo Development

PROJECT NUMBER 4385-16

PROJECT LOCATION Anchorage, AK

Exploration B-3 Sample S7
Sample Interval 20'-21.5' bgs

Exploration B3 Sample S8
Sample Interval 25'-27' bgs
Extracted Shelby Tube

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PHOTO APPENDIX

CLIENT John McGrew

PROJECT NAME Ship Creek Condo Development

PROJECT NUMBER 4385-16

PROJECT LOCATION Anchorage, AK

Exploration B-3 Sample S9
Sample Interval 30'-31.5' bgs

Exploration B3 Sample S10
Sample Interval 35'-37' bgs
Extracted Shelby Tube

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EXPLORATION B-4

NGE-TFT PROJECT NAME: Ship Creek Condo Development
NGE-TFT PROJECT NUMBER: 4385-16
PROJECT LOCATION: Anchorage, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: CME-75 w/ 340lb autohammer
EXPLORATION METHOD: Hollow Stem Auger w/ center drill rods
SAMPLING METHOD: Modified Split-spoon/Thin-walled Shelby Tube
LOGGED BY: E. Boatwright
DATE STARTED: 5/5/2016
DATE COMPLETED: 5/5/2016
EXPLORATION LOCATION: See report figure 2
GROUND ELEVATION: Approx. 19 ft above mean sea level
▽ **GROUNDWATER (ATD):** Approx. 6.5 ft bgs
▼ **GROUNDWATER (AD):** N/A
EXPLORATION COMPLETION: Backfilled with cuttings.
WEATHER CONDITIONS: Overcast

DEPTH (ft)	GRAPHIC LOG	FROZEN SOILS	MATERIAL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	SHELBY DOWN PRESSURE (psi)	LAB RESULTS	REMARKS/NOTES
0			**GRAVEL WITH SILT AND SAND** (GP-GM), some organics, loose, brown to black, damp to moist, FILL		S1	N/A	N/A	N/A	N/A	MC = 7.0% 62.1% gravel, 24.0% sand, 13.9% silt OC = 4.1%	
					S2	13	3 3 3	10	N/A	MC = 13.3%	Increased silt content at approx. 2.5' bgs.
5			**PEAT** (PT), loose, dark brown to black		S3	14	2 2 2	6	N/A	MC = 41.9%	
			▽ **SANDY SILT WITH GRAVEL** (SM), loose, brownish gray, moist to wet		S4	12	1 2 3	6	N/A	MC = 24.2% 4.0% gravel, 47.8% sand, 48.2% silt P0.02 = 31.9% FC = F3	
10			**SILT** (ML), with sand, and gravel, soft, brown to gray, lensed, moist to wet, low plasticity, occasional sand lenses		S5	16	1 1 10	9	N/A	MC = 29.3%	
15					S6	16	2 2 1	3	N/A	MC = 26.9%	

(Continued Next Page)

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EXPLORATION B-4

NGE-TFT PROJECT NAME: Ship Creek Condo Development
NGE-TFT PROJECT NUMBER: 4385-16
PROJECT LOCATION: Anchorage, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: CME-75 w/ 340lb autohammer
EXPLORATION METHOD: Hollow Stem Auger w/ center drill rods
SAMPLING METHOD: Modified Split-spoon/Thin-walled Shelby Tube
LOGGED BY: E. Boatwright
DATE STARTED: 5/5/2016
DATE COMPLETED: 5/5/2016
EXPLORATION LOCATION: See report figure 2
GROUND ELEVATION: Approx. 19 ft above mean sea level
▽ GROUNDWATER (ATD): Approx. 6.5 ft bgs
▼ GROUNDWATER (AD): N/A
EXPLORATION COMPLETION: Backfilled with cuttings.
WEATHER CONDITIONS: Overcast

DEPTH (ft)	GRAPHIC LOG	FROZEN SOILS	MATERIAL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	SHELBY DOWN PRESSURE (psi)	LAB RESULTS	REMARKS/NOTES
			SILT (ML), with sand, and gravel, soft, brown to gray, lensed, moist to wet, low plasticity, occasional sand lenses *(continued)*								
20					S7	16	1 1 1	2	N/A	MC = 31.5% P200 = 91.4% LL = 28 PL = 26 PI = 2	
25					S8	24	N/A	N/A	150 200 250 250	MC = 21.9% 0.0% gravel, 0.8% sand, 99.2% silt P0.02 = 70.5% LL = 23 PL = 20 PI = 3 FC = F4 P0.002 = 31.1% SG = 2.657	PP @ 26' bgs = 0.625 tsf TV @ 26' bgs = 0.4 tsf. PP @ 26.5' bgs = 0.5 tsf TV @ 26.5' bgs = 0.35 tsf.
30					S9	24	N/A	N/A	250 400 450 500		PP @ 31.08' bgs = 0.625 tsf TV @ 31.08' bgs = 0.2 tsf. PP @ 32' bgs = 0.25 tsf TV @ 32' bgs = 0.3 tsf.

Bottom of borehole at 32.0 feet bgs.

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PHOTO APPENDIX

CLIENT John McGrew **PROJECT NAME** Ship Creek Condo Development

PROJECT NUMBER 4385-16 **PROJECT LOCATION** Anchorage, AK

Exploration B-4 Sample S2
Sample Interval 2.5'-4' bgs

Exploration B-4 Sample S3
Sample Interval 5'-6.5' bgs

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CLIENT John McGrew **PROJECT NAME** Ship Creek Condo Development

PROJECT NUMBER 4385-16 **PROJECT LOCATION** Anchorage, AK

Exploration B-4 Sample S4
Sample Interval 10'-11.5' bgs

Exploration B-4 Sample S5
Sample Interval 10'-11.5' bgs

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PHOTO APPENDIX

CLIENT John McGrew **PROJECT NAME** Ship Creek Condo Development

PROJECT NUMBER 4385-16 **PROJECT LOCATION** Anchorage, AK

Exploration B-4 Sample S6
Sample Interval 15'-16.5' bgs

Exploration B-4 Sample S7
Sample Interval 20'-21.5' bgs

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PHOTO APPENDIX

CLIENT John McGrew

PROJECT NAME Ship Creek Condo Development

PROJECT NUMBER 4385-16

PROJECT LOCATION Anchorage, AK

Exploration B4 Sample S8
Sample Interval 25'-27' bgs
Extracted Shelby Tube

Exploration B4 Sample S9
Sample Interval 30'-32' bgs
Extracted Shelby Tube

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EXPLORATION B-5

NGE-TFT PROJECT NAME: Ship Creek Condo Development
NGE-TFT PROJECT NUMBER: 4385-16
PROJECT LOCATION: Anchorage, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: CME-75 w/ 340lb autohammer
EXPLORATION METHOD: Hollow Stem Auger w/ center drill rods
SAMPLING METHOD: Modified Split-spoon/Thin-walled Shelby Tube
LOGGED BY: E. Boatwright
DATE STARTED: 5/5/2016
DATE COMPLETED: 5/5/2016
EXPLORATION LOCATION: See report figure 2
GROUND ELEVATION: Approx. 20 ft above mean sea level
∇ **GROUNDWATER (ATD):** Approx. 15.0 ft bgs
▼ **GROUNDWATER (AD):** N/A
EXPLORATION COMPLETION: Backfilled with cuttings.
WEATHER CONDITIONS: Overcast

DEPTH (ft)	GRAPHIC LOG	FROZEN SOILS	MATERIAL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	SHELBY DOWN PRESSURE (psi)	LAB RESULTS	REMARKS/NOTES
0			**GRAVEL WITH SILT** (GP-GM), medium dense, dark brown to gray, damp to moist, FILL		S1	N/A	N/A	N/A	N/A	MC = 9.3%	
					S2	14	3 4 5	15	N/A	MC = 14.9% 3.5% gravel, 39.5% sand, 57.0% silt	Increased silt content at approx. 2.5' bgs.
5			**SILT** (ML), with sand, with gravel, medium stiff, brownish gray, FILL		S3	11	2 2 5	6	N/A	MC = 43.8% 8.1% gravel, 47.0% sand, 44.9% silt P0.02 = 35.9% FC = F3	
			PEAT (PT), loose, dark brown, moist								
			SILT (ML), with sand, with gravel, soft to medium stiff, gray to dark gray, moist to wet, low plasticity		S4	14	2 1 4	4	N/A	MC = 24.5%	
10					S5	14	2 4 4	7	N/A	MC = 17.9% LL = 28 PL = 24 PI = 4	
15		∇			S6	12	2 2 3	5	N/A	MC = 18.0% P200 = 88.8%	
20											

(Continued Next Page)

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EXPLORATION B-5

NGE-TFT PROJECT NAME: Ship Creek Condo Development
NGE-TFT PROJECT NUMBER: 4385-16
PROJECT LOCATION: Anchorage, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: CME-75 w/ 340lb autohammer
EXPLORATION METHOD: Hollow Stem Auger w/ center drill rods
SAMPLING METHOD: Modified Split-spoon/Thin-walled Shelby Tube
LOGGED BY: E. Boatwright
DATE STARTED: 5/5/2016
DATE COMPLETED: 5/5/2016
EXPLORATION LOCATION: See report figure 2
GROUND ELEVATION: Approx. 20 ft above mean sea level
▽ GROUNDWATER (ATD): Approx. 15.0 ft bgs
▼ GROUNDWATER (AD): N/A
EXPLORATION COMPLETION: Backfilled with cuttings.
WEATHER CONDITIONS: Overcast

DEPTH (ft)	GRAPHIC LOG	FROZEN SOILS	MATERIAL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	SHELBY DOWN PRESSURE (psi)	LAB RESULTS	REMARKS/NOTES
20			**SILT** (ML), with sand, with gravel, soft to medium stiff, gray to dark gray, moist to wet, low plasticity *(continued)*		S7	16	2 2 1	3	N/A	MC = 24.7%	
25					S8	24 N/A N/A	N/A	N/A	250 300 350 450		PP @ 25.66' bgs = 0.625 tsf TV @ 25.66' bgs = 0.35 tsf. PP @ 26.13' bgs = 0.5 tsf TV @ 26.13' bgs = 0.45 tsf.
30					S9	24 N/A N/A	N/A	N/A	300 400 450 500	MC = 33.4% 0.0% gravel, 0.4% sand, 99.6% silt P0.02 = 97.5% LL = 37 PL = 34 PI = 3 FC = F4 P0.002 = 65.7% SG = 2.652	PP @ 30.79' bgs = 0.5 tsf TV @ 30.79' bgs = 0.45 tsf. PP @ 31.66' bgs = 0.375 tsf TV @ 31.66' bgs = 0.40 tsf.

Bottom of borehole at 32.0 feet bgs.

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PHOTO APPENDIX

CLIENT John McGrew **PROJECT NAME** Ship Creek Condo Development

PROJECT NUMBER 4385-16 **PROJECT LOCATION** Anchorage, AK

Exploration B-5 Sample S2
Sample Interval 2.5'-4' bgs

Exploration B-5 Sample S3
Sample Interval 5'-6.5' bgs

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PHOTO APPENDIX

CLIENT John McGrew **PROJECT NAME** Ship Creek Condo Development

PROJECT NUMBER 4385-16 **PROJECT LOCATION** Anchorage, AK

Exploration B-5 Sample S4
Sample Interval 7.5'-9' bgs

Exploration B-5 Sample S5
Sample Interval 10'-11.5' bgs

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PHOTO APPENDIX

CLIENT John McGrew

PROJECT NAME Ship Creek Condo Development

PROJECT NUMBER 4385-16

PROJECT LOCATION Anchorage, AK

Exploration B-5 Sample S7
Sample Interval 20'-21.5' bgs

Exploration B5 Sample S8
Sample Interval 25'-27' bgs
Extracted Shelby Tube

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PHOTO APPENDIX

CLIENT John McGrew

PROJECT NAME Ship Creek Condo Development

PROJECT NUMBER 4385-16

PROJECT LOCATION Anchorage, AK

Exploration B5 Sample S9
Sample Interval 30'-32' bgs
Extracted Shelby Tube

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EXPLORATION LEGEND

CLIENT John McGrew
NGE-TFT PROJECT NAME Ship Creek Condo Development
NGE-TFT PROJECT NUMBER 4385-16
PROJECT LOCATION Anchorage, AK

LITHOLOGIC SYMBOLS
(Unified Soil Classification System)

GM: USCS Silty Gravel

GP: USCS Poorly-graded Gravel

GP-GM: USCS Poorly-graded Gravel with Silt

GPS: Sandy Gravel

ML: USCS Silt

PT: USCS Peat

SM: USCS Silty Sand

SAMPLER SYMBOLS

Grab Sample

Modified Penetration Test

Shelby Tube

WELL CONSTRUCTION SYMBOLS

ABBREVIATIONS

LL - LIQUID LIMIT (%)
PI - PLASTIC INDEX (%)
MC - MOISTURE CONTENT (%)
DD - DRY DENSITY (PCF)
NP - NON PLASTIC
P200 - PERCENT PASSING NO. 200 SIEVE
P0.02- PERCENT PASSING 0.02mm SIEVE
PP - POCKET PENETROMETER (TSF)
S/U - CASING STICK-UP

TV - TORVANE
PID - PHOTOIONIZATION DETECTOR
UC - UNCONFINED COMPRESSION
ppm - PARTS PER MILLION
Water Level at Time Drilling, or as Shown
Water Level After 24 Hours, or as Shown

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SOIL CLASSIFICATION CHART

CLIENT John McGrew

PROJECT NAME Ship Creek Condo Development

NGE-TFT PROJECT NUMBER 4385-16

PROJECT LOCATION Anchorage, AK

MAJOR DIVISIONS			SYMBOLS: GRAPH	SYMBOLS: LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.
DIAGONAL LINES INDICATE UNKNOWN DEPTH OF SOIL TRANSITION.

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EXPLORATION LOG KEY

CLIENT John McGrew

PROJECT NAME Ship Creek Condo Development

NGE-TFT PROJECT NUMBER 4385-16

PROJECT LOCATION Anchorage, AK

SAMPLER SYMBOLS

SPT w/ 140# Hammer
30" Drop and 2.0" O.D. Sampler

Modified SPT w/ 340# Hammer
30" Drop and 3.0 O.D. Sampler

Grab Sample

Shelby Tube Sample

Rock Core Sample

Direct Push Sample

No Recovery

N/E Not Encountered

WELL SYMBOLS

1" Slotted Pipe
Backfilled with Silica Sand

1" PVC Pipe
Backfilled with Auger Cuttings

1" PVC Pipe
with Bentonite Seal

Capped Riser

COMPONENT DEFINITIONS

COMPONENT	SIZE RANGE
Boulders	Larger than 12 in
Cobbles	3 in to 12 in
Gravel	3 in to No. 4 (4.5mm)
Coarse gravel	3 in to 3/4 in
Fine gravel	3/4 in to No. 4 (4.5 mm)
Sand	No. 4 (4.5 mm) to No. 200
Coarse sand	No. 4 (4.5 mm) to No. 10 (2.0 mm)
Medium sand	No. 10 (2.0 mm) to No. 40 (0.42 mm)
Fine sand	No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt and Clay	Smaller than No. 200 (0.074 mm)

COMPONENT PROPORTIONS

DESCRIPTIVE TERMS	RANGE OF PROPORTION
Trace	1-5%
Few	5-10%
Little	10-20%
Some	20-35%
And	35-50%

MOISTURE CONTENT

DRY	Absence of moisture, dusty, dry to the touch
DAMP	Some perceptible moisture; below optimum
MOIST	No visible water; near optimum moisture content
WET	Visible free water, usually soil is below water table

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N-VALUE

COHESIONLESS SOILS			COHESIVE SOILS		
DENSITY	N (BLOWS/FT)	APPROXIMATE RELATIVE DENSITY (%)	CONSISTENCY	N (BLOWS/FT)	APPROXIMATE UNDRAINED SHEAR STRENGTH (PSF)
VERY LOOSE	0-4	0-15	VERY SOFT	0-1	< 250
LOOSE	5-10	15-35	SOFT	2-4	250-500
MEDIUM DENSE	11-25	35-65	MEDIUM STIFF	5-8	500-1000
DENSE	26-50	65-85	STIFF	9-15	1000-2000
VERY DENSE	> 50	85-100	VERY STIFF	16-30	2000-4000
			HARD	> 30	> 4000

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EXPLORATION LOG KEY

CLIENT John McGrew

PROJECT NAME Ship Creek Condo Development

NGE-TFT PROJECT NUMBER 4385-16

PROJECT LOCATION Anchorage, AK

FROST DESIGN SOIL CLASSIFICATION

FROST GROUP (USACOE)	FROST GROUP (M.O.A.)	SOIL TYPE	% FINER THAN 0.02mm BY MASS	TYPICAL SOIL TYPES UNDER UNIFIED SOIL CLASSIFICATION SYSTEM
NFS*	NFS*	(A) GRAVELS CRUSHED STONE CRUSHED ROCK (B) SANDS	0 - 1.5 0 - 3	GW, GP SW, SP
PFS+	NFS*	(A) GRAVELS CRUSHED STONE CRUSHED ROCK	1.5 - 3	GW, GP
	F2	(B) SANDS	3 - 10	SW, SP
S1	F1	GRAVELLY SOILS	3 - 6	GW, GP, GW-GM, GP-GM
S2	F2	SANDY SOILS	3 - 6	SW, SP, SW-SM, SP-SM
F1	F1	GRAVELLY SOILS	6 - 10	GM, GW-GM, GP-GM
F2	F2	(A) GRAVELLY SOILS (B) SANDS	10 - 20 6 - 15	GM, GW-GM, GP-GM SM, SW-SM, SP-SM
F3	F3	(A) GRAVELLY SOILS (B) SANDS, EXCEPT VERY FINE SILTY SANDS (C) CLAYS, PI>12	Over 20 Over 15 ------	GM, GC SM, SC CL, CH
F4	F4	(A) ALL SILTS (B) VERY FINE SILTY SANDS (C) CLAYS, PI<12 (D) VARVED CLAYS AND OTHER FINE GRAINED, BANDED SEDIMENTS	------ Over 15 ------ ------	ML, MH SM CL, CL-ML CL & ML; CL, ML, & SM; CL, CH, & ML; CL, CH, ML, & SM

*Non-frost susceptible
+Possibly frost susceptible, but requires lab testing to determine frost design soils classification.

ICE CLASSIFICATION SYSTEM

GROUP	ICE VISIBILITY	DESCRIPTION		SYMBOL	
N	SEGREGATED ICE NOT VISIBLE BY EYE	POORLY BONDED OR FRIABLE		Nf	
		WELL BONDED	NO EXCESS ICE	Nb	Nbn
			EXCESS MICROSCOPIC ICE		Nbe
V	SEGREGATED ICE IS VISIBLE BY EYE AND IS ONE INCH OR LESS IN THICKNESS	INDIVIDUAL ICE CRYSTALS OR INCLUSIONS		Vx	
		ICE COATINGS ON PARTICLES		Vc	
		RANDOM OR IRREGULARY ORIENTED ICE		Vr	
		STRATIFIED OR DISTINCTLY ORIENTED ICE		Vs	
		UNIFORMLY DISTRIBUTED ICE		Vu	
ICE	ICE IS GREATER THAN ONE INCH IN THICKNESS	ICE WITH SOILS INCLUSIONS		ICE + Soil Type	
		ICE WITHOUT SOILS INCLUSIONS		ICE	

APPENDIX C

LABORATORY DATA

Summary of Laboratory Test Results

Ship Creek Condo Development

NGE-TFT Project #:4385-16

Exploration ID	Sample Number	Depth Interval (ft) Top	Depth Interval (ft) Bottom	Moisture Content ASTM D2216 (% By Dry Mass)	Atterberg Limits ASTM D4318 LL	PL	PI	Particle Size Analysis ASTM C136/D422/D6913 (% By Mass) Gravel	Sand	Silt/Clay	Passing #200 ASTM D1140 (% By Mass)	Passing 0.02mm ASTM D422 (% By Mass)	Passing 0.002mm ASTM D423 (% By Mass)	Frost Class. (MOA)	Specific Gravity ASTM D854	Organic Content (ASTM D2974) (% By Mass)	Unified Soil Classification ASTM D2487
B1	S1	0.0	1.0	1.0													
B1	S2	2.5	4.0	4.7				44.2	44.0	11.8		8.4		F1			(GW-GM) Well-graded gravel w/ silt and sand
B1	S3	5.0	6.5	7.3													
B1	S4	7.5	9.0	7.2				31.0	56.3	12.7							(SM) Silty sand w/ gravel
B1	S5	10.0	11.5	8.2													
B1	S6a	15.0	15.8	3.7													
B1	S6b	15.8	16.5	11.1				26.4	47.7	25.9		20.1		F3			(SM) Silty sand w/ gravel
B1	S7	20.0	21.5	3.5													
B1	S8	25.0	26.5	14.0							67.8						
B1	S9	30.0	31.5	20.4	28	24	4										
B1	S10	35.0	36.5	20.8													
B1	S11	40.0	41.5	37.1							45.2						
B1	S13	50.0	52.0	29.2	38	25	13	0.0	1.3	98.7		88.3	55.4	F4	2.694		(ML) Silt
B2	S1	0.0	1.0	3.1													
B2	S2	2.5	4.0	16.0				36.9	40.3	22.8						3.0	(SM) Silty sand w/ gravel
B2	S3	5.0	6.5	13.6				14.7	56.2	29.1		16.4		F3			(SM) Silty sand
B2	S4	7.5	9.0	33.2	28	27	1										
B2	S5a	10.0	10.8	17.5													
B2	S5b	10.8	11.5	31.0													
B2	S6	15.0	16.5	28.2							97.4						
B2	S7	20.0	21.5	31.6													
B2	S8	25.0	26.5	36.6													
B2	S9	30.0	31.5	29.2													
B2	S10	35.0	36.5	30.2													
B2	S11	40.0	41.5	26.1							99.6						
B2	S12	45.0	47.0	28.9	34	28	6	0.0	0.1	99.9		95.0	53.1	F4	2.681		(ML) Silt
B2	S13	50.0	52.0	28.5	29	21	8	0.0	0.2	99.8		84.7	36.4	F4	2.633		(CL) Lean Clay
B3	S1	0.0	1.0	7.0				37.5	39.9	22.6						3.9	(SM) Silty sand w/ gravel
B3	S2	2.5	4.0	17.1													
B3	S3	5.0	6.5	27.3				13.1	43.7	43.2		28.8		F3			(SM) Silty sand
B3	S4	7.5	9.0	22.3												6.2	
B3	S5	10.0	11.5	25.0													
B3	S6	15.0	16.5	30.6	28	28	0										
B3	S7	20.0	21.5	36.2													
B3	S9	30.0	31.5	26.2							97.4						
B3	S10	35.0	37.0	31.0	37	29	8	0.0	2.4	97.6		94.0	52.1	F4	2.671		(SM) Silt
B4	S1	0.0	1.0	7.0				62.1	24.0	13.9						4.1	(GM) Silty gravel w/ sand
B4	S2	2.5	4.0	13.3													
B4	S3	5.0	6.5	41.9													
B4	S4	7.5	9.0	24.2				4.0	47.8	48.2		31.9		F3			(SM) Silty sand
B4	S5	10.0	11.5	29.3													
B4	S6	15.0	16.5	26.9													
B4	S7	20.0	21.5	31.5	28	26	2				91.4						
B4	S8	25.0	27.0	21.9	23	20	3	0.0	0.8	99.2		70.5	31.1	F4	2.657		(ML) Silt
B5	S1	0.0	1.0	9.3													
B5	S2	2.5	4.0	14.9				3.5	39.5	57.0							(ML) Sandy silt
B5	S3	5.0	6.5	43.8				8.1	47.0	44.9		35.9		F3			(SM) Silty sand
B5	S4	7.5	9.0	24.5													
B5	S5	10.0	11.5	17.9	28	24	4										
B5	S6	15.0	16.5	18.0							88.8						
B5	S7	20.0	21.5	24.7													
B5	S9	30.0	32.0	33.4	37	34	3	0.0	0.4	99.6		97.5	65.7	F4	2.652		(ML) Silt

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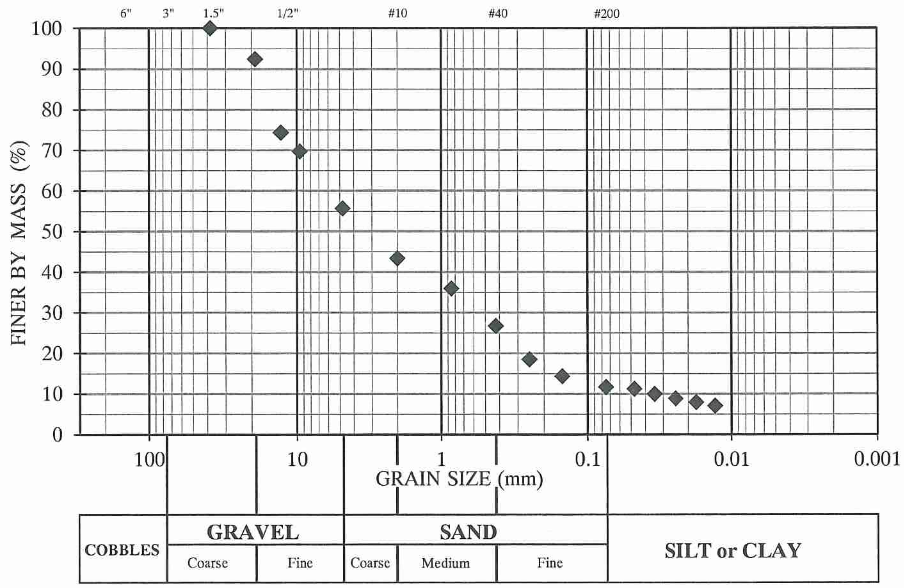
PROJECT CLIENT:	**John McGrew**
PROJECT NAME:	**Ship Creek Condo Development**
PROJECT NO.:	**4385-16**
SAMPLE LOC.:	**B1**
NUMBER/ DEPTH:	**S2 / 2.5 - 4'**
DESCRIPTION:	**Well-graded gravel w/ silt and sand**
DATE RECEIVED:	**5/9/2016**
TESTED BY:	**JA**
REVIEWED BY:	**ALF**

% GRAVEL	**44.2**	USCS	**GW-GM**
% SAND	**44.0**	MOA FC	**F1**
% SILT/CLAY	**11.8**	% PASS. 0.02 mm	**8.4**
% MOIST. CONTENT	**5.8**	% PASS. 0.002 mm	**N/A**
UNIFORMITY COEFFICIENT (C_u)		**180.7**	
COEFFICIENT OF GRADATION (C_c)		**1.5**	
ASTM D1557 (uncorrected)		**N/A**	
ASTM D4718 (corrected)		**N/A**	
OPTIMUM MOIST. CONTENT. (corrected)		**N/A**	

PARTICLE SIZE ANALYSIS ASTM D422 / C136



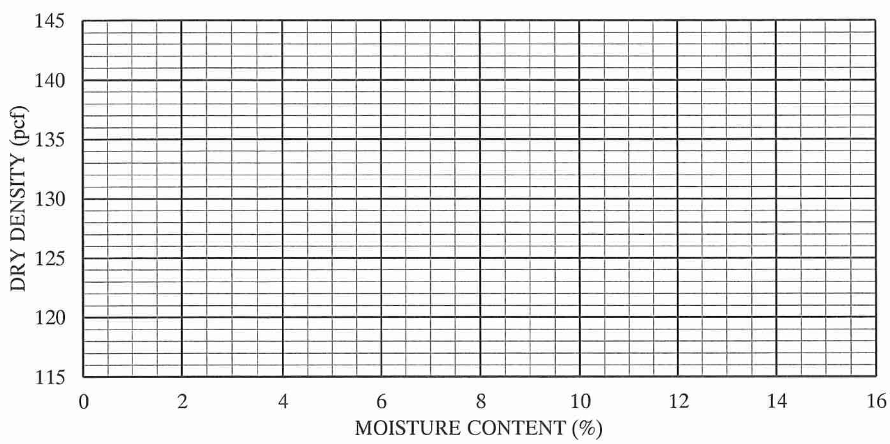
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
76.20	3"		
38.10	1.5"	**100**	
19.00	3/4"	**92**	
12.70	1/2"	**74**	
9.50	3/8"	**70**	
4.75	#4	**56**	
2.00	#10	**43**	
0.85	#20	**36**	
0.43	#40	**27**	
0.25	#60	**19**	
0.15	#100	**14**	
0.075	#200	**11.8**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1	0.0474	11.3
2	0.0344	10.0
4	0.0245	9.0
8	0.0176	8.0
15	0.0130	7.1
30		
60		
250		
1440		

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

The testing services reported herein have been performed to recognized industry standards, unless otherwise noted. No other warranty is made. Should engineering interpretation or opinion be required, NGE-TFT will provide upon written request.

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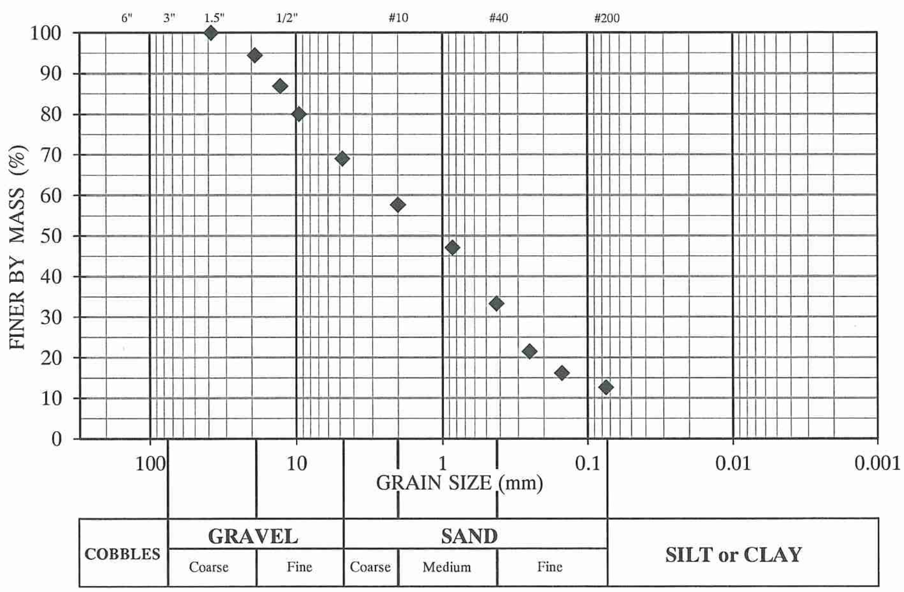
PROJECT CLIENT:	John McGrew
PROJECT NAME:	Ship Creek Condo Development
PROJECT NO.:	4385-16
SAMPLE LOC.:	B1
NUMBER/ DEPTH:	S4 / 7.5 - 9'
DESCRIPTION:	Silty sand w/ gravel
DATE RECEIVED:	5/9/2016
TESTED BY:	JA
REVIEWED BY:	ALF

% GRAVEL	31.0	USCS	SM
% SAND	56.3	USACOE FC	N/A
% SILT/CLAY	12.7	% PASS. 0.02 mm	N/A
% MOIST. CONTENT	7.2	% PASS. 0.002 mm	N/A
UNIFORMITY COEFFICIENT (C_u)		UNKNOWN	
COEFFICIENT OF GRADATION (C_c)		UNKNOWN	
ASTM D1557 (uncorrected)		N/A	
ASTM D4718 (corrected)		N/A	
OPTIMUM MOIST. CONTENT. (corrected)		N/A	

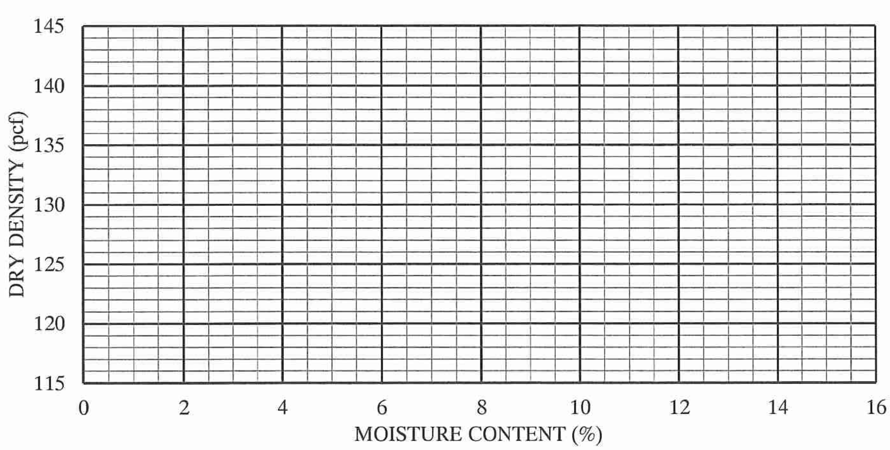
PARTICLE SIZE ANALYSIS ASTM D422 / C136



SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
76.20	3"		
38.10	1.5"	100	
19.00	3/4"	94	
12.70	1/2"	87	
9.50	3/8"	80	
4.75	#4	69	
2.00	#10	58	
0.85	#20	47	
0.43	#40	33	
0.25	#60	22	
0.15	#100	16	
0.075	#200	12.7	

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1		
2		
4		
8		
15		
30		
60		
250		
1440		

HYDRAULIC COND. (ASTM D2434)	N/A
DEGRADATION (ATM T-313)	N/A
PLASTICITY INDEX ASTM 4318	N/A

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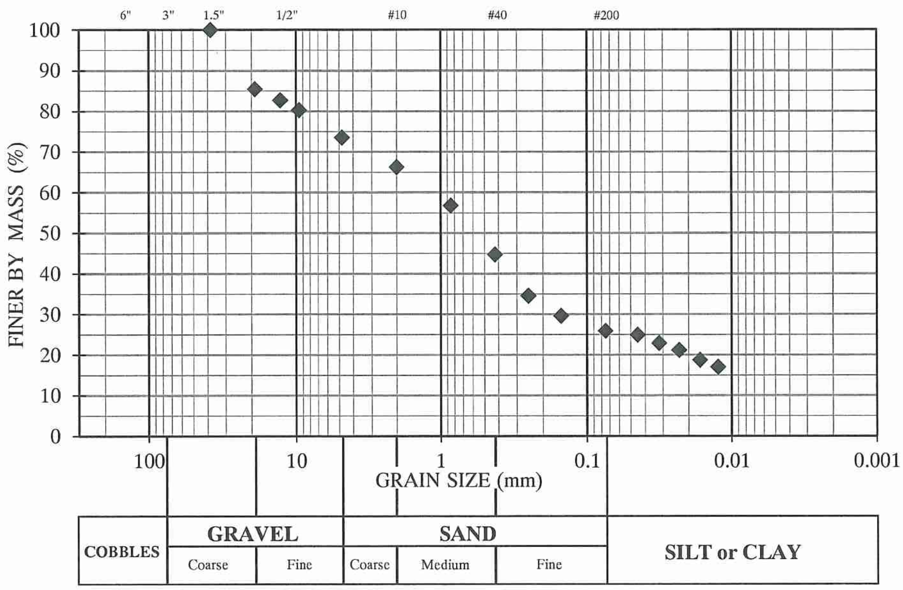
PROJECT CLIENT:	**John McGrew**
PROJECT NAME:	**Ship Creek Condo Development**
PROJECT NO.:	**4385-16**
SAMPLE LOC.:	**B1**
NUMBER/ DEPTH:	**S6b / 15.75 - 16.5'**
DESCRIPTION:	**Silty sand w/ gravel**
DATE RECEIVED:	**5/9/2016**
TESTED BY:	**JA**
REVIEWED BY:	**ALF**

% GRAVEL	**26.4**	USCS	**SM**
% SAND	**47.7**	MOA FC	**F3**
% SILT/CLAY	**25.9**	% PASS. 0.02 mm	**20.1**
% MOIST. CONTENT	**11.1**	% PASS. 0.002 mm	**N/A**

UNIFORMITY COEFFICIENT (C_u)	**UNKNOWN**
COEFFICIENT OF GRADATION (C_c)	**UNKNOWN**
ASTM D1557 (uncorrected)	**N/A**
ASTM D4718 (corrected)	**N/A**
OPTIMUM MOIST. CONTENT. (corrected)	**N/A**

PARTICLE SIZE ANALYSIS ASTM D422 / C136



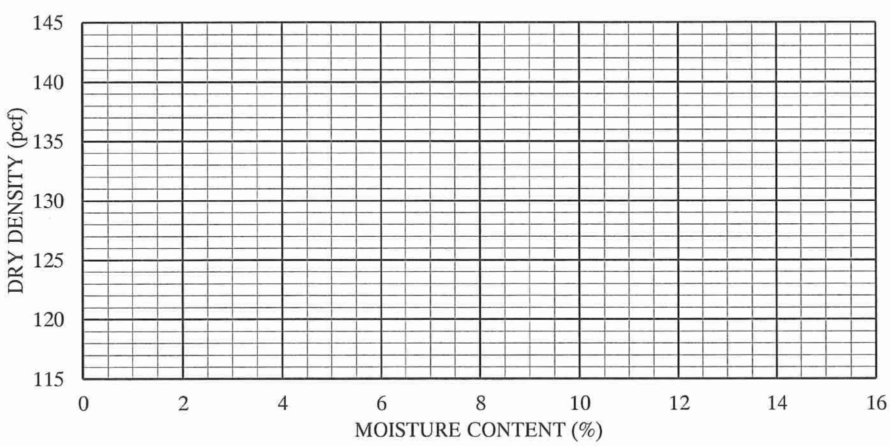
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
76.20	3"		
38.10	1.5"	**100**	
19.00	3/4"	**86**	
12.70	1/2"	**83**	
9.50	3/8"	**80**	
4.75	#4	**74**	
2.00	#10	**66**	
0.85	#20	**57**	
0.43	#40	**45**	
0.25	#60	**35**	
0.15	#100	**30**	
0.075	#200	**25.9**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1	0.0448	25.0
2	0.0320	22.9
4	0.0232	21.2
8	0.0166	18.8
15	0.0124	17.1
30		
60		
250		
1440		

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

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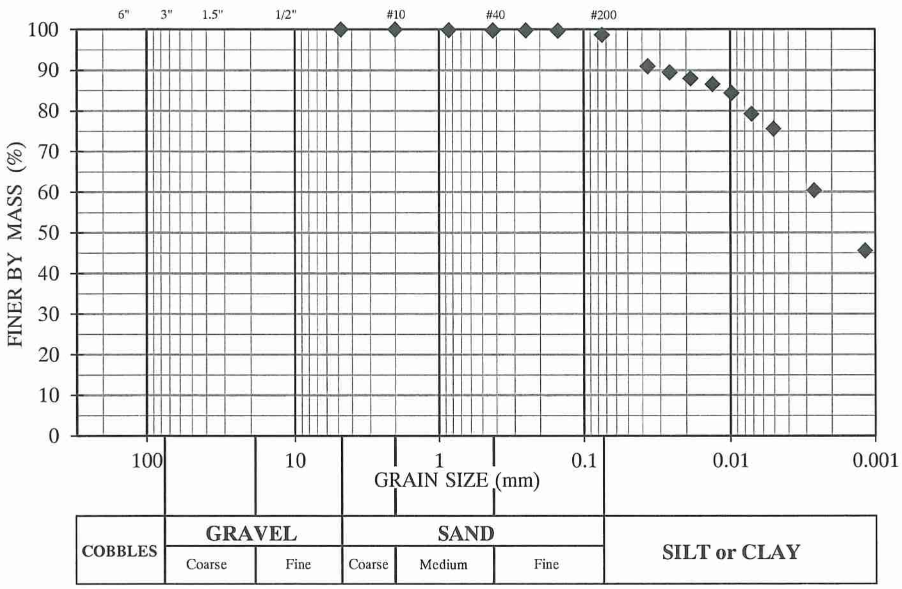
PROJECT CLIENT:	**John McGrew**
PROJECT NAME:	**Ship Creek Condo Development**
PROJECT NO.:	**4385-16**
SAMPLE LOC.:	**B1**
NUMBER/ DEPTH:	**S13 / 50-52'**
DESCRIPTION:	**Silt**
DATE RECEIVED:	**5/9/2016**
TESTED BY:	**JA**
REVIEWED BY:	**ALF**

% GRAVEL	**0.0**	USCS	**ML**
% SAND	**1.3**	MOA FC	**F4**
% SILT/CLAY	**98.7**	% PASS. 0.02 mm	**88.3**
% MOIST. CONTENT	**29.2**	% PASS. 0.002 mm	**55.4**
UNIFORMITY COEFFICIENT (C_u)		**UNKNOWN**	
COEFFICIENT OF GRADATION (C_c)		**UNKNOWN**	
ASTM D1557 (uncorrected)		**N/A**	
ASTM D4718 (corrected)		**N/A**	
OPTIMUM MOIST. CONTENT. (corrected)		**N/A**	

PARTICLE SIZE ANALYSIS ASTM D422 / C136



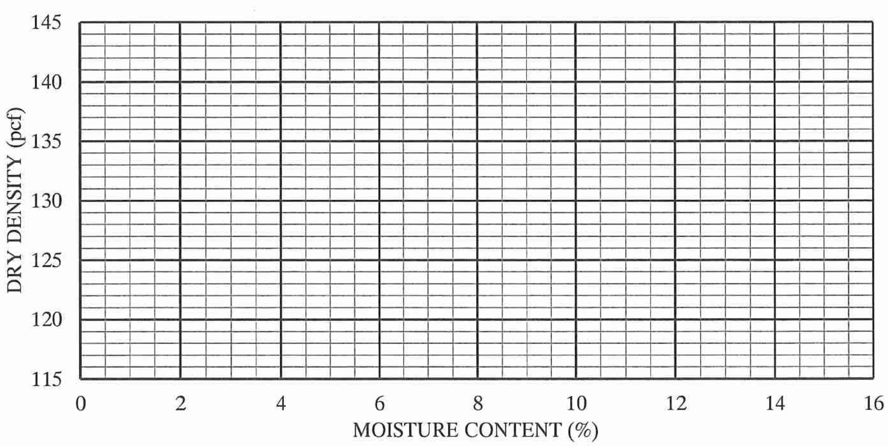
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
76.20	3"		
38.10	1.5"		
19.00	3/4"		
12.70	1/2"		
9.50	3/8"		
4.75	#4	**100**	
2.00	#10	**100**	
0.85	#20	**100**	
0.43	#40	**100**	
0.25	#60	**100**	
0.15	#100	**100**	
0.075	#200	**98.7**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1	0.0366	90.9
2	0.0259	89.5
4	0.0187	88.0
8	0.0132	86.5
15	0.0098	84.4
30	0.0072	79.3
60	0.0050	75.6
250	0.0026	60.5
1440	0.0012	45.7

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

The testing services reported herein have been performed to recognized industry standards, unless otherwise noted. No other warranty is made. Should engineering interpretation or opinion be required, NGE-TFT will provide upon written request.

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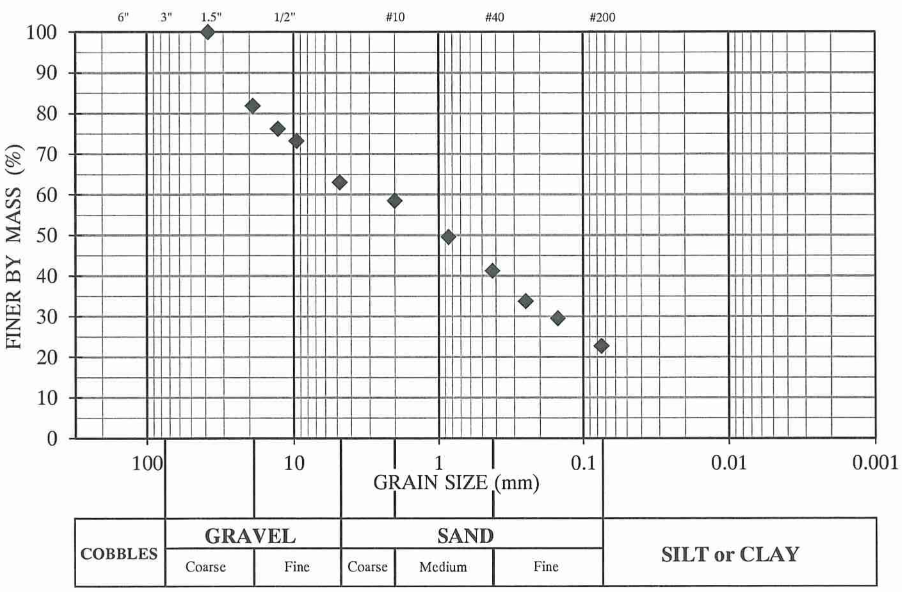
PROJECT CLIENT:	**John McGrew**
PROJECT NAME:	**Ship Creek Condo Development**
PROJECT NO.:	**4385-16**
SAMPLE LOC.:	**B2**
NUMBER/ DEPTH:	**S2 / 2.5 - 4'**
DESCRIPTION:	**Silty sand w/ gravel**
DATE RECEIVED:	**5/9/2016**
TESTED BY:	**JA**
REVIEWED BY:	**ALF**

% GRAVEL	**36.9**	USCS	**SM**
% SAND	**40.3**	USACOE FC	**N/A**
% SILT/CLAY	**22.8**	% PASS. 0.02 mm	**N/A**
% MOIST. CONTENT	**16.0**	% PASS. 0.002 mm	**N/A**
UNIFORMITY COEFFICIENT (C_u)		**UNKNOWN**	
COEFFICIENT OF GRADATION (C_c)		**UNKNOWN**	
ASTM D1557 (uncorrected)		**N/A**	
ASTM D4718 (corrected)		**N/A**	
OPTIMUM MOIST. CONTENT. (corrected)		**N/A**	

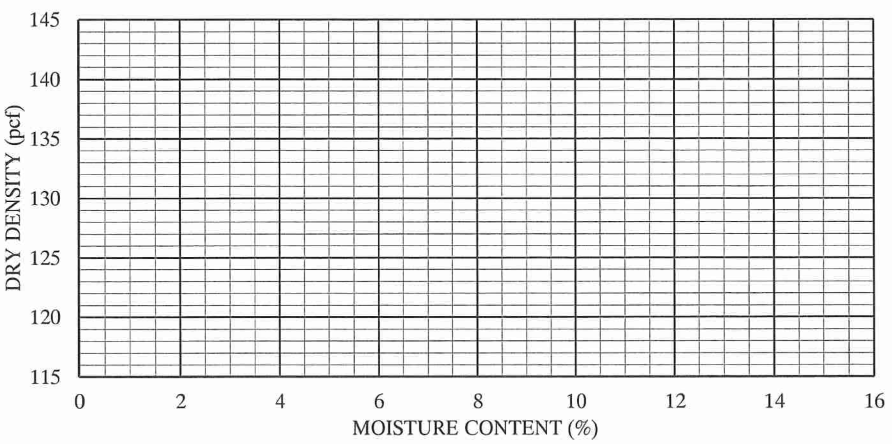
PARTICLE SIZE ANALYSIS ASTM D422 / C136



SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
76.20	3"		
38.10	1.5"	**100**	
19.00	3/4"	**82**	
12.70	1/2"	**76**	
9.50	3/8"	**73**	
4.75	#4	**63**	
2.00	#10	**59**	
0.85	#20	**50**	
0.43	#40	**41**	
0.25	#60	**34**	
0.15	#100	**30**	
0.075	#200	**22.8**	

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1		
2		
4		
8		
15		
30		
60		
250		
1440		

HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

The testing services reported herein have been performed to recognized industry standards, unless otherwise noted. No other warranty is made. Should engineering interpretation or opinion be required, NGE-TFT will provide upon written request.

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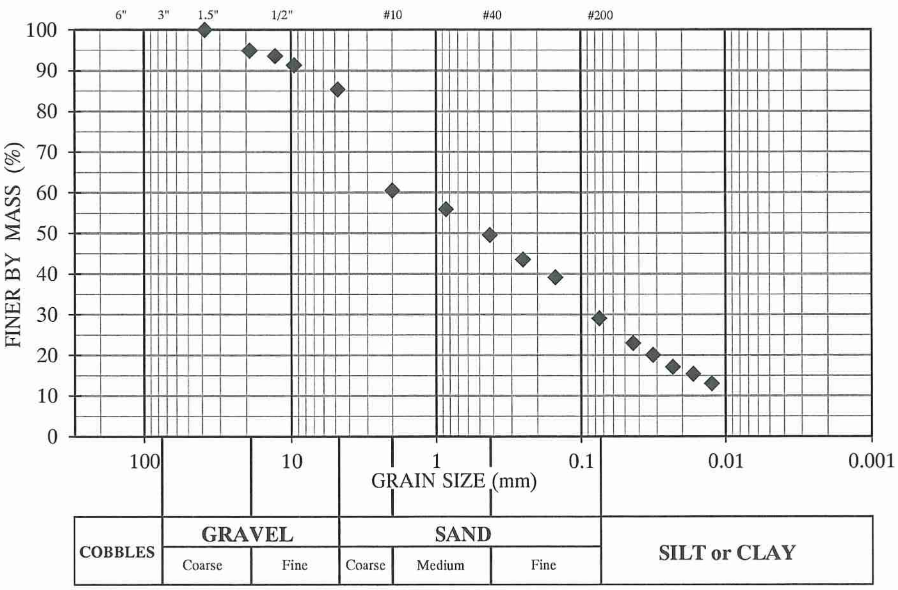
PROJECT CLIENT:	**John McGrew**
PROJECT NAME:	**Ship Creek Condo Development**
PROJECT NO.:	**4385-16**
SAMPLE LOC.:	**B2**
NUMBER/ DEPTH:	**S3 / 5 - 6.5'**
DESCRIPTION:	**Silty sand**
DATE RECEIVED:	**5/9/2016**
TESTED BY:	**JA**
REVIEWED BY:	**ALF**

% GRAVEL	**14.7**	USCS	**SM**
% SAND	**56.2**	MOA FC	**F3**
% SILT/CLAY	**29.1**	% PASS. 0.02 mm	**16.4**
% MOIST. CONTENT	**13.6**	% PASS. 0.002 mm	**N/A**

UNIFORMITY COEFFICIENT (C_u)	**UNKNOWN**
COEFFICIENT OF GRADATION (C_c)	**UNKNOWN**
ASTM D1557 (uncorrected)	**N/A**
ASTM D4718 (corrected)	**N/A**
OPTIMUM MOIST. CONTENT. (corrected)	**N/A**

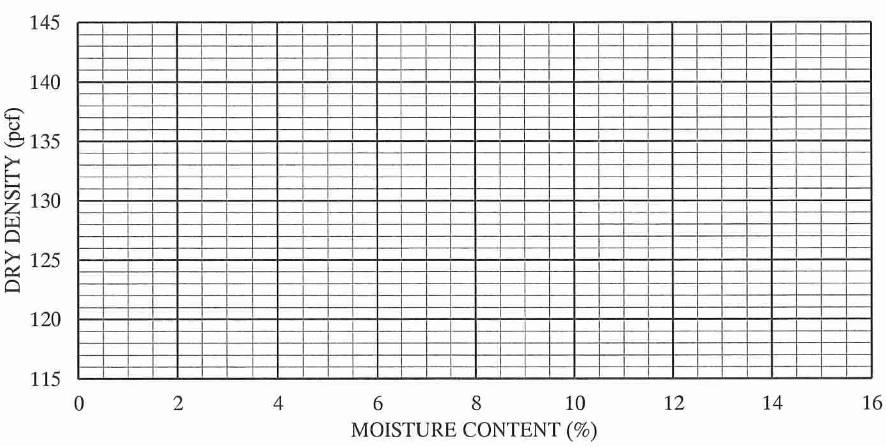
PARTICLE SIZE ANALYSIS ASTM D422 / C136



SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
76.20	3"		
38.10	1.5"	**100**	
19.00	3/4"	**95**	
12.70	1/2"	**94**	
9.50	3/8"	**91**	
4.75	#4	**85**	
2.00	#10	**61**	
0.85	#20	**56**	
0.43	#40	**50**	
0.25	#60	**44**	
0.15	#100	**39**	
0.075	#200	**29.1**	

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1	0.0437	23.0
2	0.0320	20.1
4	0.0232	17.2
8	0.0168	15.4
15	0.0125	13.1
30		
60		
250		
1440		

HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

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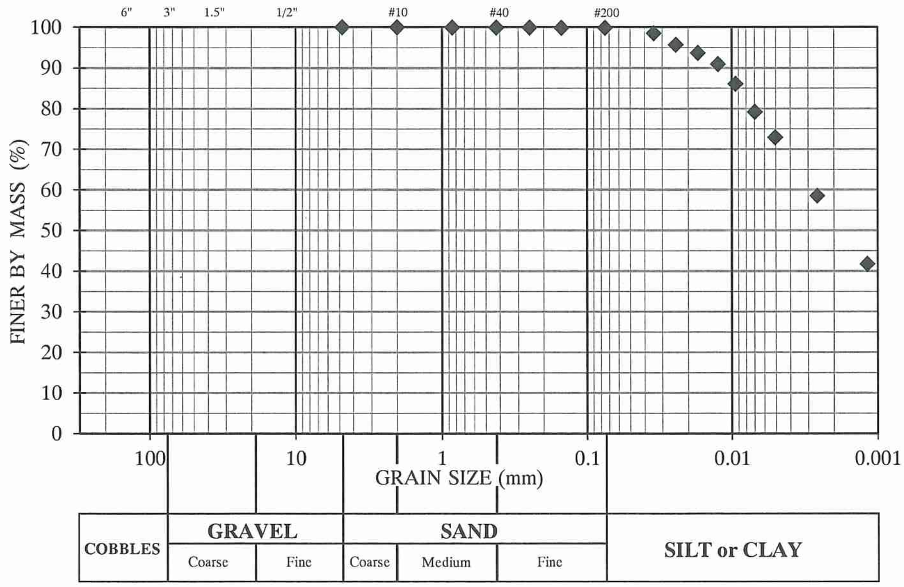
PROJECT CLIENT:	**John McGrew**
PROJECT NAME:	**Ship Creek Condo Development**
PROJECT NO.:	**4385-16**
SAMPLE LOC.:	**B2**
NUMBER/ DEPTH:	**S12 / 45.0'-47.0'**
DESCRIPTION:	**Silt**
DATE RECEIVED:	**6/30/2016**
TESTED BY:	**JA**
REVIEWED BY:	**ALF**

% GRAVEL	**0.0**	USCS	**ML**
% SAND	**0.1**	MOA FC	**F4**
% SILT/CLAY	**99.9**	% PASS. 0.02 mm	**95.0**
% MOIST. CONTENT	**28.9**	% PASS. 0.002 mm	**53.1**
UNIFORMITY COEFFICIENT (C_u)		**UNKNOWN**	
COEFFICIENT OF GRADATION (C_c)		**UNKNOWN**	
ASTM D1557 (uncorrected)		**N/A**	
ASTM D4718 (corrected)		**N/A**	
OPTIMUM MOIST. CONTENT. (corrected)		**N/A**	

PARTICLE SIZE ANALYSIS ASTM D422 / C136



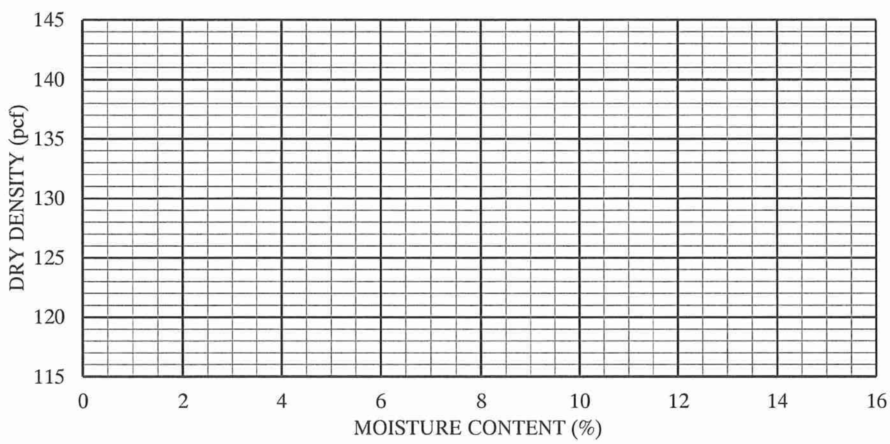
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
76.20	3"		
38.10	1.5"		
19.00	3/4"		
12.70	1/2"		
9.50	3/8"		
4.75	#4	**100**	
2.00	#10	**100**	
0.85	#20	**100**	
0.43	#40	**100**	
0.25	#60	**100**	
0.15	#100	**100**	
0.075	#200	**99.9**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1	0.0344	98.5
2	0.0243	95.7
4	0.0172	93.7
8	0.0125	90.9
15	0.0094	86.1
30	0.0070	79.2
60	0.0050	72.9
250	0.0026	58.6
1440	0.0012	41.8

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

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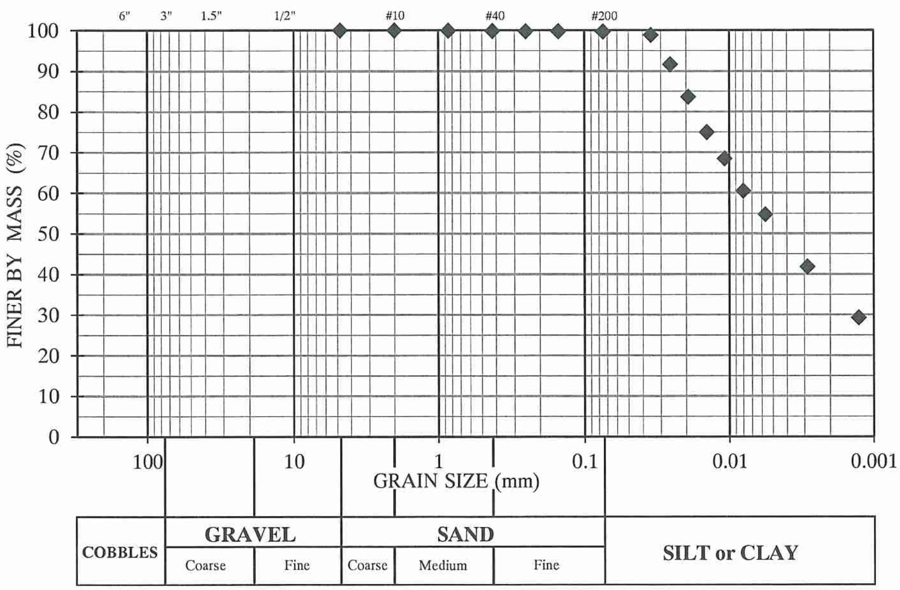
PROJECT CLIENT:	John McGrew
PROJECT NAME:	Ship Creek Condo Development
PROJECT NO.:	4385-16
SAMPLE LOC.:	B2
NUMBER/ DEPTH:	S13 / 50.0'-52.0'
DESCRIPTION:	Lean Clay
DATE RECEIVED:	6/30/2016
TESTED BY:	JA
REVIEWED BY:	ALF

% GRAVEL	0.0	USCS	CL
% SAND	0.2	MOA FC	F4
% SILT/CLAY	99.8	% PASS. 0.02 mm	84.7
% MOIST. CONTENT	28.5	% PASS. 0.002 mm	36.4
UNIFORMITY COEFFICIENT (C_u)		UNKNOWN	
COEFFICIENT OF GRADATION (C_c)		UNKNOWN	
ASTM D1557 (uncorrected)		N/A	
ASTM D4718 (corrected)		N/A	
OPTIMUM MOIST. CONTENT. (corrected)		N/A	

PARTICLE SIZE ANALYSIS ASTM D422 / C136



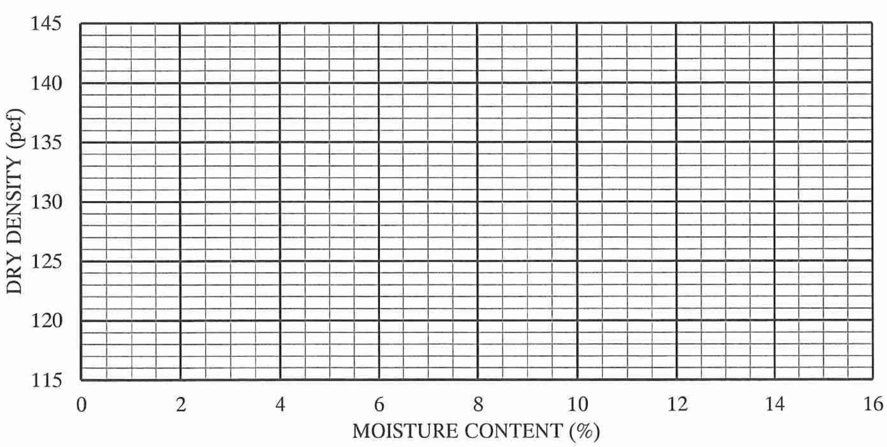
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
76.20	3"		
38.10	1.5"		
19.00	3/4"		
12.70	1/2"		
9.50	3/8"		
4.75	#4	100	
2.00	#10	100	
0.85	#20	100	
0.43	#40	100	
0.25	#60	100	
0.15	#100	100	
0.075	#200	99.8	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1	0.0349	98.9
2	0.0259	91.7
4	0.0193	83.7
8	0.0144	75.0
15	0.0108	68.5
30	0.0080	60.6
60	0.0056	54.7
250	0.0029	41.9
1440	0.0013	29.4

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDRAULIC COND. (ASTM D2434)	N/A
DEGRADATION (ATM T-313)	N/A
PLASTICITY INDEX ASTM 4318	N/A

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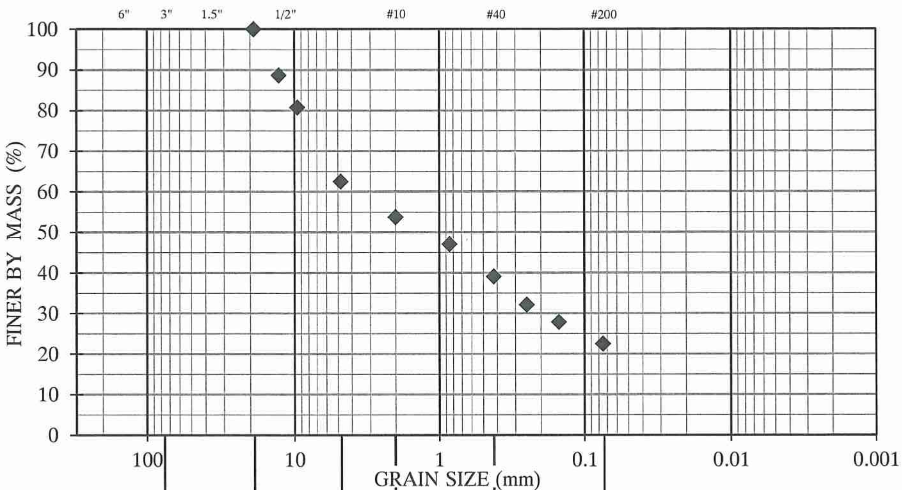
PROJECT CLIENT:	**John McGrew**
PROJECT NAME:	**Ship Creek Condo Development**
PROJECT NO.:	**4385-16**
SAMPLE LOC.:	**B3**
NUMBER/ DEPTH:	**S1 / 0 - 1'**
DESCRIPTION:	**Silty sand w/ gravel**
DATE RECEIVED:	**5/9/2016**
TESTED BY:	**JA**
REVIEWED BY:	**ALF**

% GRAVEL	**37.5**	USCS	**SM**
% SAND	**39.9**	USACOE FC	**N/A**
% SILT/CLAY	**22.6**	% PASS. 0.02 mm	**N/A**
% MOIST. CONTENT	**7.0**	% PASS. 0.002 mm	**N/A**

UNIFORMITY COEFFICIENT (C_u)	**UNKNOWN**
COEFFICIENT OF GRADATION (C_c)	**UNKNOWN**
ASTM D1557 (uncorrected)	**N/A**
ASTM D4718 (corrected)	**N/A**
OPTIMUM MOIST. CONTENT. (corrected)	**N/A**

PARTICLE SIZE ANALYSIS ASTM D422 / C136



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

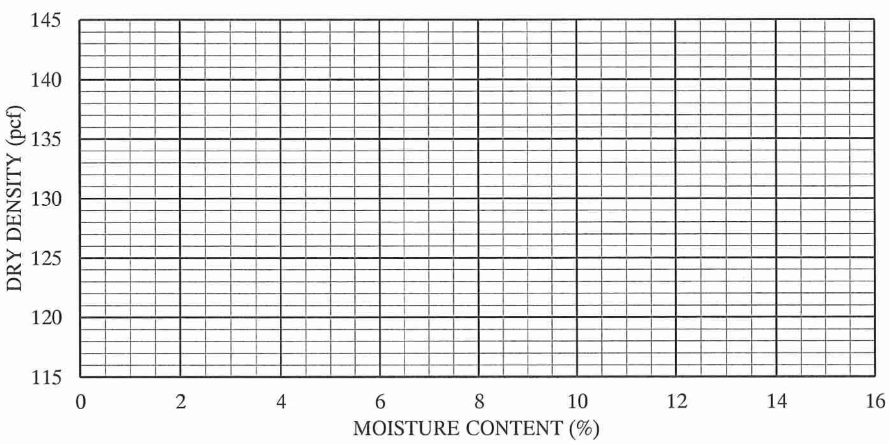
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
76.20	3"		
38.10	1.5"		
19.00	3/4"	**100**	
12.70	1/2"	**89**	
9.50	3/8"	**81**	
4.75	#4	**63**	
2.00	#10	**54**	
0.85	#20	**47**	
0.43	#40	**39**	
0.25	#60	**32**	
0.15	#100	**28**	
0.075	#200	**22.6**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1		
2		
4		
8		
15		
30		
60		
250		
1440		

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

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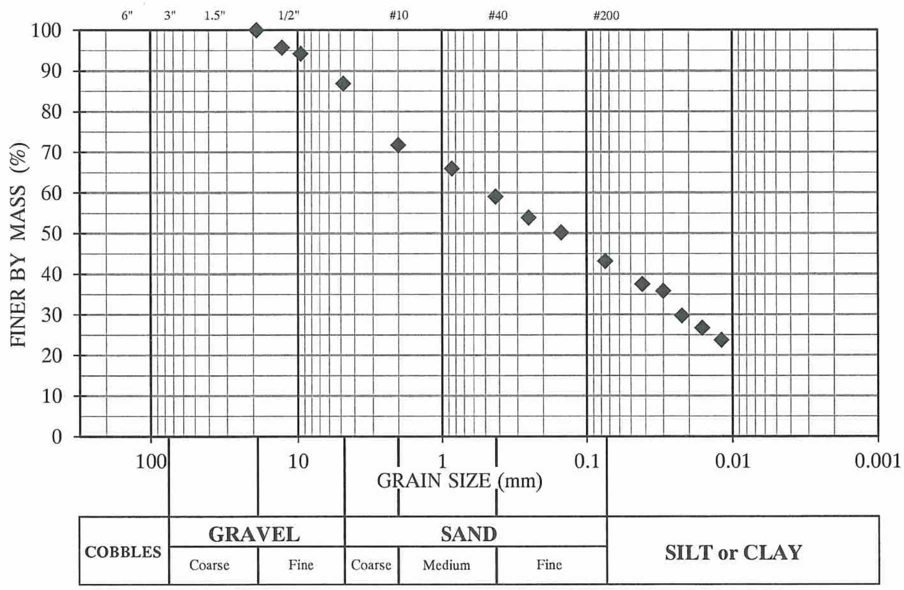
PROJECT CLIENT:	**John McGrew**
PROJECT NAME:	**Ship Creek Condo Development**
PROJECT NO.:	**4385-16**
SAMPLE LOC.:	**B3**
NUMBER/ DEPTH:	**S3 / 5 - 6.5'**
DESCRIPTION:	**Silty sand**
DATE RECEIVED:	**5/9/2016**
TESTED BY:	**JA**
REVIEWED BY:	**ALF**

% GRAVEL	**13.1**	USCS	**SM**
% SAND	**43.7**	MOA FC	**F3**
% SILT/CLAY	**43.2**	% PASS. 0.02 mm	**28.8**
% MOIST. CONTENT	**27.3**	% PASS. 0.002 mm	**N/A**
UNIFORMITY COEFFICIENT (C_u)		**UNKNOWN**	
COEFFICIENT OF GRADATION (C_c)		**UNKNOWN**	
ASTM D1557 (uncorrected)		**N/A**	
ASTM D4718 (corrected)		**N/A**	
OPTIMUM MOIST. CONTENT. (corrected)		**N/A**	

PARTICLE SIZE ANALYSIS ASTM D422 / C136



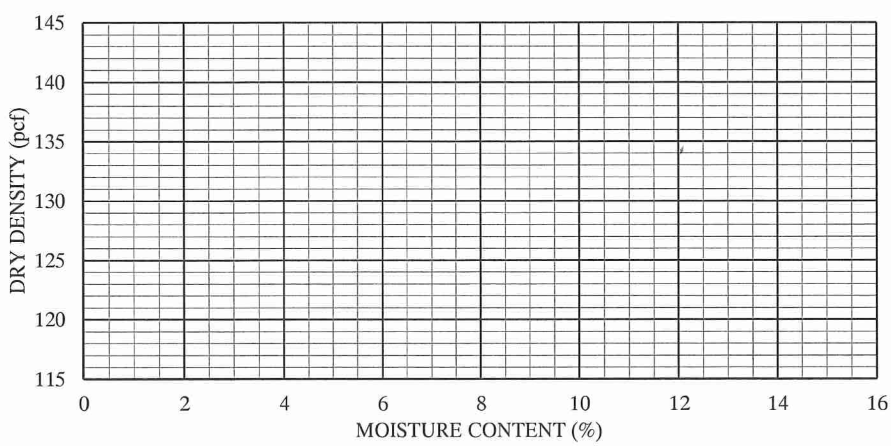
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
76.20	3"		
38.10	1.5"		
19.00	3/4"	**100**	
12.70	1/2"	**96**	
9.50	3/8"	**94**	
4.75	#4	**87**	
2.00	#10	**72**	
0.85	#20	**66**	
0.43	#40	**59**	
0.25	#60	**54**	
0.15	#100	**50**	
0.075	#200	**43.2**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1	0.0419	37.6
2	0.0301	35.9
4	0.0223	29.8
8	0.0162	26.8
15	0.0120	23.7
30		
60		
250		
1440		

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

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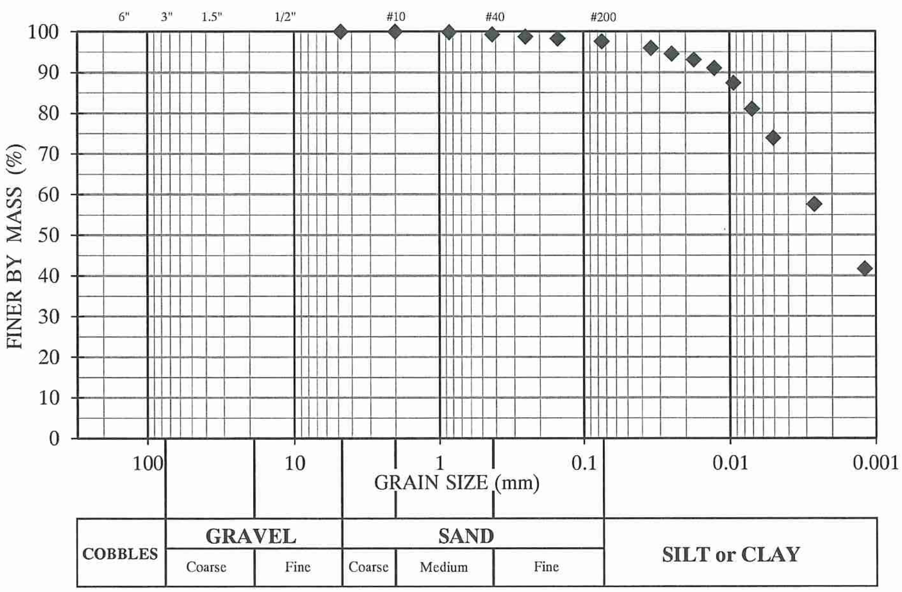
PROJECT CLIENT:	**John McGrew**
PROJECT NAME:	**Ship Creek Condo Development**
PROJECT NO.:	**4385-16**
SAMPLE LOC.:	**B3**
NUMBER/ DEPTH:	**S10 / 35.0'-37.0'**
DESCRIPTION:	**Silt**
DATE RECEIVED:	**6/30/2016**
TESTED BY:	**JA**
REVIEWED BY:	**ALF**

% GRAVEL	**0.0**	USCS	**ML**
% SAND	**2.4**	MOA FC	**F4**
% SILT/CLAY	**97.6**	% PASS. 0.02 mm	**94.0**
% MOIST. CONTENT	**31.0**	% PASS. 0.002 mm	**52.1**
UNIFORMITY COEFFICIENT (C_u)			**UNKNOWN**
COEFFICIENT OF GRADATION (C_c)			**UNKNOWN**
ASTM D1557 (uncorrected)			**N/A**
ASTM D4718 (corrected)			**N/A**
OPTIMUM MOIST. CONTENT. (corrected)			**N/A**

PARTICLE SIZE ANALYSIS ASTM D422 / C136



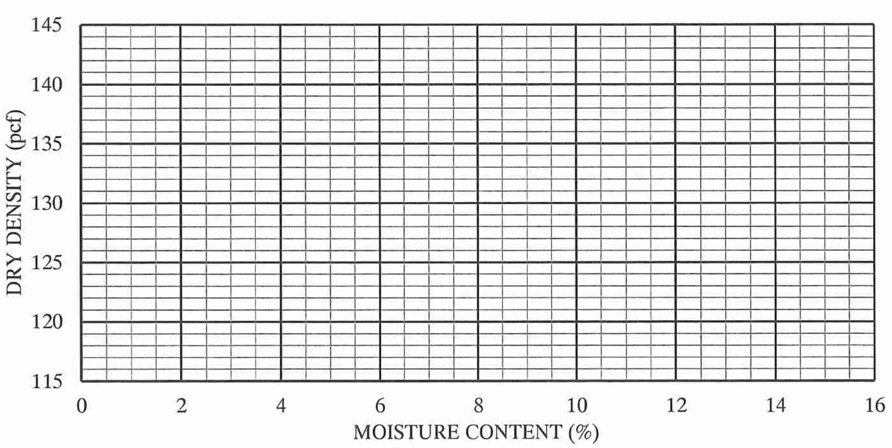
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
76.20	3"		
38.10	1.5"		
19.00	3/4"		
12.70	1/2"		
9.50	3/8"		
4.75	#4	**100**	
2.00	#10	**100**	
0.85	#20	**100**	
0.43	#40	**99**	
0.25	#60	**99**	
0.15	#100	**98**	
0.075	#200	**97.6**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1	0.0344	96.0
2	0.0249	94.5
4	0.0176	93.1
8	0.0127	91.0
15	0.0094	87.4
30	0.0070	81.0
60	0.0050	73.9
250	0.0026	57.6
1440	0.0012	41.8

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

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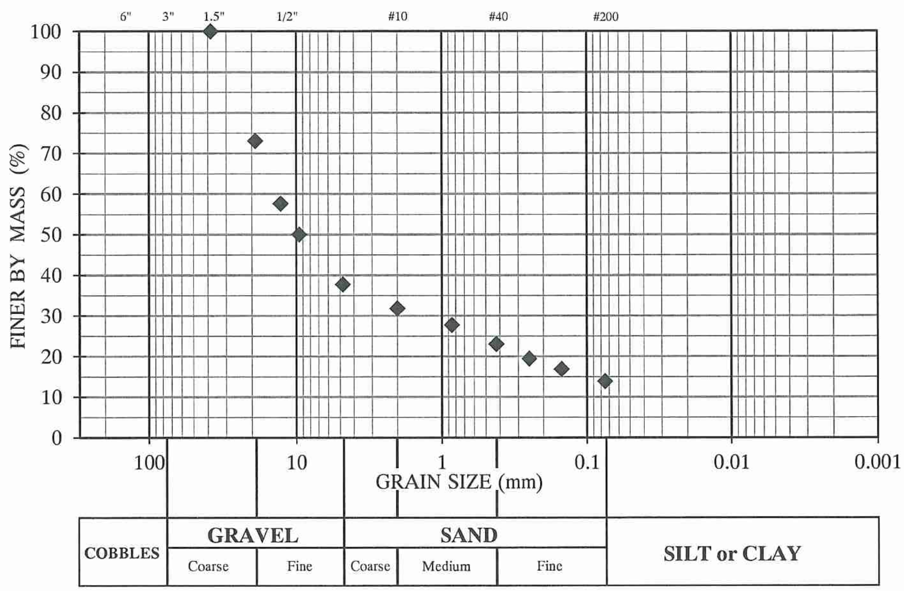
PROJECT CLIENT:	**John McGrew**
PROJECT NAME:	**Ship Creek Condo Development**
PROJECT NO.:	**4385-16**
SAMPLE LOC.:	**B4**
NUMBER/ DEPTH:	**S1 / 0 - 1'**
DESCRIPTION:	**Silty gravel w/ sand**
DATE RECEIVED:	**5/9/2016**
TESTED BY:	**JA**
REVIEWED BY:	**ALF**

% GRAVEL	**62.1**	USCS	**GM**
% SAND	**24.0**	USACOE FC	**N/A**
% SILT/CLAY	**13.9**	% PASS. 0.02 mm	**N/A**
% MOIST. CONTENT	**7.0**	% PASS. 0.002 mm	**N/A**

UNIFORMITY COEFFICIENT (C_u)	**UNKNOWN**
COEFFICIENT OF GRADATION (C_c)	**UNKNOWN**
ASTM D1557 (uncorrected)	**N/A**
ASTM D4718 (corrected)	**N/A**
OPTIMUM MOIST. CONTENT. (corrected)	**N/A**

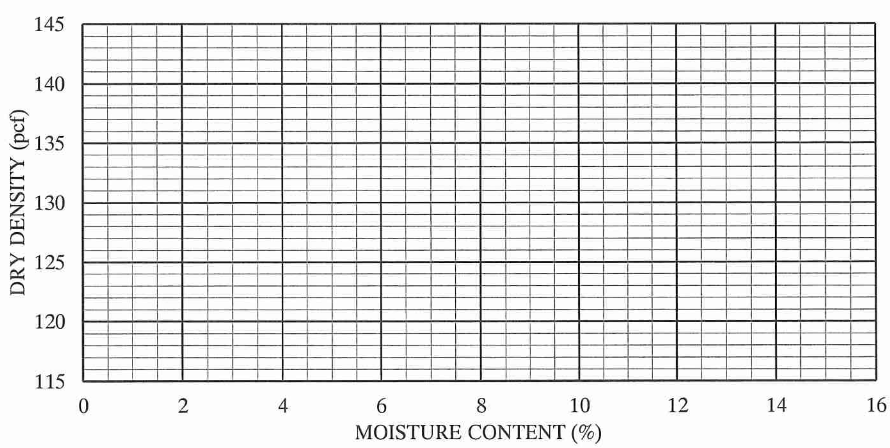
PARTICLE SIZE ANALYSIS ASTM D422 / C136



SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
76.20	3"		
38.10	1.5"	**100**	
19.00	3/4"	**73**	
12.70	1/2"	**58**	
9.50	3/8"	**50**	
4.75	#4	**38**	
2.00	#10	**32**	
0.85	#20	**28**	
0.43	#40	**23**	
0.25	#60	**19**	
0.15	#100	**17**	
0.075	#200	**13.9**	

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1		
2		
4		
8		
15		
30		
60		
250		
1440		

HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

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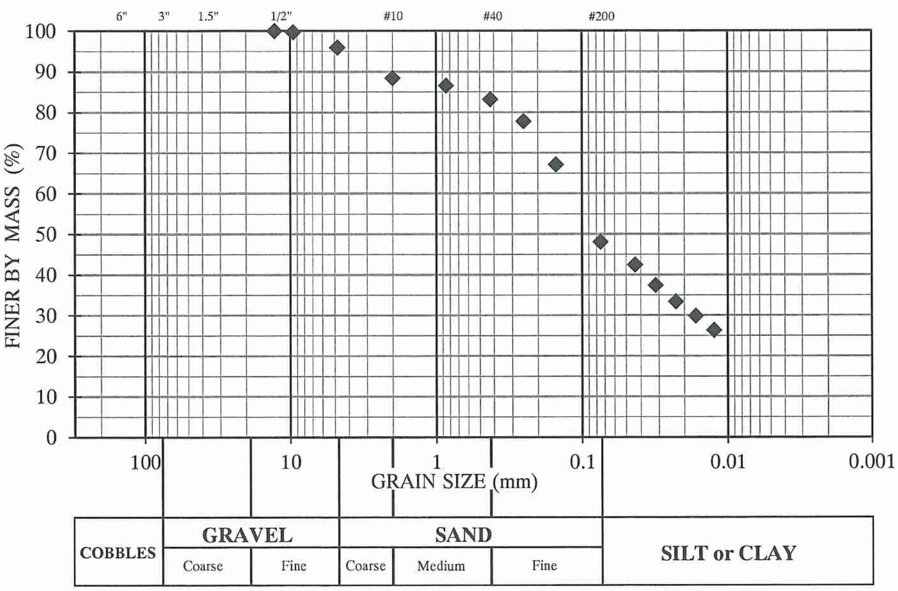
PROJECT CLIENT:	**John McGrew**
PROJECT NAME:	**Ship Creek Condo Development**
PROJECT NO.:	**4385-16**
SAMPLE LOC.:	**B4**
NUMBER/ DEPTH:	**S4 / 7.5 - 9'**
DESCRIPTION:	**Silty sand**
DATE RECEIVED:	**5/9/2016**
TESTED BY:	**JA**
REVIEWED BY:	**ALF**

% GRAVEL	**4.0**	USCS	**SM**
% SAND	**47.8**	MOA FC	**F3**
% SILT/CLAY	**48.2**	% PASS. 0.02 mm	**31.9**
% MOIST. CONTENT	**24.2**	% PASS. 0.002 mm	**N/A**
UNIFORMITY COEFFICIENT (C_u)	**UNKNOWN**		
COEFFICIENT OF GRADATION (C_c)	**UNKNOWN**		
ASTM D1557 (uncorrected)	**N/A**		
ASTM D4718 (corrected)	**N/A**		
OPTIMUM MOIST. CONTENT. (corrected)	**N/A**		

PARTICLE SIZE ANALYSIS ASTM D422 / C136



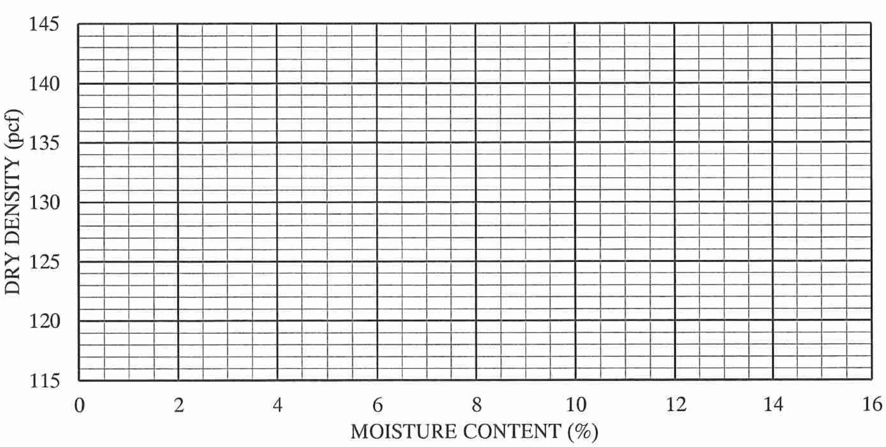
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
76.20	3"		
38.10	1.5"		
19.00	3/4"		
12.70	1/2"	**100**	
9.50	3/8"	**100**	
4.75	#4	**96**	
2.00	#10	**88**	
0.85	#20	**87**	
0.43	#40	**83**	
0.25	#60	**78**	
0.15	#100	**67**	
0.075	#200	**48.2**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1	0.0436	42.5
2	0.0316	37.4
4	0.0229	33.4
8	0.0166	29.9
15	0.0124	26.4
30		
60		
250		
1440		

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDRAULIC COND. (ASTM D2434)	N/A
DEGRADATION (ATM T-313)	N/A
PLASTICITY INDEX ASTM 4318	N/A

The testing services reported herein have been performed to recognized industry standards, unless otherwise noted. No other warranty is made. Should engineering interpretation or opinion be required, NGE-TFT will provide upon written request.

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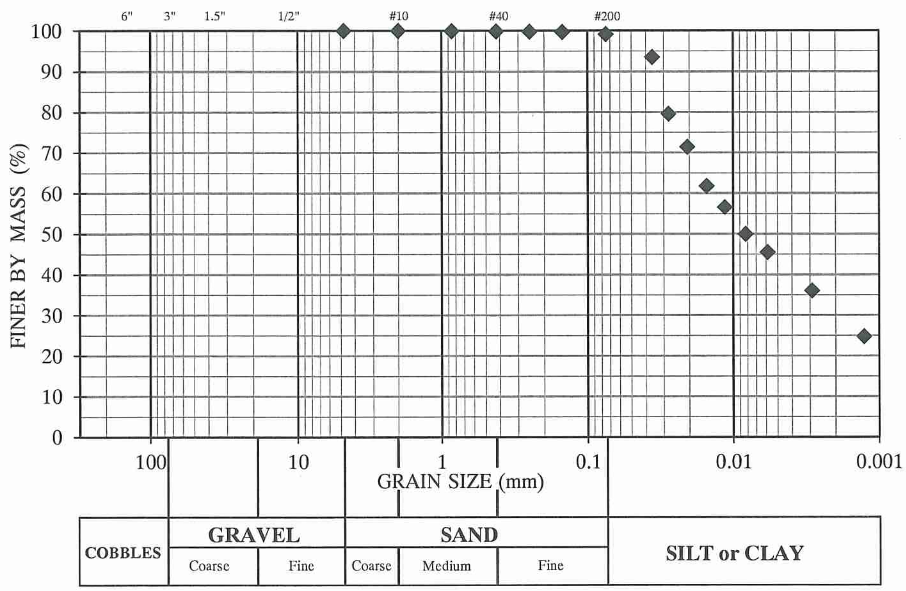
PROJECT CLIENT:	**John McGrew**
PROJECT NAME:	**Ship Creek Condo Development**
PROJECT NO.:	**4385-16**
SAMPLE LOC.:	**B4**
NUMBER/ DEPTH:	**S8 / 25.0'-27.0'**
DESCRIPTION:	**Silt**
DATE RECEIVED:	**6/30/2016**
TESTED BY:	**JA**
REVIEWED BY:	**ALF**

% GRAVEL	**0.0**	USCS	**ML**
% SAND	**0.8**	MOA FC	**F4**
% SILT/CLAY	**99.2**	% PASS. 0.02 mm	**70.5**
% MOIST. CONTENT	**21.9**	% PASS. 0.002 mm	**31.1**
UNIFORMITY COEFFICIENT (C_u)		**UNKNOWN**	
COEFFICIENT OF GRADATION (C_c)		**UNKNOWN**	
ASTM D1557 (uncorrected)		**N/A**	
ASTM D4718 (corrected)		**N/A**	
OPTIMUM MOIST. CONTENT. (corrected)		**N/A**	

PARTICLE SIZE ANALYSIS ASTM D422 / C136



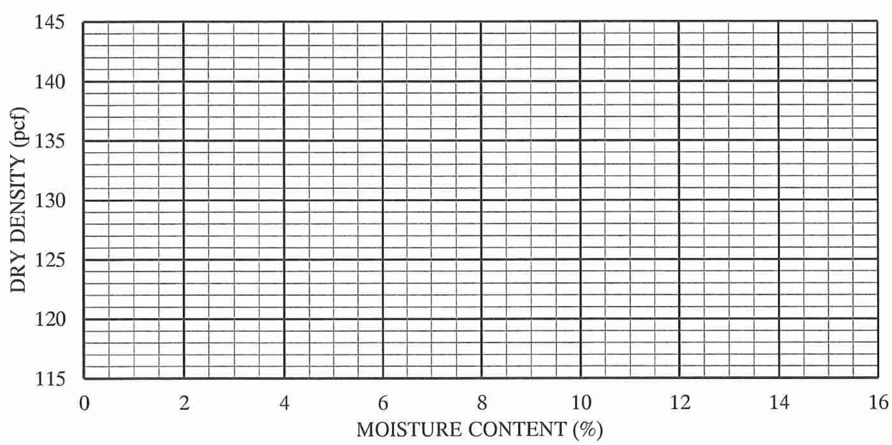
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
76.20	3"		
38.10	1.5"		
19.00	3/4"		
12.70	1/2"		
9.50	3/8"		
4.75	#4	**100**	
2.00	#10	**100**	
0.85	#20	**100**	
0.43	#40	**100**	
0.25	#60	**100**	
0.15	#100	**100**	
0.075	#200	**99.2**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1	0.0360	93.6
2	0.0278	79.6
4	0.0206	71.4
8	0.0152	61.8
15	0.0114	56.7
30	0.0083	50.0
60	0.0058	45.6
250	0.0029	36.1
1440	0.0013	24.8

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

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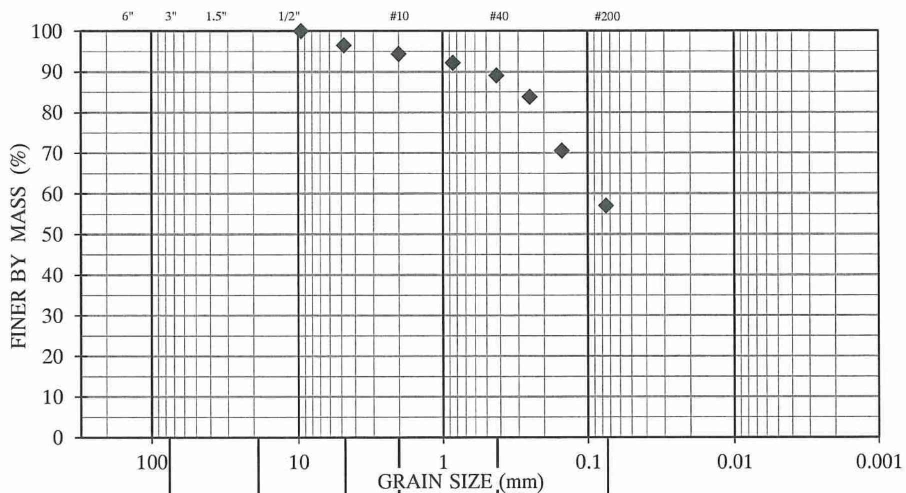
PROJECT CLIENT:	**John McGrew**
PROJECT NAME:	**Ship Creek Condo Development**
PROJECT NO.:	**4385-16**
SAMPLE LOC.:	**B5**
NUMBER/ DEPTH:	**S2 / 2.5 - 4'**
DESCRIPTION:	**Sandy silt**
DATE RECEIVED:	**5/9/2016**
TESTED BY:	**JA**
REVIEWED BY:	**ALF**

% GRAVEL	**3.5**	USCS	**ML**
% SAND	**39.5**	USACOE FC	**N/A**
% SILT/CLAY	**57.0**	% PASS. 0.02 mm	**N/A**
% MOIST. CONTENT	**14.9**	% PASS. 0.002 mm	**N/A**
UNIFORMITY COEFFICIENT (C_u)		**UNKNOWN**	
COEFFICIENT OF GRADATION (C_c)		**UNKNOWN**	
ASTM D1557 (uncorrected)		**N/A**	
ASTM D4718 (corrected)		**N/A**	
OPTIMUM MOIST. CONTENT. (corrected)		**N/A**	

PARTICLE SIZE ANALYSIS ASTM D422 / C136



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

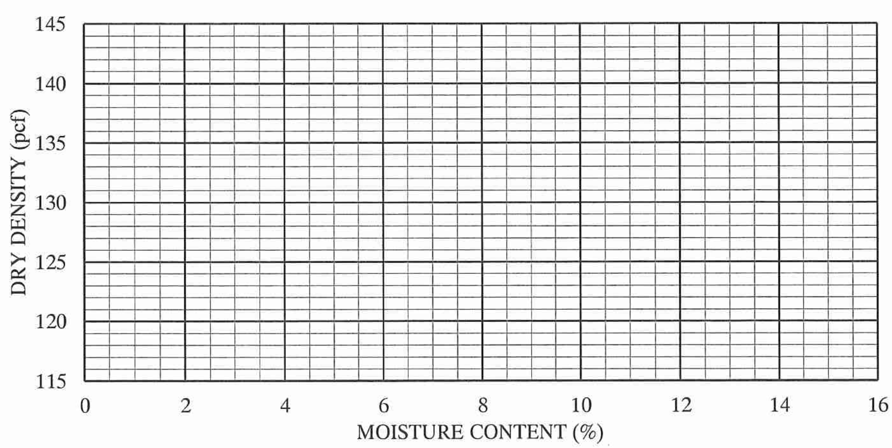
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
76.20	3"		
38.10	1.5"		
19.00	3/4"		
12.70	1/2"		
9.50	3/8"	**100**	
4.75	#4	**97**	
2.00	#10	**94**	
0.85	#20	**92**	
0.43	#40	**89**	
0.25	#60	**84**	
0.15	#100	**71**	
0.075	#200	**57.1**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1		
2		
4		
8		
15		
30		
60		
250		
1440		

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

The testing services reported herein have been performed to recognized industry standards, unless otherwise noted. No other warranty is made. Should engineering interpretation or opinion be required, NGE-TFT will provide upon written request.

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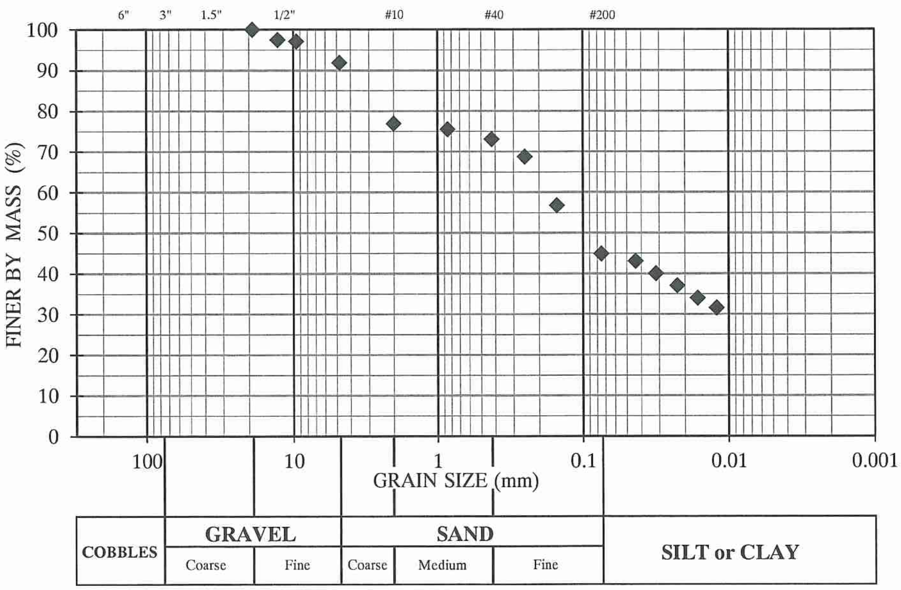
PROJECT CLIENT:	**John McGrew**
PROJECT NAME:	**Ship Creek Condo Development**
PROJECT NO.:	**4385-16**
SAMPLE LOC.:	**B5**
NUMBER/ DEPTH:	**S3 / 5 - 6.5'**
DESCRIPTION:	**Silty sand**
DATE RECEIVED:	**5/9/2016**
TESTED BY:	**JA**
REVIEWED BY:	**ALF**

% GRAVEL	**8.1**	USCS	**SM**
% SAND	**47.0**	MOA FC	**F3**
% SILT/CLAY	**44.9**	% PASS. 0.02 mm	**35.9**
% MOIST. CONTENT	**43.8**	% PASS. 0.002 mm	**N/A**
UNIFORMITY COEFFICIENT (C_u)		**UNKNOWN**	
COEFFICIENT OF GRADATION (C_c)		**UNKNOWN**	
ASTM D1557 (uncorrected)		**N/A**	
ASTM D4718 (corrected)		**N/A**	
OPTIMUM MOIST. CONTENT. (corrected)		**N/A**	

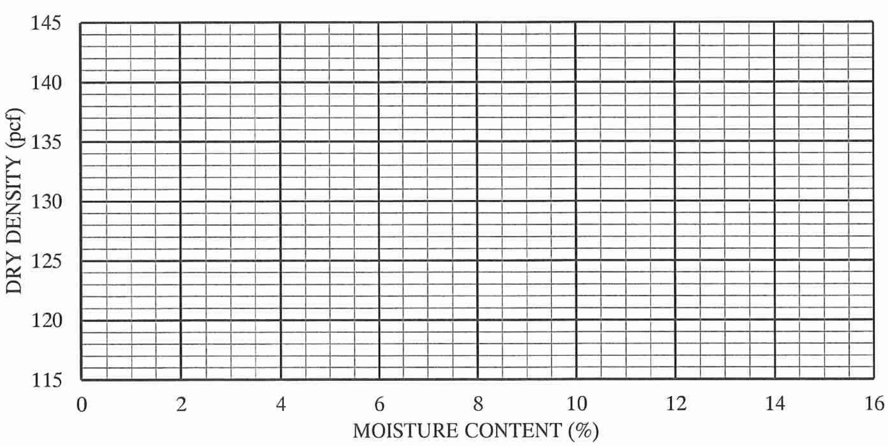
PARTICLE SIZE ANALYSIS ASTM D422 / C136



SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
76.20	3"		
38.10	1.5"		
19.00	3/4"	**100**	
12.70	1/2"	**98**	
9.50	3/8"	**97**	
4.75	#4	**92**	
2.00	#10	**77**	
0.85	#20	**75**	
0.43	#40	**73**	
0.25	#60	**69**	
0.15	#100	**57**	
0.075	#200	**44.9**	

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1	0.0434	43.1
2	0.0317	40.1
4	0.0226	37.1
8	0.0164	34.1
15	0.0121	31.6
30		
60		
250		
1440		

HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

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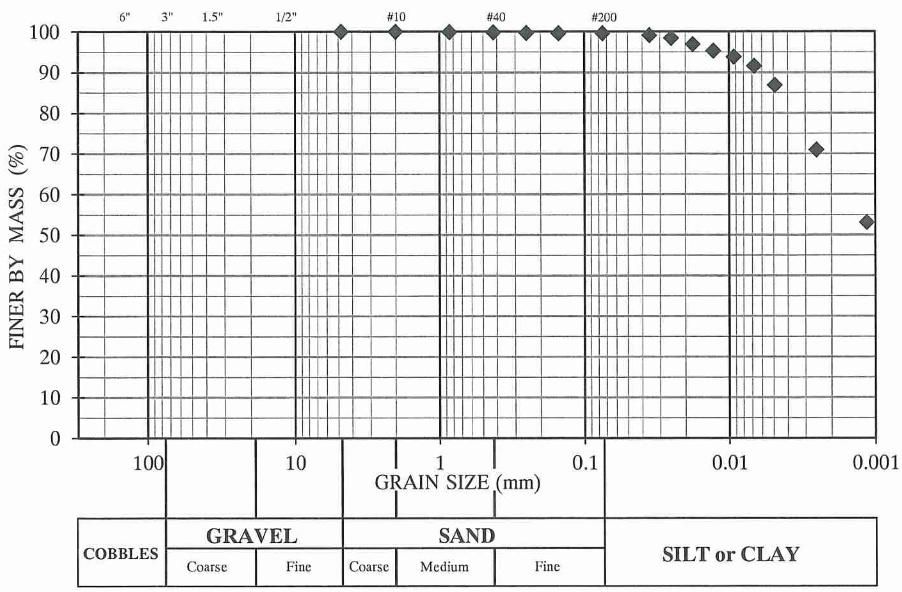
PROJECT CLIENT:	**John McGrew**
PROJECT NAME:	**Ship Creek Condo Development**
PROJECT NO.:	**4385-16**
SAMPLE LOC.:	**B5**
NUMBER/ DEPTH:	**S9 / 30.0'-32.0'**
DESCRIPTION:	**Silt**
DATE RECEIVED:	**6/30/2016**
TESTED BY:	**JA**
REVIEWED BY:	**ALF**

% GRAVEL	**0.0**	USCS	**ML**
% SAND	**0.4**	MOA FC	**F4**
% SILT/CLAY	**99.6**	% PASS. 0.02 mm	**97.5**
% MOIST. CONTENT	**33.4**	% PASS. 0.002 mm	**65.7**
UNIFORMITY COEFFICIENT (C_u)		**UNKNOWN**	
COEFFICIENT OF GRADATION (C_c)		**UNKNOWN**	
ASTM D1557 (uncorrected)		**N/A**	
ASTM D4718 (corrected)		**N/A**	
OPTIMUM MOIST. CONTENT. (corrected)		**N/A**	

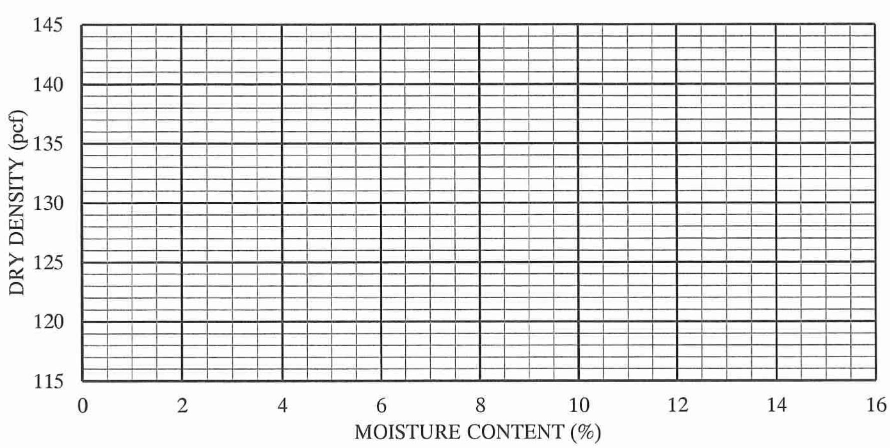
PARTICLE SIZE ANALYSIS ASTM D422 / C136



SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
76.20	3"		
38.10	1.5"		
19.00	3/4"		
12.70	1/2"		
9.50	3/8"		
4.75	#4	**100**	
2.00	#10	**100**	
0.85	#20	**100**	
0.43	#40	**100**	
0.25	#60	**100**	
0.15	#100	**100**	
0.075	#200	**99.6**	

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
0.5		
1	0.0356	99.2
2	0.0252	98.5
4	0.0178	96.9
8	0.0128	95.4
15	0.0093	93.9
30	0.0067	91.6
60	0.0049	86.8
250	0.0025	71.0
1440	0.0011	53.2

HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

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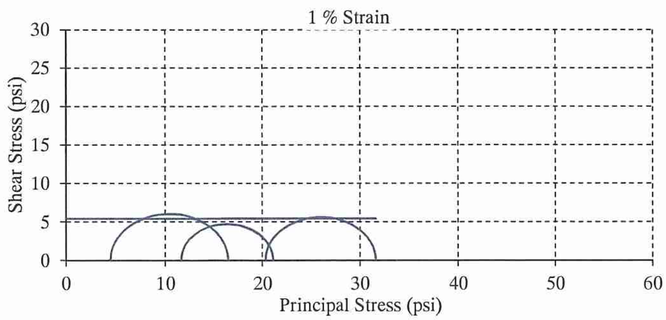
ASTM D4767: Standard Test Method For Consolidated Undrained Triaxial Compression Test

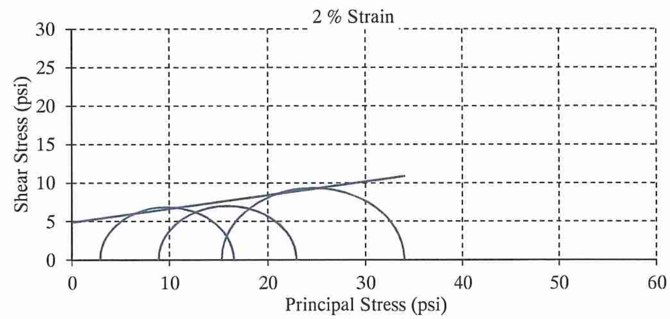
PROJECT NO:	**4385-16**	SAMPLE A:	**B1-S13**
PROJECT NAME:	**Ship Creek Condo Development**	SAMPLE B:	**B2-S12**
TESTED BY:	**AF**	SAMPLE C:	**B2-S13**
CHECKED BY:	**AF**	DATE RECEIVED:	**5/12/16**

SAMPLE DESCRIP.:	**[ML / CL]**	STRAIN RATE:	**7.1 % min^{-1}**



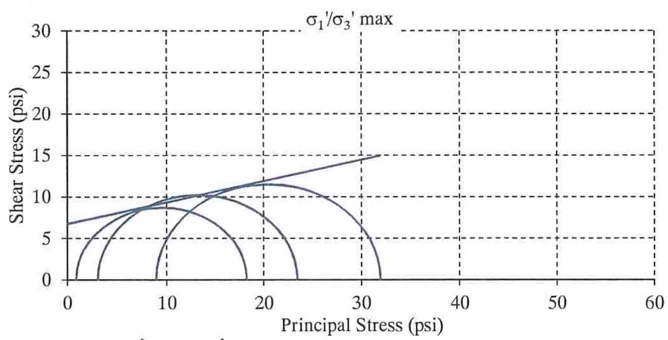
	σ_1' (psi)	σ_3' (psi)	ε (%)
A	21.1	11.6	1.0
B	16.5	4.4	1.0
C	31.6	20.3	1.0
D			
E			

Friction Angle = **0°**
Cohesion = **5 psi**
R^2= **0.03**



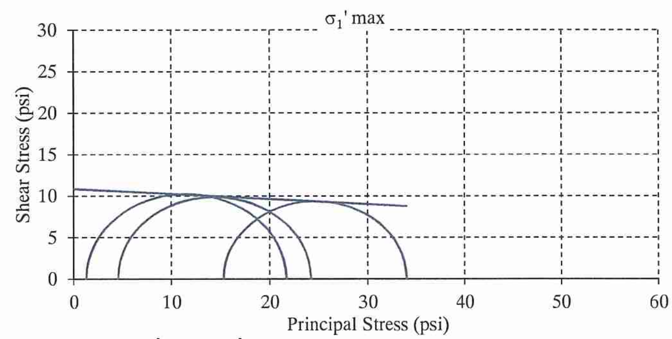
	σ_1' (psi)	σ_3' (psi)	ε (%)
A	22.9	8.9	2.0
B	16.6	2.9	2.0
C	34.0	15.3	2.0
D			
E			

Friction Angle = **10°**
Cohesion = **5 psi**
R^2= **0.86**



	σ_1' (psi)	σ_3' (psi)	ε (%)
A	23.4	3.0	7.6
B	18.2	0.9	6.9
C	31.9	8.9	8.7
D			
E			

Friction Angle = **14°**
Cohesion = **7 psi**
R^2= **0.94**



	σ_1' (psi)	σ_3' (psi)	ε (%)
A	24.2	4.5	5.4
B	21.7	1.3	14.4
C	34.0	15.3	2.2
D			
E			

Friction Angle = **-4°**
Cohesion = **11 psi**
R^2= **0.95**

COMMENTS

NORTHERN GEOTECHNICAL ENGINEERING, INC. / TERRA FIRMA TESTING

Laboratory Testing | Geotechnical Engineering | Instrumentation | Construction Monitoring Services | Thermal Analysis

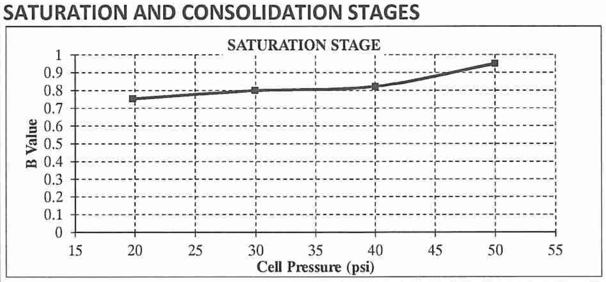
ASTM D4767: STANDARD TEST METHOD FOR UNCONSOLIDATED UNDRAINED TRIAXIAL TEST

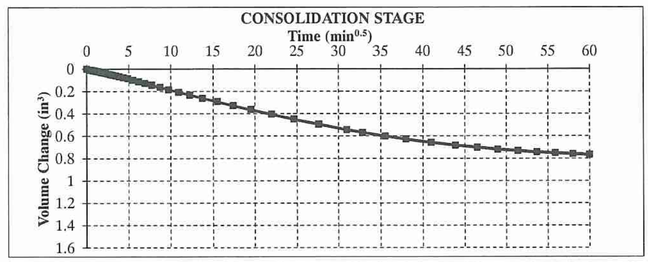
PROJECT NO:	4385-16	BOREHOLE NO.:	B1
PROJECT NAME:	Ship Creek Condo Development	SAMPLE NO.:	S13
CLIENT	John McGrew	SAMPLE DEPTH:	50.3'-50.8'
TESTED & CHECKED BY:	AF	DATE RECEIVED:	5/12/2016

UNITED SOIL CLASS.:	[ML] SILT	STRAIN RATE	7.1 [% min^{-1}]

SAMPLE TYPE:	SHELBY TUBE	PARTICLE SPEC. GRAVITY:	2.69
MOUNTING METHOD:	WET	INITIAL MOIST. CONT.:	34.5 [%]
VARIATIONS FROM PROC.:	N/A	BULK DENSITY:	122.1 [lbf ft^{-3}]
LABORATORY TEMP.:	65 °F	DRY UNIT WEIGHT:	90.8 [lbf ft^{-3}]
INITIAL HEIGHT:	5.6 [inches]	INITIAL VOIDS RATIO:	N/A
INITIAL DIAMETER:	2.8 [inches]	INITIAL DEG. OF SAT.:	N/A [%]

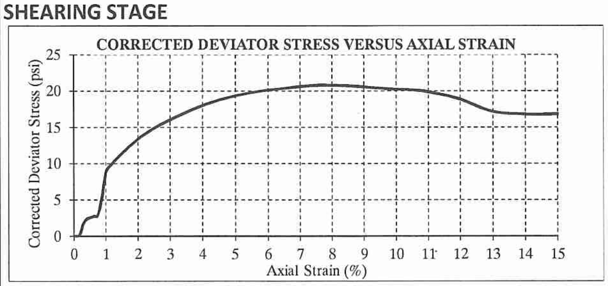
SATURATION AND CONSOLIDATION STAGES

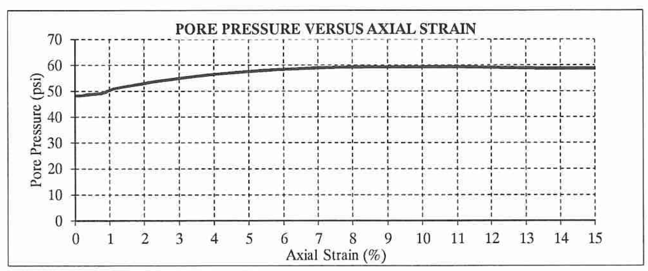


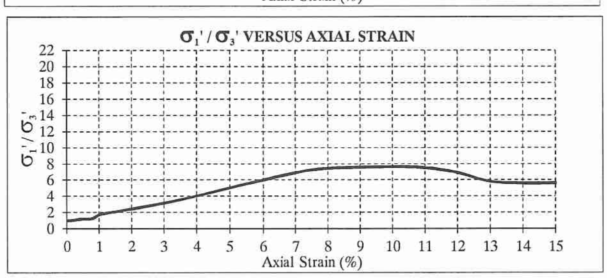


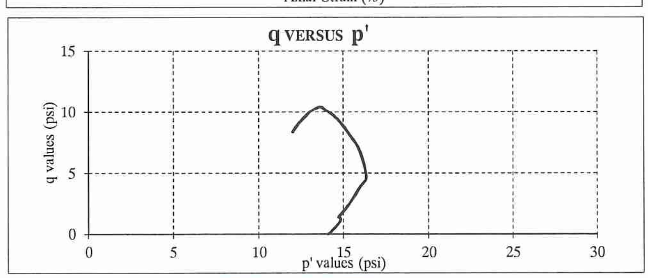
FINAL B VALUE:	0.95	CELL PRESSURE:	63.0 [psi]
FINAL BACK PRESSURE:	47.8 [psi]	TIME TO 50% PRIM. CONS.:	N/A [min]
		DRY UNIT WEIGHT:	N/A [lbf ft^{-3}]
		VOID RATIO:	N/A
		SATURATION:	N/A [%]
		MOISTURE CONTENT:	33.3 [%]

SHEARING STAGE









CELL PRESSURE:	62.4	[psi]
INITIAL PORE PRESSURE:	48.3	[psi]
EFF. STRESS AT START:	14.1	[psi]

NORTHERN GEOTECHNICAL ENGINEERING, INC. / TERRA FIRMA TESTING

Laboratory Testing | Geotechnical Engineering | Instrumentation | Construction Monitoring Services | Thermal Analysis

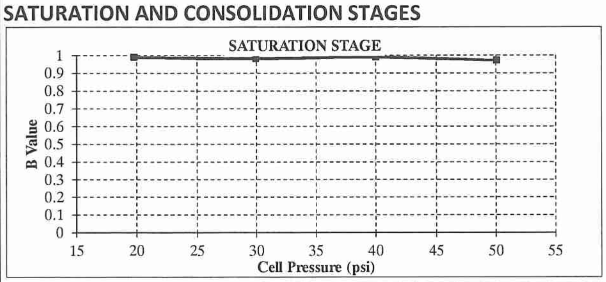
ASTM D4767: STANDARD TEST METHOD FOR UNCONSOLIDATED UNDRAINED TRIAXIAL TEST

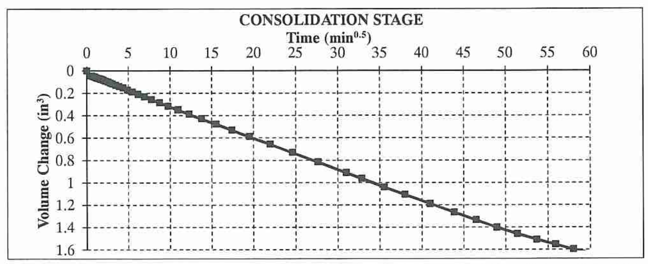
PROJECT NO:	4385-16	BOREHOLE NO.:	B2
PROJECT NAME:	Ship Creek Condo Development	SAMPLE NO.:	S12
CLIENT	John McGrew	SAMPLE DEPTH:	45.0'-45.5'
TESTED & CHECKED BY:	AF	DATE RECEIVED:	5/12/2016

UNITED SOIL CLASS.:	[ML] SILT	STRAIN RATE	7.1 [% min^{-1}]

SAMPLE TYPE:	SHELBY TUBE	PARTICLE SPEC. GRAVITY:	2.68
MOUNTING METHOD:	WET	INITIAL MOIST. CONT.:	28.6 [%]
VARIATIONS FROM PROC.:	N/A	BULK DENSITY:	122.9 [lbf ft^{-3}]
LABORATORY TEMP.:	65 °F	DRY UNIT WEIGHT:	95.6 [lbf ft^{-3}]
INITIAL HEIGHT:	5.6 [inches]	INITIAL VOIDS RATIO:	N/A
INITIAL DIAMETER:	2.8 [inches]	INITIAL DEG. OF SAT.:	N/A [%]

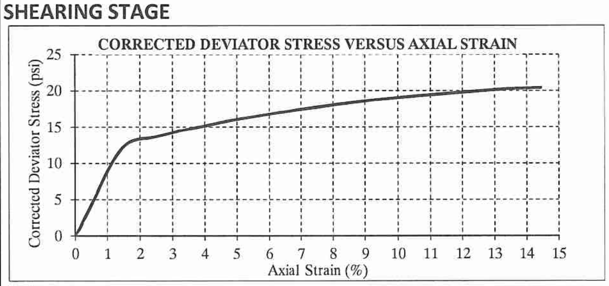
SATURATION AND CONSOLIDATION STAGES

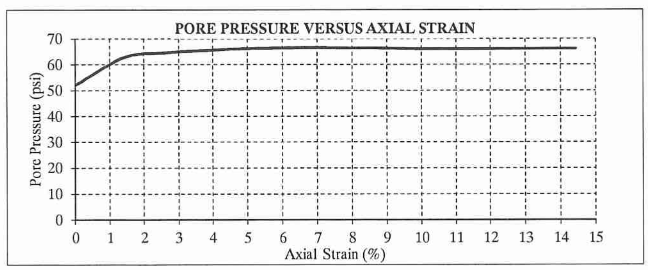


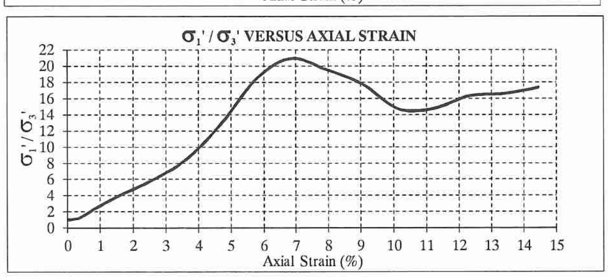


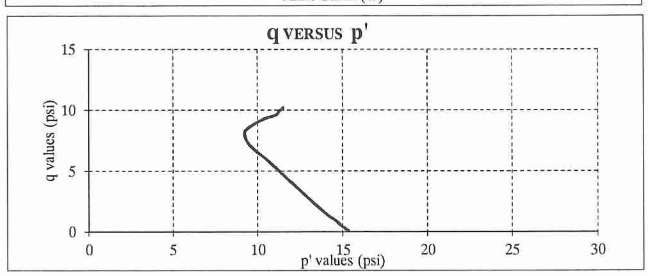
FINAL B VALUE:	0.97	CELL PRESSURE:	68.1 [psi]
FINAL BACK PRESSURE:	47.7 [psi]	TIME TO 50% PRIM. CONS.:	N/A [min]
		DRY UNIT WEIGHT:	N/A [lbf ft^{-3}]
		VOID RATIO:	N/A
		SATURATION:	N/A [%]
		MOISTURE CONTENT:	24.7 [%]

SHEARING STAGE









CELL PRESSURE:	62.4	[psi]
INITIAL PORE PRESSURE:	48.3	[psi]
EFF. STRESS AT START:	14.1	[psi]

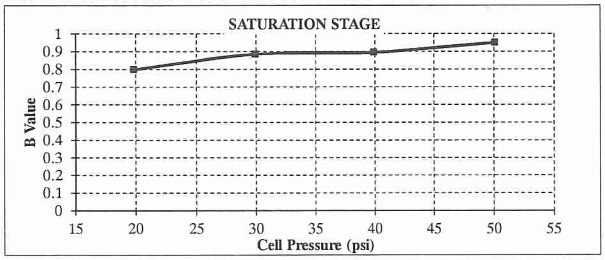
NORTHERN GEOTECHNICAL ENGINEERING, INC. / TERRA FIRMA TESTING

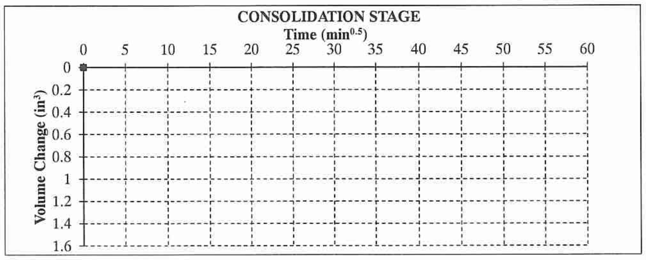
Laboratory Testing | Geotechnical Engineering | Instrumentation | Construction Monitoring Services | Thermal Analysis

ASTM D4767: STANDARD TEST METHOD FOR UNCONSOLIDATED UNDRAINED TRIAXIAL TEST

PROJECT NO:	4385-16	BOREHOLE NO.:	B2
PROJECT NAME:	Ship Creek Condo Development	SAMPLE NO.:	S13
CLIENT	John McGrew	SAMPLE DEPTH:	51.0'-51.5'
TESTED & CHECKED BY:	AF	DATE RECEIVED:	5/12/2016
UNITED SOIL CLASS.:	[CL] LEAN CLAY	STRAIN RATE	7.1 [% min^{-1}]
SAMPLE TYPE:	SHELBY TUBE	PARTICLE SPEC. GRAVITY:	2.63
MOUNTING METHOD:	WET	INITIAL MOIST. CONT.:	35.6 [%]
VARIATIONS FROM PROC.:	N/A	BULK DENSITY:	120.9 [lbf ft^{-3}]
LABORATORY TEMP.:	65 °F	DRY UNIT WEIGHT:	89.2 [lbf ft^{-3}]
INITIAL HEIGHT:	5.6 [inches]	INITIAL VOIDS RATIO:	N/A
INITIAL DIAMETER:	2.8 [inches]	INITIAL DEG. OF SAT.:	N/A [%]

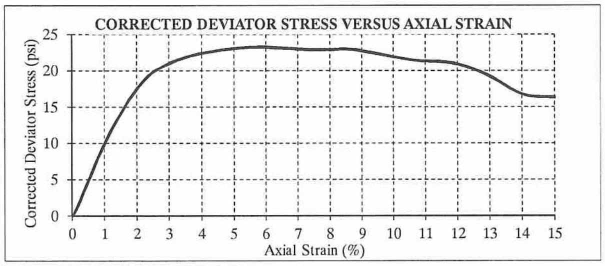
SATURATION AND CONSOLIDATION STAGES

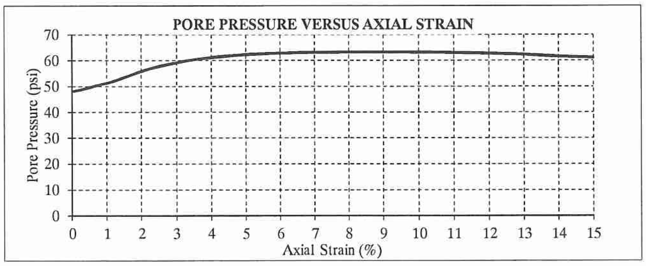


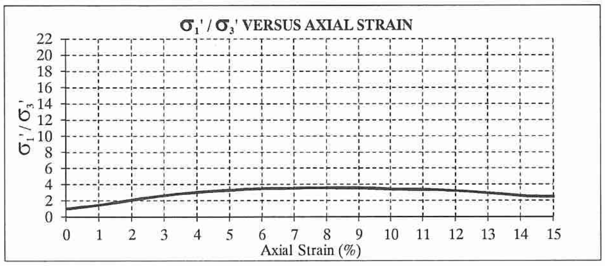


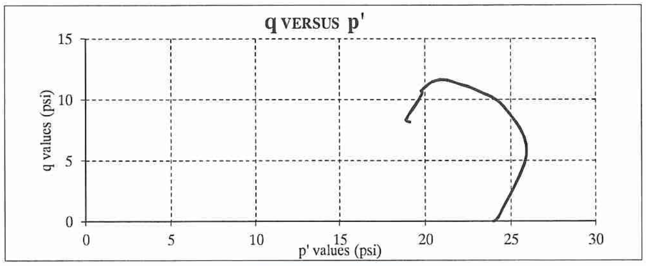
FINAL B VALUE:	0.95	CELL PRESSURE:	72.8 [psi]
FINAL BACK PRESSURE:	47.6 [psi]	TIME TO 50% PRIM. CONS.:	N/A [min]
		DRY UNIT WEIGHT:	N/A [lbf ft^{-3}]
		VOID RATIO:	N/A
		SATURATION:	N/A [%]
		MOISTURE CONTENT:	33.8 [%]

SHEARING STAGE









CELL PRESSURE:	72.2 [psi]
INITIAL PORE PRESSURE:	48.2 [psi]
EFF. STRESS AT START:	24.0 [psi]

APPENDIX D

USGS SEISMIC DESIGN MAPS SUMMARY REPORT

USGS Design Maps Summary Report

User–Specified Input

Report Title 4385-16 Ship Creek Development - Site Class D
Thu June 9, 2016 20:57:54 UTC

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

Site Coordinates 61.22055°N, 149.89504°W

Site Soil Classification Site Class D – "Stiff Soil"

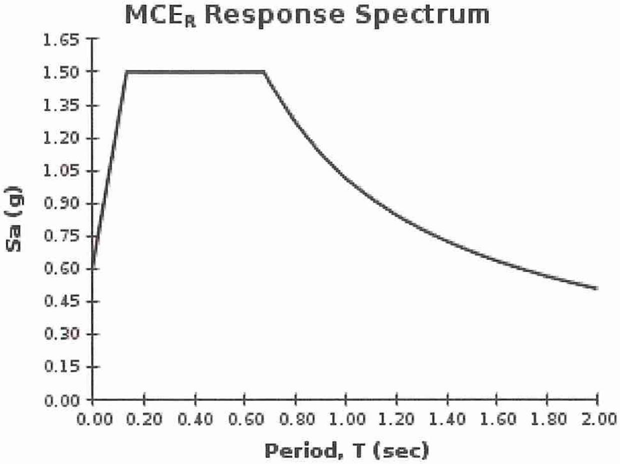
Risk Category I/II/III

USGS–Provided Output

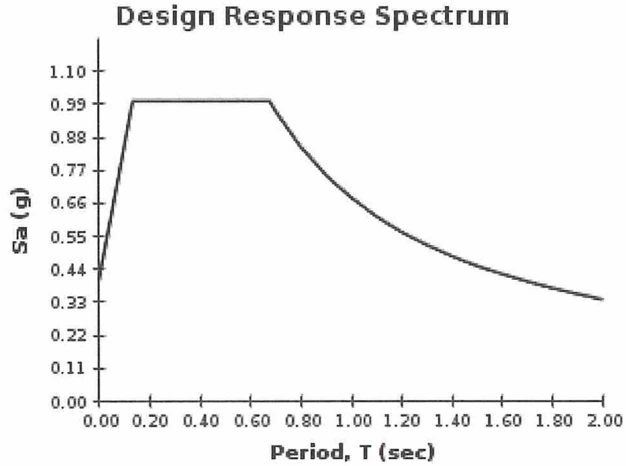
$S_S =$	1.500 g	$S_{MS} =$	1.500 g	$S_{DS} =$	1.000 g
$S_1 =$	0.676 g	$S_{M1} =$	1.014 g	$S_{D1} =$	0.676 g

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



Design Response Spectrum



USGS Design Maps Detailed Report

2012 International Building Code (61.22055°N, 149.89504°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 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(4) [1] S_S = 1.500 g

From Figure 1613.3.1(5) [2] S_1 = 0.676 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 $w \geq$ 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				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
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	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 = 1.500 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				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
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	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 = 0.676 g, F_v = 1.500

Equation (16-37): $S_{MS} = F_a S_S = 1.000 \times 1.500 = 1.500$ g

---

Equation (16-38): $S_{M1} = F_v S_1 = 1.500 \times 0.676 = 1.014$ g

---

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39): $S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 1.500 = 1.000$ g

---

Equation (16-40): $S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 1.014 = 0.676$ 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} = 1.000 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} = 0.676 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(4)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(4).pdf
2. *Figure 1613.3.1(5)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(5).pdf

USGS Design Maps Summary Report

User–Specified Input

Report Title 4385-16 Ship Creek Development - Site Class E
Thu June 9, 2016 20:58:57 UTC

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

Site Coordinates 61.22055°N, 149.89504°W

Site Soil Classification Site Class E – "Soft Clay Soil"

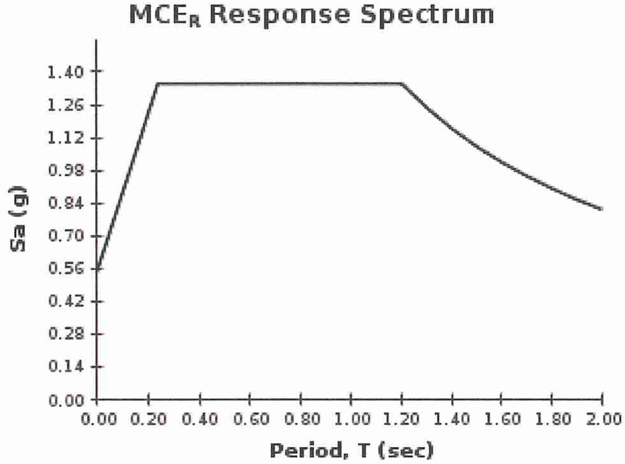
Risk Category I/II/III

USGS–Provided Output

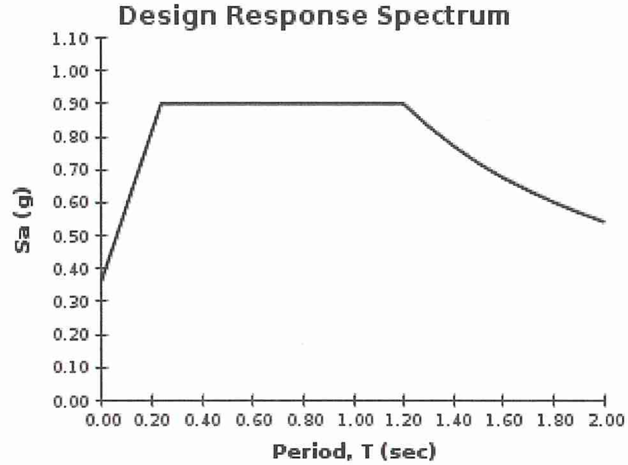
S_S =	1.500 g	S_{MS} =	1.350 g	S_{DS} =	0.900 g
S_1 =	0.676 g	S_{M1} =	1.623 g	S_{D1} =	1.082 g

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



Design Response Spectrum



Design Maps Detailed Report

2012 International Building Code (61.22055°N, 149.89504°W)

Site Class E – "Soft Clay 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 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(4) [1]	S_S = 1.500 g
From Figure 1613.3.1(5) [2]	S_1 = 0.676 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 E, 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				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
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	1.0
E	2.5	1.7	1.2	0.9	$\boxed{0.9}$
F	See Section 11.4.7 of ASCE 7				

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

For Site Class = E and S_S = 1.500 g, F_a = 0.900

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_v

Site Class	Mapped Spectral Response Acceleration at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
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	1.5
E	3.5	3.2	2.8	2.4	$\boxed{2.4}$
F	See Section 11.4.7 of ASCE 7				

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

For Site Class = E and S_1 = 0.676 g, F_v = 2.400

Equation (16-37): $S_{MS} = F_aS_S = 0.900 \times 1.500 = 1.350$ g

Equation (16-38): $S_{M1} = F_vS_1 = 2.400 \times 0.676 = 1.623$ g

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39): $S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 1.350 = 0.900$ g

Equation (16-40): $S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 1.623 = 1.082$ 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} = 0.900 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} = 1.082 g, Seismic Design Category = D

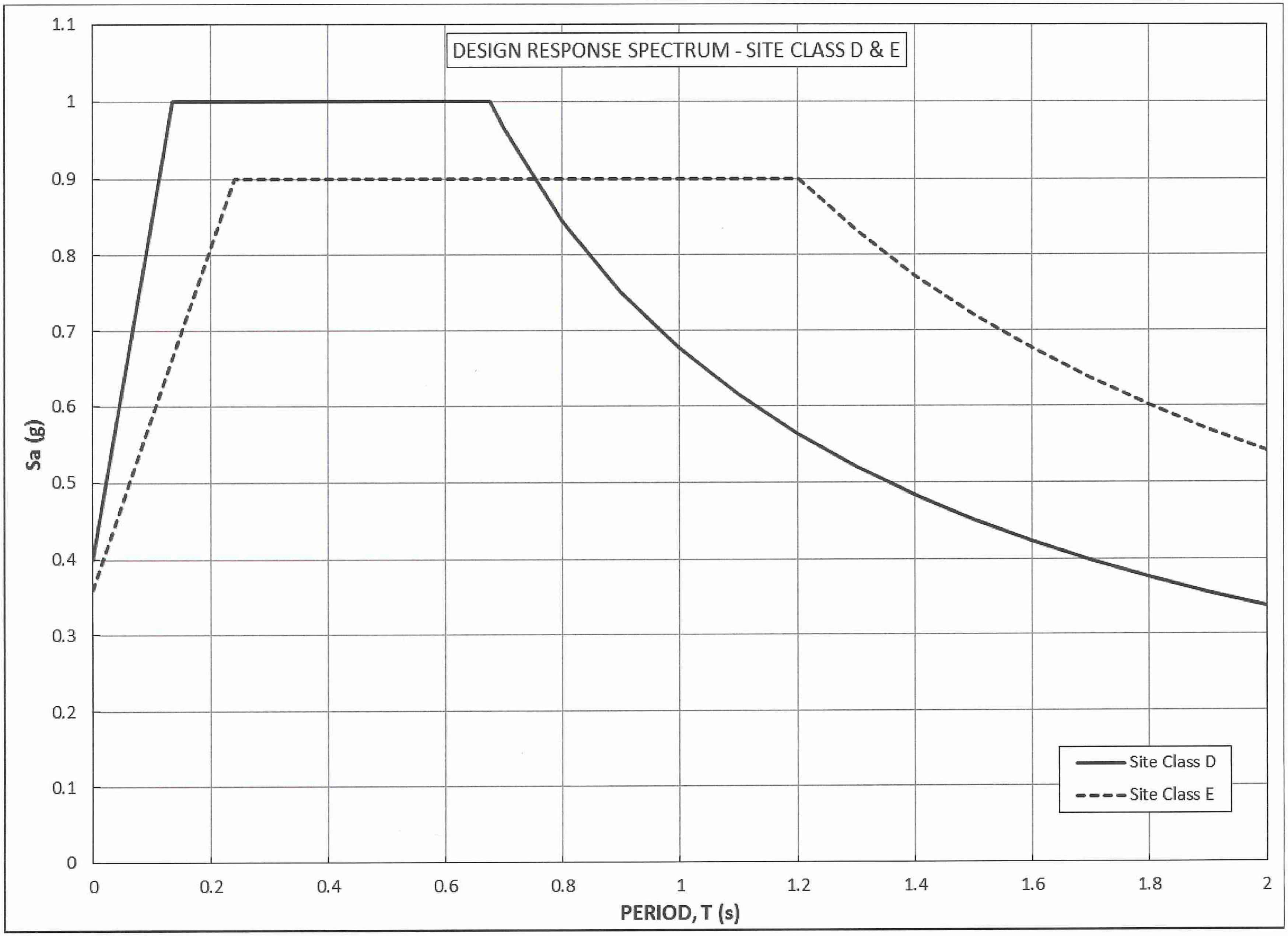
Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

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(4)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(4).pdf
2. *Figure 1613.3.1(5)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(5).pdf



NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

APPENDIX TITLE: DESIGN RESPONSE SPECTRUM—SITE CLASS D & E	
PROJECT NAME: SHIP CREEK DEVELOPMENT	PROJECT ID: 4385-16
PROJECT LOCATION: ANCHORAGE, AK	APPENDIX NUMBER: D-1

APPENDIX E

LATERAL SEISMIC SOIL PRESSURE: AN UPDATED APPROACH

Calculations

Wall to be evaluated: 9 ft high wall retains 9 ft earth, and restrained by concrete slab at top

Footing width minimum 16 in, maximum 4.5 ft

To reduce seismic loading, footing should be as small as possible

Slab at base not rigidly connected

Use 0.20 g to match the MOA code amendments for Seismic Hazard Zone 5

Equation reference to "Lateral Seismic Soil Pressure" in Appendix E

Frequency (Equation 1, Page 2-9)

$f = V_s/(4 * H)$

V_s estimated for near surface = 500 ft/sec

For 9 ft wall (9 ft soil retained): $f = 500/(4 * 9)$ = 13.9 Hz > 0.1 Hz

Soil density $\rho = 125/32.2$ = 3.88 slugs/ft^3

Equation 2, page 2-11 $\psi_v = 2/\sqrt{(1-\nu)(2-\nu)}$ = 1.90

ν = Poisson's Ratio = ⅓

Embedment Ratio 13 ft wall (12.5 ft soil retained) => 1.33/9 = 0.15 < 0.5

Use α = 0.27, maximum footing width: 9 ft wall (9 ft soil retained) = 4.5 ft

General procedure for calculations on page 2-15 (Appendix E)

$F = \alpha * \rho * H^2 * S_A * \psi_v$ (Equation 5, page 2-15)

For 9 ft wall (9 ft soil retained):

$F = (0.27)(3.88)(9^2)(0.20)(32.2)(1.9)$= 1038 lb/lineal ft of wall

13 ft wall Area under curve = 0.632(9) = 5.7

Area also = 1038 lb/lineal ft of wall

Factor for distribution curve = 1038/5.7 = 182

The pressure distribution is presented on Figure 13 of this report

LATERAL SEISMIC SOIL PRESSURE

AN UPDATED APPROACH

By

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Menlo Park, California

INTRODUCTION

The effect of ground motion on retaining walls was recognized by Okabe (1924) and Mononobe and Matsuo (1929) following the great Kanto Earthquake of 1923 in Japan. The method proposed by Mononobe and Okabe, currently known as the M-O method, was based on the Coulomb's theory of static soil pressure developed more than 200 years ago. In the last 30 years, a great deal of research work both in the analytical and in experimental areas has been performed to evaluate the adequacy of the M-O method or to extend the method for specific applications. Discussion of the all the research work on the seismic soil pressure is extensive and is beyond the scope of this study. Rather, only the milestones that have influenced the design practice are described below.

Seed and Whitman (1970)

In 1970, the M-O method and the associated analytical relationships were simplified by Seed and Whitman (1970) for design of earth retaining structures for dynamic loads. Using the charts, the designer only needs to know the basic properties of the backfill (the angle of internal friction) and the peak ground acceleration to obtain the seismic soil pressure. As suggested by Seed and Whitman, the basic assumptions used in the development of the M-O method should always be considered in design applications. These assumptions are:

- The backfill materials are dry cohesionless materials.
- The retaining wall yields equally and sufficiently to produce minimum active soil pressure.
- The active soil pressure is associated with a soil wedge behind the wall which is at the point of incipient failure and the maximum shear strength is mobilized along the potential sliding surface.
- The soil behind the wall behaves as a rigid body and the acceleration is uniform in the soil wedge behind the wall.

Whitman et al. (1979, 1990, 1991)

The effect of some of the limiting assumptions used in the M-O method above has been investigated by, among others, Whitman et al. (1979, 1990, 1991) and Nadim and Whitman (1984). The non-yielding wall conditions and the amplifications of the motion in the soil mass were found to be significant in some cases. However, no practical tools were proposed for design applications to circumvent the limiting assumptions used in the M-O method. Judging from the results of model tests by several researchers, Whitman (1990) found that use of the M-O method for design of relatively simple gravity walls up to 30 ft high is acceptable. However, for higher walls and non-yielding walls, he recommended a more careful analysis be performed.

Richards and Elms (1979)

One of the more important recent developments in characterizing the seismic soil pressure for retaining walls was the work performed by Richards and Elms (1979). Using the M-O method and the Newmark's sliding-block analogy, the authors proposed a displacement-controlled method which incorporates basic ground motion parameters (maximum acceleration and maximum velocity) and reduces the seismic soil pressure based on the acceptable amount of the wall movement. In practice, the method is currently used for designing walls for which limiting horizontal displacements are of no concern.

Wood (1973)

While the M-O method was developed for yielding walls, Wood (1973) developed an equivalent static elastic solution for seismic soil pressure for non-yielding walls. The solution is based on finite element analysis of a soil-wall system for a wall resting on a rigid base and a uniform soil layer behind the wall. In general, Wood's solution amounts to a lateral force that acts about 0.63 times the height of the wall above the base of the wall which corresponds approximately to a parabolic distribution of soil pressure unlike M-O's inverted triangular distribution. Wood's solution predicts seismic soil pressure larger (by a factor of 2 to 3) than the pressure predicted by the M-O method. The elastic solution proposed by Wood has been adopted by ASCE Standards for Nuclear Structures (1986) and has been used in many applications. Wood's solution requires knowledge of the maximum ground acceleration along with the density and Poisson's ratio of the soil to obtain the seismic soil pressure behind the wall.

Matsuzawa et al. (1984), Ishibashi et al. (1985)

To address saturated backfill conditions and to include the hydrodynamic forces, the M-O method was extended by Matsuzawa et al. (1984) and Ishibashi et al. (1985). A comprehensive summary of the all the M-O based methods and their applications to various retaining wall conditions are documented in a recent US Army publication (Ebeling and Morrison, 1992).

Veletsos et al. (1994a, 1994b)

More recently, Veletsos and Younan (1994a, 1994b) developed an analytical model to compute seismic soil pressure for rigid vertical walls resting on a rigid base. The proposed model is based on the series of elastically supported semiinfinte horizontal bars with distributed mass to model the soil medium behind the wall. The model was developed for vertically propagating shear waves with the assumption that horizontal variation of vertical displacements in the soil medium is negligible. In this model, contrary to Wood's equivalent static solution, amplification of motion in the soil medium behind the wall is considered. The model highlights the effects of several parameters including the frequency of vibration on the seismic soil pressure magnitude and distribution. The model was subsequently expanded for application to cylindrical vaults and storage buildings (Veletsos and Younan, 1994c; 1995).

Significance of Seismic Soil Pressure in Design

Seed and Whitman (1970) summarized damage to wall structures during earthquakes. Damage to retaining walls with saturated backfills is typically more dramatic and is frequently reported in the literature. However, damage reports of walls above the water table are not uncommon. A number of soil retaining structures were damaged in the San Fernando earthquake of 1971. Wood (1973) reports that the walls of a large reinforced concrete underground reservoir at the Balboa Water Treatment Plant failed as a result of increased soil pressure during the earthquake. The walls were approximately 20 ft high and were restrained by the top and bottom slabs.

Damage has been reported for a number of underground reinforced concrete box-type flood control channels. Richards and Elms (1979) report damage to abutment of bridges after the 1968 earthquake in Inangahua, New Zealand. Out of the 39 bridges inspected, 24 showed measurable movement and 15 suffered damage on bridge abutments. In the Madang earthquake of 1970 in New Guinea, the damage patterns were similar. Out of 29 bridges repaired, some experienced abutment lateral movements as much as 20 inches. Reports on failed or damaged bridge abutments indicate mainly settlement of the backfill and pounding of the bridge superstructure against the abutment in longitudinal and transverse directions.

Nazarian and Hadjian (1979) also summarized damage to soil-retaining structures during past earthquakes. Damage to bridges has also been reported from various earthquakes including 1960 Chilean, 1964 Alaskan, 1964 Nigata, 1971 San Fernando, and 1974 Lima. Most of the reported damage can be attributed to the increased lateral pressure during earthquakes.

Numerous damage reports are available from recent earthquakes which report damage to the embedded walls of buildings. However, contribution of the seismic soil pressure to the damage can not be quantified since the embedded walls often carry the inertia load of the superstructure with cracks extending in all directions in the walls of the buildings. On the other hand, simple structures, such as underground box-type structures, retaining walls, and bridge abutments have suffered damage due to the increased soil pressure. All of these reports and others not mentioned highlight the significance of using appropriate seismic soil pressure in design.

RECENT EXPERIMENTS AND OBSERVATIONS

Lotung Experiment

Soil-structure interaction (SSI) effects play a significant role in the dynamic response of critical structures and internal components. Recognizing these effects, the Electric Power Research Institute (EPRI) with the cooperation from Taiwan Power Company (TPC) and the United States Nuclear Regulatory Commission (NRC) sponsored a large-scale experiment in the earthquake active area of Lotung, Taiwan. The objective of the experiment was to evaluate the SSI analyses methodologies and to reduce uncertainties in the design. In this experiment, a 1/4-scale containment model was constructed. Instrumentation was installed both in the containment model and at the site. Since completion of the model and its instrumentation in October 1985, a number of recordings from earthquakes ranging in Richter magnitude 4.5 to 7.0 have been made at the site. The information on site condition, soil properties, and structural drawings were distributed to selected teams from the industry and academia (a total of 13 groups one which was Bechtel) to predict the responses on a round-robin basis. The results of this extensive experiment and follow up studies are published in several EPRI reports (EPRI, 1989; EPRI, 1991). The senior author also participated in the studies performed by Bechtel.

The Lotung site is a relatively flat with a relatively soft surface layer with thickness of 200 ft to 260 ft (60 m to 80 m) overlying deep alluvium stratum. The soil properties in terms of low-strain shear and compression wave velocities were measured at the site. The shear wave velocity is about 100 m/sec increasing to 250 m/sec at the depth. In addition cyclic laboratory testing was performed on soil samples and the strain-dependent soil properties were obtained.

The instrumentation for the experiment is extensive and consists of accelerometers and pressure gages in the model and in the free-field. Pressure gages were installed beneath the basemat for monitoring uplifting and bonding/de-bonding of the basemat from the supporting soil layer. In addition, pressure gages were also installed on the perimeter of the containment shell to measure seismic lateral soil pressure.

A number of earthquakes up to magnitude 7 were recorded at the site. For the purpose of this study, only the records from earthquake event LSST07 are used. The LSST07 event occurred on May 20, 1986 at about 40 miles(66.2 km) from the Lotung experiment. This event had a Richter magnitude of 6.5. The peak ground acceleration in the free-field at the ground surface were 0.16g, 0.21g, and 0.04g in the east-west, north-south, and vertical directions, respectively.

A typical recorded rime histories of seismic soil pressure is shown in Figures 1. Most time histories show a drift in the response and substantial residual pressure at the end of the shaking. Some of the pressure time histories have also been examined by Chang et al. (1990). As suggested by Chang et al., the drift in the time history and the residual pressure are attributed to the compaction of the backfill material during shaking and particle re-arrangement of the materials in the soil near the instrument. For this reason the recorded pressure time histories were corrected to eliminate the drift and the residual pressure in order to obtain the peak transient stresses. The corrected pressure time history is also shown in Figure 1 with positive sign indicating pressure and negative sign indicating extension.

The seismic soil pressure shown in Figure 1 is the normal stress component with the direction normal to the body of the containment shell in the North-South direction. The magnitude of the stress is a function of the relative motion of the containment and the surrounding soil and the soil properties. In the Lotung experiment, the relative motion was caused primarily by the rigid body rocking motion of the containment shell. To evaluate the effect of rocking motion on the lateral seismic soil pressure, frequency contents of the rocking motion are compared with the frequency contents of the pressure time history at one location, as shown in Figure 2. Comparison of the pair of spectra shows that, while the nature of the spectral amplitudes are different and are expected to have different amplitudes, the frequency content of the two motions are very similar, particularly at the rocking frequency of the containment shell (2.2 Hz).

The overall comparison of the results (see Ostadan and White, 1997) indicates that the seismic soil pressure is caused by the relative motion of the structure with respect to the surrounding soil and as such it is a SSI response. This implies that the seismic soil pressure will not only be affected by the soil properties and the characteristics of the ground motion, but also the structural properties as well as the size of the structure and its foundation embedment.

Finally, the result of the SSI analysis using the computer program SASSI (Lysmer et al., 1981) in terms of seismic soil pressure was obtained and compared with the recorded pressure in terms of spectral amplitudes in Figure 3

Other Observations From Recent Field and Experimental Data

In recent years, several field and laboratory experiments have been conducted to resolve the complexities associated with the seismic soil pressure and to develop a more realistic design parameter for the design of embedded structures. A summary of the selected recent investigations is presented below.

Case 1 - Deeply Embedded Reactor Building

Hirota et al. (1992) have collected and studied the soil pressure data from instrumented buildings since 1989. Specifically, the data from a deeply embedded reactor building (embedment depth of 120 ft) in a suburb of Tokyo have been presented and evaluated. The data from a total of eight earthquake records are presented. The principal conclusions of the study are as follows:

- The seismic soil pressure is significantly affected by the low-frequency content of the earthquake motion.
- Comparison of the pressure time history with the derived relative displacement time history between the structure and the far-field shows similar characteristics in phase and amplitude.

Case 2 - Deeply Embedded Structure

Matsumoto et al. (1991) and Watakabe et al. (1992) present the results of a study using the recorded data for a deeply embedded building in a suburb of Tokyo. The site consists of a soft alluvial layer with a thickness of 120 ft underlain by a much stiffer formation. The shear wave velocity of the upper layer ranges from 300 ft/sec to 1000 ft/sec. The building foundation rests on the stiff formation. The records from a total of 21 earthquakes have been collected and examined. The main points of the investigation are as follows:

- Frequency content of the soil pressure was examined by comparing the normalized response spectra of the soil pressure with the normalized velocity spectra of the motion in the soil layers at the respective elevations. The shapes of the normalized spectra closely matched.
- The finite element method employed was able to predict the soil-interaction effects. This conclusion confirms the use of finite element and soil-structure interaction techniques to predict seismic soil pressure.

Case 3 - Underground LNG Storage Tanks

Koyama et al. (1988,1992) collected and examined the earthquake and seismic soil pressure records from two large scale Liquid Natural Gas (LNG) underground storage tanks. The instrumented tanks are large diameter concrete tanks (200 ft diameter, 120 ft high). The site soil is a medium dense sand with a shear wave velocity of 1300 ft/sec. Over the 8-year period, records from 70 earthquakes have been collected and examined. The authors concluded that the seismic soil pressure is strongly correlated to the acceleration and the relative displacement of the tank and the ground.

In addition to the field experiments, a number of laboratory tests have been recently performed Kazama and Inatoi (1988) and Itoh and Nogami (1990). Evaluation of the test results showed that:

- The dynamic soil pressure is amplified near the resonant frequency of the backfill sand.
- The effect of soil nonlinearity on the peak dynamic pressure can be observed by increasing the amplitude of the vibration.
- The dynamic soil pressure distribution is consistent with the relative displacement between the ground and the caisson.
- Finite element analysis methods are able to reproduce the measured data.
- At the soil column resonant frequency, the seismic soil pressure acts in the direction of the basement movement to drive the structure, whereas at the structural resonant frequency, the dynamic pressure acts in the opposite direction of the basement movement to restrain the movement of the structure.

Recognition of the Problem and Objective of the Study

In spite of the much better understanding of the soil-wall interaction behavior that have evolved over the years, the M-O method continues to be widely used despite many criticisms and its limitations. As stated above, the method was developed for gravity retaining walls with cohesionless backfill materials. In design applications, however, the M-O method or any of its derivatives is commonly used for below ground building walls. In this regard, the M-O method is one of the most abused methods in the geotechnical practice.

In view of the overwhelming information and evidence on the dynamic behavior of buildings, some of which was outlined above, the United States Nuclear Regulatory Commission (US NRC, 1991) recently issued a position paper on the subject of the seismic soil pressure. Pertinent excerpts are quoted as follows:

> "The use of the M-O method of analysis to compute pressure on embedded walls of structures like the nuclear island (NI) structure of ….. is not considered appropriate since the development of the limit conditions in the soil requires wall movements which are most likely inappropriate for SSI conditions anticipated. The M-O approach will generally lead to a lower bound estimate for soil loads (using active state conditions in the soil) since the soil in the active wedge is assumed to transfer part of the load to the soil below through its own shear strength..."

It is the objective of this study to develop a simple and practical method to predict lateral seismic soil pressure for building walls.

- The walls of the buildings are often of the non-yielding type. The movement of the walls is limited due to the presence of the floor diaphragms, and displacements to allow development of the limit-state conditions are unlikely to develop during the design earthquake.
- The frequency content of the design motion is fully considered. Use of a single parameter as a measure of design motion such as peak ground acceleration may misrepresent the energy content of the motion, at frequencies important for soil amplifications.
- Appropriate soil properties are included in the analysis. For soil dynamic problems, the most important soil property is the shear wave velocity followed by the material damping, Poisson's ratio, and density of the soil.
- The method is flexible to allow for consideration of soil nonlinear effect where soil nonlinearity is expected to be important.
- The interaction between the soil and the building is represented. This includes consideration for the building rocking motion, amplification and variation of the motion in the soil, geometry, and embedment depth of the building.

SIMPLIFIED METHOD TO PREDICT LATERAL SEISMIC SOIL PRESSURE FOR BUILDING WALLS ON ROCK OR FIRM FOUNDATIONS

In this section, the dynamic characteristics of lateral seismic soil pressure for buildings with basemat resting on rock or firm soil layers are examined and a simplified method for predicting seismic soil pressure is presented. It is assumed that the building walls are effectively rigid. The condition that the basemat rests on a firm soil layer also simplifies the problem in that the rocking vibration of the buildings becomes insignificant. With this assumption, the embedment ratio of the building (embedment depth to basemat width) will not play a role in the results. The extension of the method for buildings embedded in deep soil layers is presented in the next section.

To investigate the characteristics of the lateral seismic soil pressure, a series of seismic soil-structure interaction analyses was performed using the Computer Program SASSI. A typical SASSI model of a building basement is shown in Figure 4. The embedment depth is designated by H and the soil layer is identified by the shear wave velocity, Vs, the Poisson's ratio, ν, total mass density, ρ, and the soil material damping, β. The basemat is resting on rock or a firm soil layer. A column of soil elements next to the wall is included in the model in order to retrieve the pressure responses from the results.

For this analysis, the acceleration time history of the input motion was specified at the top of the rock layer corresponding to the basemat elevation in the free-field. In order to characterize the dynamic behavior of the soil pressure, the most commonly used wave field consisting of vertically propagating shear waves was specified as input motion. The frequency characteristics of the pressure response were examined using harmonic shear waves for a wide range of frequencies. For each harmonic wave, the amplitude of the normal soil pressure acting on the building wall at three locations (Elements 2, 10, and 15 in Figure 4) was obtained. The pressure responses are presented in terms of pressure transfer function amplitudes which are the ratio of the amplitude of the seismic soil pressure in the respective element to the amplitude of the input motion (1g harmonic acceleration) in the free-field for each harmonic frequency. The analyses were performed for a building with embedment of 50 ft and soil shear wave velocities of 500, 1000, 1500, and 2000 ft/sec, all with the Poisson's ratio of 1/3. The material damping in the soil was specified to be 5%. The transfer function results for Element 2 (see Figure 4) are shown in Figure 5. As shown in this figure, the amplification of the pressure amplitude takes place at distinct frequencies. These frequencies increase as the soil shear wave velocity increases. The amplitude of soil pressure at low frequency was used to normalize the amplitude of the pressure transfer functions for each element. The frequency axis was also normalized using the soil column frequency which was obtained from the following relationship:

$$f = Vs / (4 \times H) \tag{1}$$

In the above equation, Vs is the soil shear wave velocity and H is the embedment depth of the building. The normalized transfer functions are shown in Figure 6. As shown in this figure, the amplification of the pressure is about the same for all the shear wave velocities considered. In all cases the maximum amplification takes place at the frequency corresponding to the soil column frequency. Similarly, the results for points in the mid-height and bottom of the wall were examined (Ostadan and White, 1997). These results also showed the same characteristics described above.

Examining the dynamic characteristics of the normalized pressure amplitudes (such as those shown in Figure 6), it is readily evident that such characteristics are those of a single degree-of-freedom (SDOF) system. Each response begins at a value of one and increases to a peak value at a distinct frequency and subsequently reduces to a small value at high frequency. Dynamic behavior of a SDOF system is completely defined by the mass, stiffness and associated damping constant. It is generally recognized that response of a SDOF system is controlled by the stiffness at low frequency, by damping at resonant frequency, and by the inertia at high frequencies.

Following the analogy for a SDOF system and in order to characterize the stiffness component, the pressure amplitude at low frequencies for all elements (Elements 1 through 15 in Figure 4) was obtained and plotted as shown in Figure 7 in terms of the normalized height (Y/H, H=50 ft; Y is the distance from the base of the wall as shown in Figure 4). The pressure amplitudes at low frequency are almost identical for the wide range of the soil shear wave velocity profiles considered. The sudden increase shown at the top of the profile is due to the zero stress boundary condition near the ground surface and can be improved if finer elements are used. However, it is also generally recognized that soils particularly at shallow depths with low confining pressure have low shear strength and are subject to softening during vibration. For this reason, the normalized pressure profile was adjusted to have a vertical tip as shown in Figure 7. The shape of the normalized pressure will be used as a basis to determine seismic soil pressure along the height of the building wall. This will be discussed after the seismic soil pressure is examined for cases in which input motion is specified at the ground surface level.

A similar series of parametric studies were also performed by specifying the input motion at the ground surface level (Ostadan and White, 1997). The results of these studies also showed that the seismic soil pressure in normalized form can be represented by a single degree-of-freedom (SDOF) system. For both cases considered, the low frequency pressure profiles depict the same distribution of the pressure along the height of the wall as shown in Figure 7. This observation is consistent with the results of the analytical model developed by Veletsos and Younan (1994a). Since all the soil-structure interaction analyses were performed for the Poisson's ratio of 1/3, the pressure distribution was adjusted for the soil's Poisson's ratio using the factor recommended by Veletsos and Younan (1994a). The ψ_v factor is defined by:

$$\psi_{\nu} = \frac{2}{\sqrt{(1-\nu)(2-\nu)}} \tag{2}$$

For the Poisson's ratio of 1/3 used in the analysis, ψ_{ν} is 1.897. Use of ψ_{ν} in the formulation allows correction of the soil pressure amplitude for various Poisson's ratios. The adjusted soil pressure distribution is also shown in Figure 7. Using the adjusted pressure distribution, a polynomial relationship was developed to fit the normalized pressure curve. The relationship in terms of normalized height, y = Y/H (Y is measured from the bottom of the wall and varies from 0 to H), is as follows:

$$p(y) = -.0015 + 5.05y - 15.84y^2 + 28.25y^3 - 24.59y^4 + 8.14y^5 \tag{3}$$

The area under the curve can be obtained from integration of the pressure distribution over the height of the wall. The total area is 0.744 in terms of normalized wall height or 0.744H for the wall with the height H.

Having obtained the normalized shape of the pressure distribution, the amplitudes of the seismic pressure can be also obtained from the concept of a SDOF. The response of a SDOF system subjected to earthquake loading is readily obtained from the acceleration response spectrum of the input motion at the damping value and frequency corresponding to the SDOF. The total load is subsequently obtained from the product of the total mass times the acceleration spectral value at the respective frequency of the system.

To investigate the effective damping associated with the seismic soil pressure amplification and the total mass associated with the SDOF system, the system in Figure 4 with wall height of 50 ft and soil shear wave velocity of 1500 ft/sec was subjected to six different input motions in successive analyses. The motions were specified at the ground surface level in the free-field. The acceleration response spectra of the input motions at 5% are shown in Figure 8. The motions are typical design motions used for analyses of critical structures. From the set of six shown in Figure 8, two motions labeled EUS local and distant are the design motions for sites in Eastern US with locations close and far away from a major fault. The ATC S1 motion is the ATC recommended motion for S1 soil conditions. The WUS motion is the design motion for a site close to a major fault in Western US. The RG1.60 motion is the standard site-independent motion used for nuclear plant structures. Finally, the Loma Prieta motion is the recorded motion from the Loma Prieta earthquake scaled to 0.3g maximum acceleration. This motion is used in the analysis as described in later sections. All motions are scaled to 0.30g and limited to frequency cut-off of 20 Hz for use in the analysis. The cut-off frequency of 20 Hz reduces the peak ground acceleration of the EUS local motion to less than 0.30g due to high frequency content of this motion as shown in Figure 8.

The maximum seismic soil pressure values at each depth obtained from the analyses for the various input motions are shown in Figure 9. The amplitudes of the pressure vary from one motion to the other with larger values associated with use of RG1.60 motion. Using the pressure profiles in Figure 9, the lateral force acting on the wall for each input motion was computed. The lateral force represents the total inertia force of a SDOF for which the system frequency is known. The system frequency for the case under consideration is the soil column frequency which is 7.5 Hz based on Eqn (1). The total force divided by the spectral acceleration of the system at 7.5 Hz at the appropriate damping ratio amounts to the mass of the SDOF. To identify the applicable damping ratio, the acceleration response spectrum of the free-field response motions at the depth of 50 ft were computed for all six motions shown in Figure 8 for damping ratios of 5, 10, 20, 30, 40, 50, and 60 percents. Knowing the total force of the SDOF, the frequency of the system, and the input motion to the SDOF system, the relationship in the form proposed by Veletsos and Younan (1994a) was used to compute the total mass and the damping of the SDOF system. For the total mass, the relationship is

$$m = 0.50 \text{ x } \rho \text{ x } H^2 \text{ x } \psi_\nu \tag{4}$$

where ρ is the mass density of the soil, H is the height of the wall, and ψ_ν is the factor to account for the Poisson's ratio as defined in Eqn (2). In the analytical model developed by Veletsos and Younan, a constant coefficient of 0.543 was used in the formulation of the total mass. Study of the soil pressure transfer functions and free-field response motions at the depth of 50 ft showed that spectral values at the soil column frequency and at 30% damping have the best correlation with the forces computed directly from the SSI analysis. In the Veletsos and Younan's model, a damping of 27.5 + β percent has been proposed where β is the material damping of the soil (%). For the case of 5% soil material damping, the proposed spectral damping amounts to 32.5%. However, as shown by Ostadan and White (1997), the spectral values of the various motions considered are insensitive to the spectral damping ratios at the soil column frequency of 7.5. The various motions, however, have significantly different spectral values at the soil column frequency. This observation leads to the conclusion that while the frequency of the input motion particularly at the soil column frequency is an important component for magnitude of the seismic soil pressure, the spectral damping ratio selected is much less important in terms of pressure amplitudes. The role of soil material damping is discussed by Ostadan and White (1997).

Simplified Method: Computational Steps

To predict the lateral seismic soil pressure for below ground building walls resting on firm foundation and assuming rigid walls, the following steps should be taken:

1. Perform free-field soil column analysis and obtain the ground response motion at the depth corresponding to the base of the wall in the free-field. The response motion in terms of acceleration response spectrum at 30% damping should be obtained. The free-field soil column analysis may be performed using the Computer Program SHAKE (Schnabel et al., 1972) with input motion specified either at the ground surface or at the depth of the foundation basemat. The choice for location of control motion is an important decision that needs to be made consistent with the development of the design motion. The location of input motion may significantly affect the dynamic responses of the building and the seismic soil pressure amplitudes.

2. Use Eqn (4) to compute the total mass for a representative SDOF system using the Poisson's ratio and mass density of the soil.

3. Obtain the lateral seismic force from the product of the total mass obtained in Step 2 and the acceleration spectral value of the free-field response at the soil column frequency obtained at the depth of the bottom of the wall (Step 1).

4. Obtain the maximum lateral seismic soil pressure at the ground surface level by dividing the lateral force obtained in Step 3 by the area under the normalized seismic soil pressure, 0.744 H.

5. Obtain the pressure profile by multiplying the peak pressure with the pressure distribution relationship shown in Eqn (3).

One of the attractive aspects of the simplified method is its ability to consider soil nonlinear effect. The soil nonlinearity is commonly considered by use of the equivalent method and the strain-dependent soil properties. Depending on the intensity of the design motion and soil properties, the effect of soil nonlinearity can be important in changing the soil column frequency and therefore, amplitude of the spectral response at the soil column frequency.

Accuracy of the Simplified Method

The simplified method outlined above was tested for building walls with heights of 15, 30 and 50 ft using up to six different time histories as input motion. The results computed directly with SASSI are compared with the results obtained from the simplified solution. A typical comparison is shown in Figure 10. More extensive validation of the method is presented by Ostadan and White (1997).

Comparison to Other Commonly Used Solutions

The seismic soil pressure results obtained for a building wall 30 ft high embedded in a soil layer with shear wave velocity of 1000 ft/sec using the M-O, Wood and the proposed simplified methods are compared in

Figure 11. For the simplified method, the input motions defined in Figure 8 were used. The M-O method results in the smallest pressure values. This is understood since this method relies on the wall movement to relieve the pressure behind the wall. Wood's solution generally results in the maximum soil pressure and is independent of the input motion as long as the peak acceleration is 0.3 g. The proposed method results in a wide range of pressure profiles depending on the frequency contents of the input motion, particularly at the soil column frequency. For those motions for which the ground response motions at the soil column frequency are about the same as the peak ground acceleration of the input motion, e.g., RG1.60 motion, the results of the proposed method are close to Wood's solution. Similar trend in the results is observed if sum of the lateral forces and the overturning moments from the above three methods are compared (Ostadan and White, 1997).

The simplified method was extended for application to soil layered system and soil deposits with parabolic distribution of the shear modulus. The extended method and its verification are discussed by Ostadan and White (1997).

SIMPLIFIED METHOD TO PREDICT LATERAL SEISMIC SOIL PRESSURE FOR BUILDINGS IN DEEP SOIL SITES

One of the distinct dynamic characteristics of a building in a deep soil site is its rocking vibration which has a significant role on distribution of the pressure depending on the embedment ratio (embedment depth versus plan dimensions), dynamic properties of the soil, and frequency contents of the ground motion under consideration.

Mita and Luco (1989) have reported the harmonic response of an embedded square foundation subjected to vertically propagating shear waves. The results adopted from the authors but modified to reflect the same nomenclature used in this report are shown in Figure 12. The results are for a square foundation with plan dimensions of 2B x 2B and embedment depth H. The halfspace is characterized by the shear wave velocity of Vs. The free-field motion has a unit amplitude at the ground surface at each harmonic frequency. The horizontal translational motion of the foundation (D) at the middle point corresponding to the basemat motion and the normalized rocking motion represented in terms of HxT are shown in terms of dimensionless frequency ratio $a_1 = \omega x H/Vs$ where T is the angle of rocking rotation and ω is the circular frequency at each harmonic frequency under consideration. The dimensionless frequency is a measure of the harmonic shear wave length as compared to the embedment depth H. The free-field motion corresponding to the basemat depth (depth of H) in the free-field shows decreasing amplitude with increasing frequency. At the soil column frequency of $f = Vs/(4xH)$, the dimensionless frequency a1 is 1.57 at which the amplitude of the free-field motion is zero. The foundation motion is a function of the frequency of vibration and the embedment ratio (H/B).

In order to examine the effect of rocking motion on seismic soil pressure, a series of SSI analyses were performed using the soil shear wave velocities of 500, 1000, 1500, and 2000 ft/sec. In all cases, the wall height considered was H=50 ft but the foundation width (2B) was changed successively from 50 ft, to 100 ft, 200 ft, and to 400 ft, resulting in embedment ratios of B/H=0.5, 1, 2, and 4. The input motion was specified at the basemat level in the free-field. A typical result in terms of amplitude of pressure transfer function is shown in Figures 13. For each soil case, the results from all three elements are clustered together with the same peak frequency which leads to the conclusion that (1) the soil column frequency continues to be the most significant frequency for the response in terms of maximum value of the seismic soil pressure, and (2) the frequency of the peak response is not affected by the embedment ratio. However, the distribution of the maximum soil pressure in terms of amplitude of the pressure in Elements 2, 5, and 10 is significantly affected by the rocking motion of the building and thus the embedment ratio. The effects of rocking motion on distribution of maximum seismic soil pressure for four different aspect ratios are shown in Figure 14. As shown, for buildings with narrow width, the rocking motion tends to reduce the amplitude of the soil pressure at top of the wall.

The results of the parametric studies performed for deep soil sites were also examined in detail. Limitation of space prohibits detail discussion of the studies performed. The computational steps for deep soil sites are, however, similar to the rigid case and consist of the following:

1. Perform free-field soil column analysis and obtain the response motion in terms of acceleration response spectrum at 30% damping at the depth corresponding to the basemat elevation in the free-field.
2. Obtain the soil column frequency using Eqn (1) and obtain the spectral value at the soil column frequency using the results of Step 1.
3. Use the following relationship to obtain the lateral force acting on the wall:

$$F = \alpha \times \rho \times H^2 \times Sa \times \Psi_v \qquad (5)$$

where ρ is the mass density of the soil, H is height of the wall, Sa is the spectral value of the free-field response obtained in Step 2, and Ψ_v is the function that considers the effect of soil Poisson's ratio and can be obtained using Eqn (2). In order to represent the effect of the embedment ratio and reduction of soil pressure due to rocking motion as well as its increase beyond the rigid base cases for wide buildings, the parameter α is defined in the equation above. This parameter was determined from back-calculation of the lateral force obtained from soil pressure and the shear stress under the basemat to hold the equilibrium of forces in the horizontal direction. Using the results of the all the parametric studies, the following values were obtained for α:

Embedment Ratio, B/H	Parameter α
0.50	0.27
1.0	0.43
2.0	0.62
4.0	0.92

4. Obtain the maximum soil pressure by dividing the lateral force obtained from Step 3 to the area under the soil pressure curve provided in Eqns(6) through (9) below depending on the embedment ratio. For an embedment ratio that falls in between the ratios considered, use interpolation.

Embedment ratio of B/H=0.50

$$p(y) = -2.58y^3 + 0.32\,y^2 + 2.46\,y - 0.03 \qquad (6)$$

Maximum pressure at the depth y = 0.625

Area under the curve = 0.632H

Point of application for resultant force, Y = 0.55H

Embedment ratio of B/H=1.0

$$p(y) = 0.60y^3 - 3.09y^2 + 3.34y - 0.025 \qquad (7)$$

Maximum pressure at the depth y = 0.625

Area under the curve = 0.77H

Point of application for resultant force, Y = 0.58H

Embedment ratio of B/H=2.0

$$p(y) = -1.33y^4 + 4.38y^3 - 5.66y^2 + 3.44y + 0.17 \qquad (8)$$

Maximum pressure at top of the wall y = 1

Area under the curve = 0.832H

Point of application for resultant force, Y = 0.57H

Embedment ratio of B/H=4.0

$$p(y) = -0.085y^2 + 0.47y + 0.61 \qquad (9)$$

Maximum pressure at top of the wall y = 1

Area under the curve = 0.82H

Point of application for resultant force, Y = 0.54H

5. Multiply the maximum lateral soil pressure from Step 4 by the relationships provided in Eqns (6) through (9) to get the pressure distribution depending on the embedment ratio of the foundation under

consideration. Judgment should be exercised to obtain the distribution for embedment ratios in between the four embedment ratios considered above.

The simplified method for deep soil sites was also tested extensively for a wide range of soil properties and foundation embedment ratios (Ostadan and White, 1997).

A comparison of the simplified method with the M-O and Wood's methods for a building with four different embedment ratios is shown in Figure 15. The results clearly demonstrates the effect of the rocking motion on distribution of the seismic soil pressure.

SUMMARY AND CONCLUSION

The Mononobe-Okabe (M-O) method was developed in the 1920's. Since then, a great deal of research work has been performed to evaluate its adequacy and to improve it. The method is, strictly speaking, applicable to gravity retaining walls which, upon experiencing seismic loading, undergo relatively large movement to initiate the sliding wedge behind the wall and to relieve the pressure to its active state. Unfortunately, the method has been and continues to be used extensively for embedded walls of the buildings as well. Recent field observations and experimental data, along with enhancements in analytical techniques have shown that hardly any of the assumptions used in the development of the M-O method are applicable to building walls. The data and the follow up detail analysis have clearly shown that the seismic soil-pressure is an outcome of the interaction between the soil and the building during the seismic excitation and as such is function of all parameters that affect soil-structure interaction (SSI) response. Some of the more recent observations and experimental data were presented in the paper. The new understanding of the attributes of seismic soil pressure prompted the United Sates Nuclear Regulatory Committee (NRC) to reject the M-O and the M-O based methods for application to critical structures. At this time, while elaborate finite element techniques are available to obtain the soil pressure for design, no simple method has been proposed for quick prediction of the maximum soil pressure, thus hindering the designer's ability to use an appropriate method in practice. To remedy this problem, the current research was conducted to develop a simple method which incorporates the main parameters affecting the seismic soil pressure for buildings.

Using the concept of the single degree-of-freedom, a simplified method was developed to predict maximum seismic soil pressures for buildings resting on firm foundation materials. The method incorporates the dynamic soil properties and the frequency content characteristics of the design motion in its formulation. It was found that the controlling frequency that determines the maximum soil pressure is that corresponding to the soil column adjacent to the embedded wall of the building. The proposed method requires the use of conventionally-used simple one-dimensional soil column analysis to obtain the relevant soil response at the base of the wall. More importantly, this approach allows soil nonlinear effects to be considered in the process. The effect of soil nonlinearity can be important for some applications depending on the intensity of the design motion and the soil properties. Following one-dimensional soil column analysis, the proposed method involves a number of simple hand calculations in order to arrive at the distribution of the seismic soil pressure for design. The accuracy of the method relative to the more elaborate finite element analysis was verified for a wide range of soil properties, earthquake motions, and wall heights.

The method was extended to include buildings on deep soil sites . The complexity of the seismic soil pressure for such cases is compounded by the rocking motion of the structure. The rocking motion is in turn, a function of soil properties, frequency content of the design motion, and embedment ratio of the structure. A wide range of parametric studies were performed that cover many practical cases. The steps for the analysis are similar to the steps outlined for buildings on rock except that an appropriate pressure distribution curve should be selected to observe the effect of the embedment ratio. Similarly, the accuracy of the proposed method was verified against a more detailed SSI analysis.

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KINNEY
ENGINEERING, LLC

November 14, 2016

John McGrew
9831 Main Tree Drive
Anchorage, AK 99507

Subject: Christensen Drive and 2nd Avenue Development Traffic Impact Analysis

Dear Mr McGrew:

This letter conveys our investigation of the traffic impacts that can be expected to occur as a result of the proposed development of the Alaska Railroad Corporation parcel on the Northwest corner of the intersection of Christensen Drive and West 2nd Avenue (Christensen and 2nd). Based on a conversation with the Municipality of Anchorage (MOA) Traffic Department in July of 2016, this study focused on validating sight distance at the proposed driveways, measuring clearances from the intersection of Christensen and 2nd, and analyzing the performance of the intersection at Christensen and 2nd with and without the proposed development. This scope of work is consistent with the MOA's Traffic Impact Analysis policy for small developments (<100 vehicles per hour). This study shows that the proposed driveways have adequate sight distance and intersection clearance and the additional traffic generated on the site will have an insignificant impact on the performance of the stopped approaches of Christensen and 2nd, which is currently operating at satisfactory levels of service.

Background

The tract in question is approximately 1.7 acres. In general, the lot is level with a steep grade descending on the north side of the site. The lot frontage extends approximately 470 feet along 2nd Avenue to the west of the intersection, and approximately 250 feet along Christensen Drive to the north of the intersection. The lot is currently developed with a vacant Alaska Railroad structure, which would be removed as a part of this project. There are three existing driveways onto the site, one onto Christensen Drive and two onto 2nd Avenue. The driveway onto Christensen is approximately 90 feet north of the intersection of Christensen and 2nd. The two driveways onto 2nd Avenue are approximately 50 feet and 200 feet west of the intersection.

The Municipality of Anchorage 2014 Official Streets and Highways Plan classifies Christensen Drive as a "Neighborhood Collector" from 3rd Avenue through the entire length of the project site. 2nd Avenue is classified as a "Local Road". Both streets have posted speed limits of 25 miles per hour.

2nd Avenue terminates west of the study site. At the western end of 2nd Avenue is the northern terminus of the Tony Knowles Coastal Trail. This trail is heavily used by pedestrians and bicyclists which then travel adjacent to the project site. Many of the trips on the Coastal Trail travel to or from the Ship Creek Trail, which is north of the study site along Christensen Drive.

The proposed development includes 28 condominium units. The development will access the road network with two separate driveways. The main driveway will serve 22 of the new residential units and access Christensen Drive approximately 230 feet north of the Christensen and 2nd intersection. The second driveway will serve the remaining six residential units and access the road network at a new driveway onto 2nd Avenue, approximately 230 feet west of the intersection. The site plan used for this analysis is attached.

Figure 1 on page 2 shows a map of the project vicinity.

Figure 1 - Proposed Project Site and Driveway Locations

Sight Distance Analysis

The Municipality of Anchorage's *Design Criteria Manual 2007* defines the design parameters for driveways in an attached memo dated December 11, 2006. In this memo, the required sight distance at driveways is intersection sight distance for a vehicle turning left across traffic from a stop, as defined by the American Association of State Highway and Transportation Officials (AASHTO) *A Policy on the Geometric Design of Highways and Streets 2011.*

The sight distance in the required case is contingent on the grade of the major road approach. Grades were measured by Kinney Engineering, LLC (KE) on the various approaches to the proposed driveways and the approaches to the intersection of 2nd and Christensen. The results of the approach grade study are shown in Figure 2 on page 3.

Figure 2 - Road Grades

At 25 miles per hour on a 6% upgrade, the required sight distance is 252 feet. The required sight distance on a downgrade is 308 feet. The sight distance requirement on 2nd Avenue where grades are less than 6% is 280 feet in either direction. Intersection sight distance on the 2nd Avenue driveway is 300 feet to the end of the road to the west, and over 400 feet to the east through the stop controlled intersection.

Curves along Christensen Avenue north and south of the site raised concerns about sight distance at Driveway #1. The available sight distance at the proposed driveways were measured by KE in August of 2016. Sight distance measurements were made using AASHTO green book methodology, which measured the in lane distance from the first point on the approach road where there is unobstructed sight to a point at the driveway 3.5 feet high and 14.5 feet off the traveled way.

The results of the sight distance study are shown in Figure 3 on page 4.

Figure 3 - Field Measured Sight Distances at Driveway #1

The field measured sight distance to the north of driveway #1 is 256 feet, which exceeds the required distance of 252 feet. Note that the sight line is through the parking lot on the south side of Christensen. The sight line was partially obstructed by parked vehicles and a short fence until a distance of 230 feet prior to the driveway. The case of partial obstruction by trees and other objects is allowed in AASHTO.

Note, that the absolute minimum sight distance for the southbound approach, per AASHTO, is stopping sight distance which is 140 feet at 25mph on a 6% upgrade.

Figure 4 on page 5 shows a photo taken at the point near where the driveway is first visible traveling southbound on Christensen Drive.

Figure 4 - Sight Distance to Driveway #1 through Parking Lot

The field measured sight distance to the south of driveway #1 is 360 feet, which exceeds the required 308 feet for intersection sight distance and 173 feet for stopping sight distance.

Intersection Performance Analysis

The Christensen and 2nd intersection is two way stop control with free movements along Christensen and stop control for both of the 2nd Avenue approaches. The performance of the intersection was studied by first observing existing turning movement volumes between the hours of 5:00 and 7:00 on Wednesday, August 24, 2016. Performance during the peak hour period was computed using 2010 Highway Capacity Manual methodology in Synchro software.

Figure 5 on page 6 shows the observed turning movement volumes and the performance grades on each of the approaches.

Figure 5 – Intersection Performance at Christensen Drive and 2nd Avenue, 2016 PM

MOA strives to maintain at least LOS D (maximum delay of 35 seconds) at the intersection of a local road and a collector.

The new development will add additional traffic to the intersection. However, the proposed development of 28 condominium units will generate relatively low levels of traffic.

The Institute of Transportation Engineer's (ITE) *Trip Generation* (9th Edition), reports that the average trip generation for residential condominium/Townhouses (land use 230) is as follows:

Land Use and Inputs			Trip Generation by Period						
Land Use No.	Description	Units	Daily Trips	AM Peak Trips	AM in	AM out	PM Peak Trips	PM in	PM out
230	Residential Condominium/Townhouse	28	213	19	4	15	21	14	7

PM peak hour trips are projected to be 65 percent entering and 35 percent exiting. Six out of the total 28 units will share a driveway onto 2nd Avenue with the remaining units using a common driveway onto Christensen Drive. The increased volumes were overlaid onto the observed volumes and the effect of the new volumes were less than a half second of increased delay. None of the performance grades would be diminished as a result of the development.

Intersection Clearance

The MOA design standards for driveways include recommended corner clearance from intersections. The clearances are based on the per hour traffic volumes produced by the generator, the functional classification of the road, and whether the road is curbed or uncurbed.

The conditions on Driveway #1 require 60 feet of separation from the intersecting cross street. The proposed location provides 230 feet of separation. Likewise, the required clearance for Driveway #2 is 40 feet, and the available clearance at the current proposed location is 90 feet. Therefore, the proposed driveway locations exceed the required clearances.

Summary

The proposed construction of 28 condominium units northwest of the intersection of Christensen Drive and 2nd Avenue will generate approximately 213 daily vehicle trips, 21 of which will occur in the PM peak hour. This additional traffic will not reduce the level of service at the adjacent intersection below the existing level of service B. In addition, the proposed driveway locations meet MOA corner clearance and sight distance requirements.

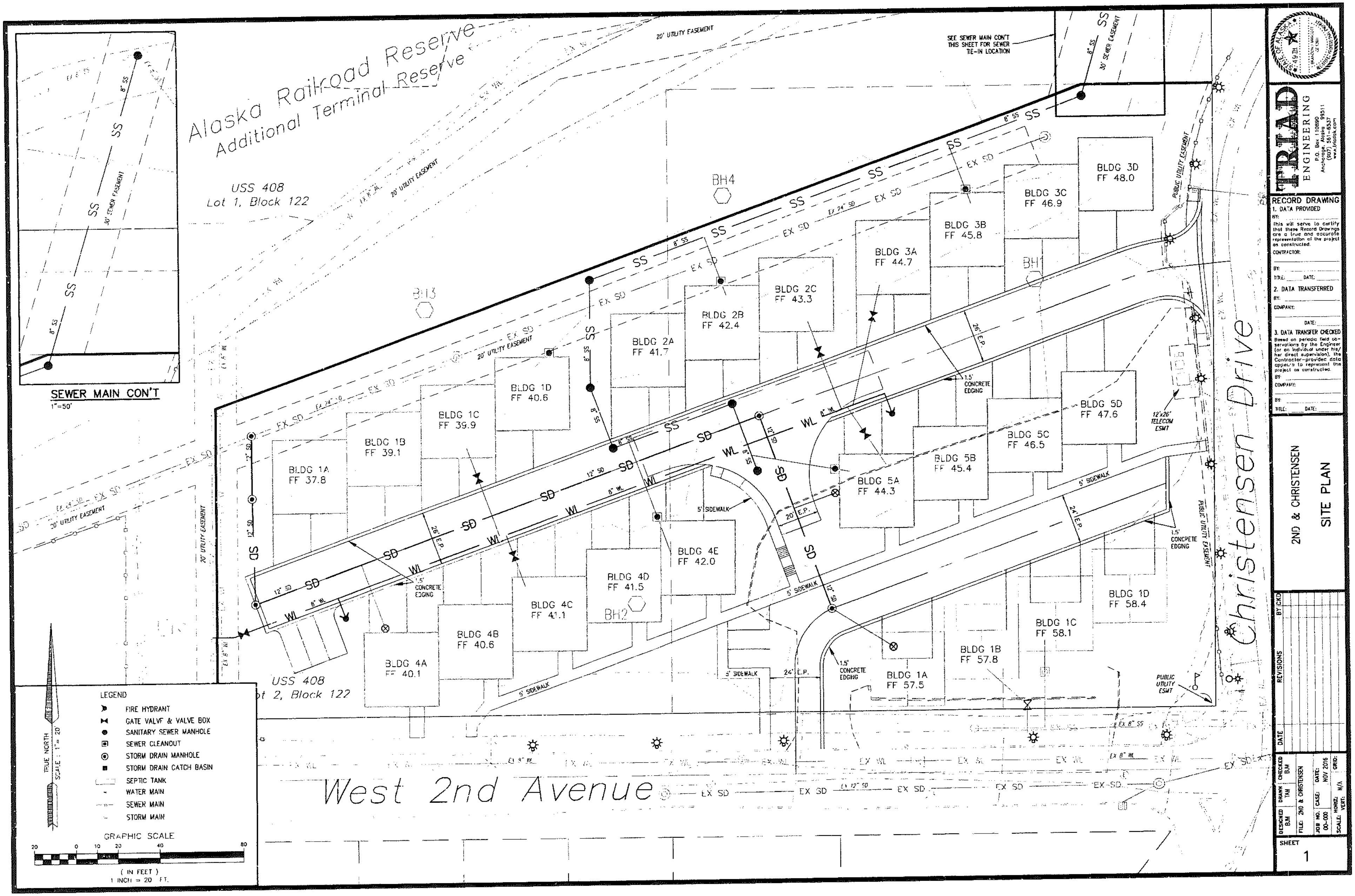
Sincerely,

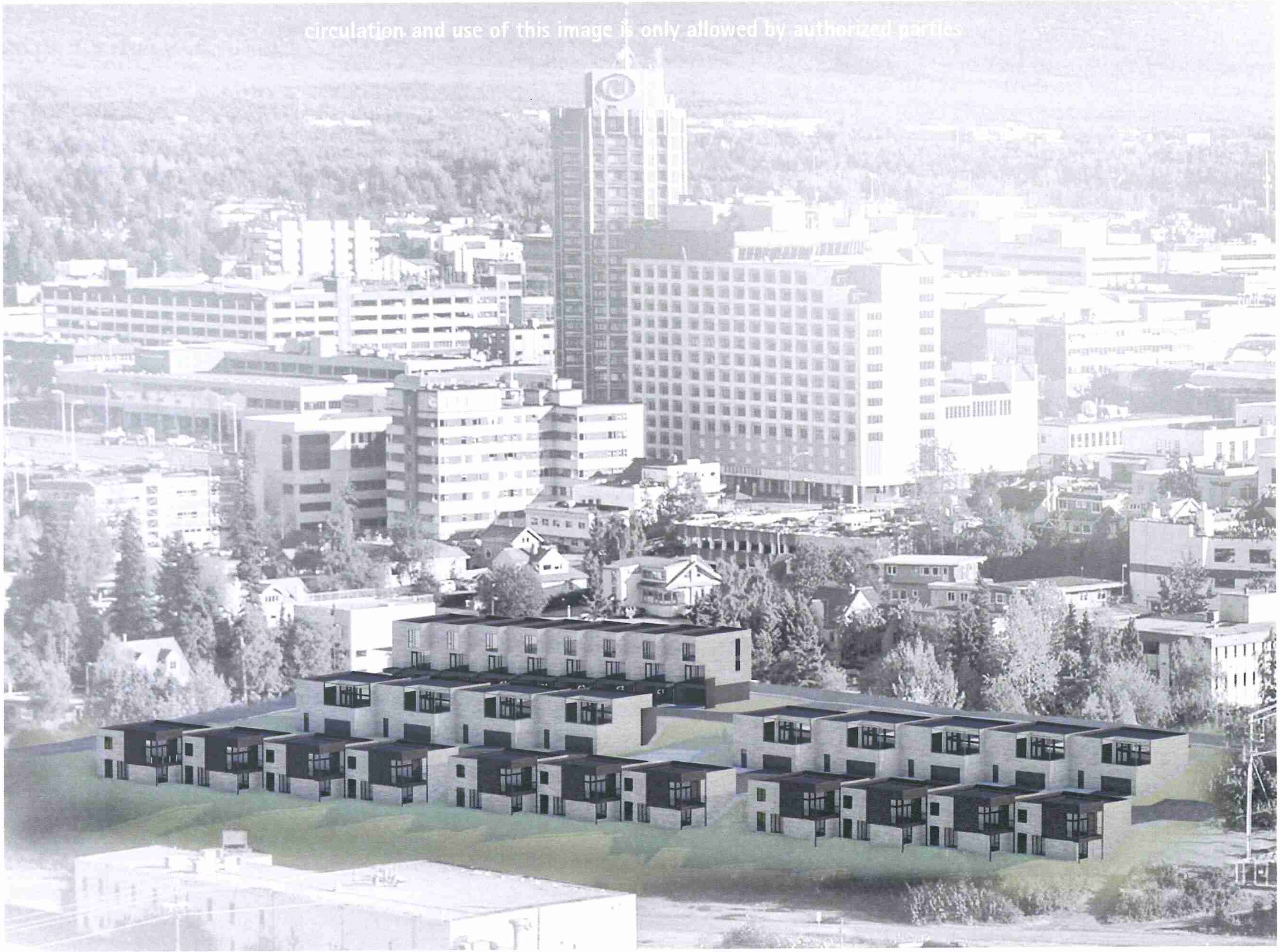
KINNEY ENGINEERING, LLC

Will Webb

Will Webb, P.E., PTOE
Senior Civil Engineer

Attachments: Site Plan

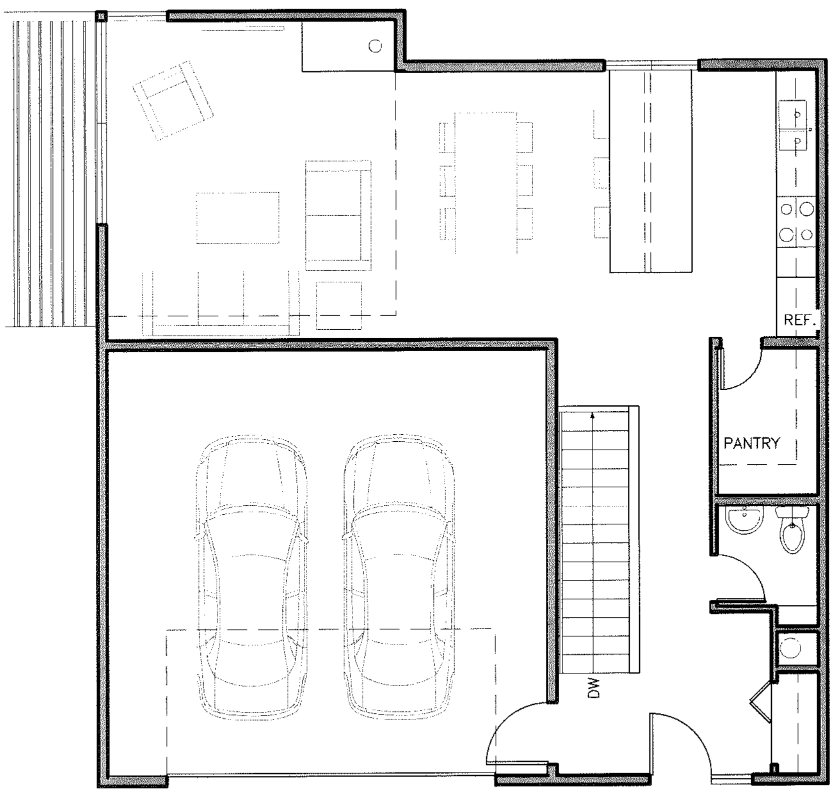

Alaska Railroad Reserve
Additional Terminal Reserve
USS 408
Lot 1, Block 122
USS 408
Lot 2, Block 122
West 2nd Avenue
Christensen Drive
BLDG 1A FF 37.8
BLDG 1B FF 39.1
BLDG 1C FF 39.9
BLDG 1D FF 40.6
BLDG 2A FF 41.7
BLDG 2B FF 42.4
BLDG 2C FF 43.3
BLDG 3A FF 44.7
BLDG 3B FF 45.8
BLDG 3C FF 46.9
BLDG 3D FF 48.0
BLDG 4A FF 40.1
BLDG 4B FF 40.6
BLDG 4C FF 41.1
BLDG 4D FF 41.5
BLDG 4E FF 42.0
BLDG 5A FF 44.3
BLDG 5B FF 45.4
BLDG 5C FF 46.5
BLDG 5D FF 47.6
BLDG 1A FF 57.5
BLDG 1B FF 57.8
BLDG 1C FF 58.1
BLDG 1D FF 58.4
BH1
BH2
BH3
BH4
SEE SEWER MAIN CON'T THIS SHEET FOR SEWER TIE-IN LOCATION
30' SEWER EASEMENT
20' UTILITY EASEMENT
PUBLIC UTILITY EASEMENT
PUBLIC UTILITY ESMT
12'x26' TELECOM ESMT
1.5' CONCRETE EDGING
5' SIDEWALK
26' E.P.
24' E.P.
20' E.P.
SEWER MAIN CON'T
1"=50'
LEGEND
FIRE HYDRANT
GATE VALVE & VALVE BOX
SANITARY SEWER MANHOLE
SEWER CLEANOUT
STORM DRAIN MANHOLE
STORM DRAIN CATCH BASIN
SEPTIC TANK
WATER MAIN
SEWER MAIN
STORM MAIN
GRAPHIC SCALE
(IN FEET)
1 INCH = 20 FT.
TRUE NORTH
SCALE : 1" = 20'
TRIAD
ENGINEERING
P.O. Box 110690
Anchorage, Alaska 99511
(907) 561-6537
www.triadak.com
RECORD DRAWING
1. DATA PROVIDED
This will serve to certify that these Record Drawings are a true and accurate representation of the project as constructed.
CONTRACTOR:
BY:
TITLE:
DATE:
2. DATA TRANSFERRED
BY:
COMPANY:
DATE:
3. DATA TRANSFER CHECKED
Based on periodic field observations by the Engineer (or an individual under his/her direct supervision), the Contractor-provided data appears to represent the project as constructed.
BY:
COMPANY:
BY:
TITLE:
DATE:
2ND & CHRISTENSEN
SITE PLAN
DATE
REVISIONS
BY CKD
DESIGNED B.M
DRAWN TAM
CHECKED B.M
FILE: 2ND & CHRISTENSEN
JOB NO. 00-000
CASE:
DATE: NOV 2016
HORIZ:
VERT: N/A
SCALE:
GRID:
SHEET
1



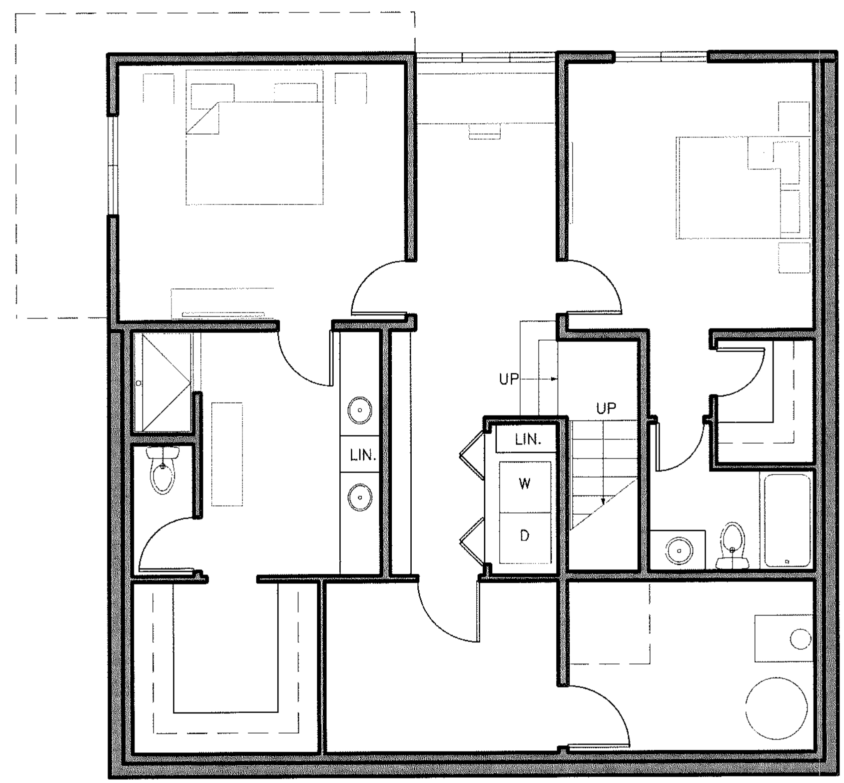
overall aerial view

arrc lumen design 12 15 2016

site plan 50′ arrc lumen design 12 15 2016



level 2:
821 gross sf living
477 gross sf garage
54 gross sf deck

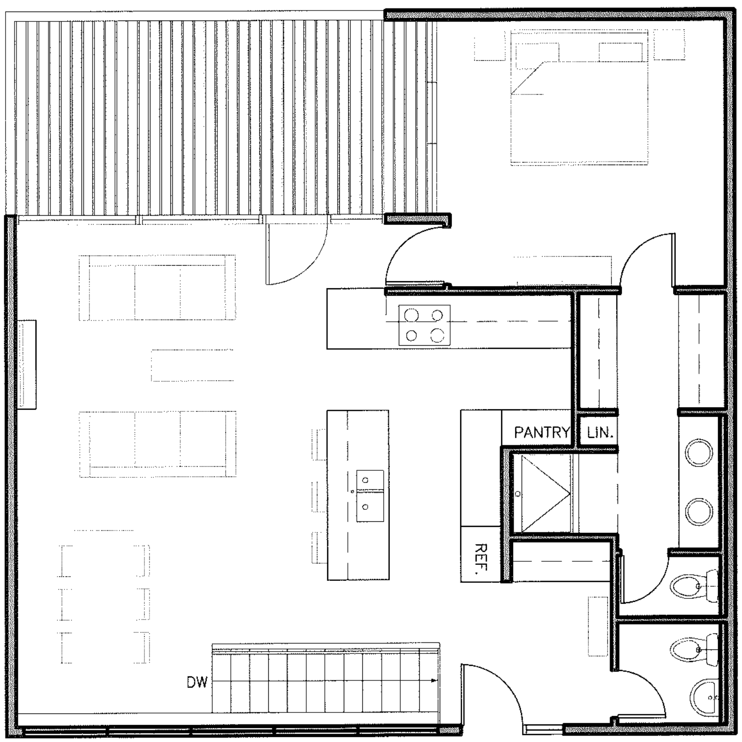


level 1:
1129 gross sf living
131 gross sf utility storage

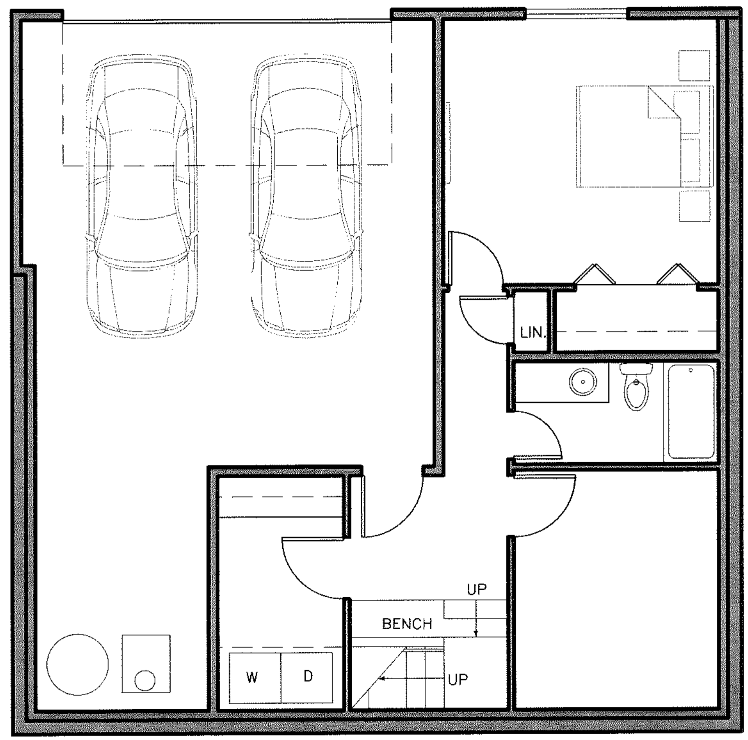
unit A plans

10'

arrc lumen design 12 15 2016



level 2:
1055 gross sf living
205 gross sf deck

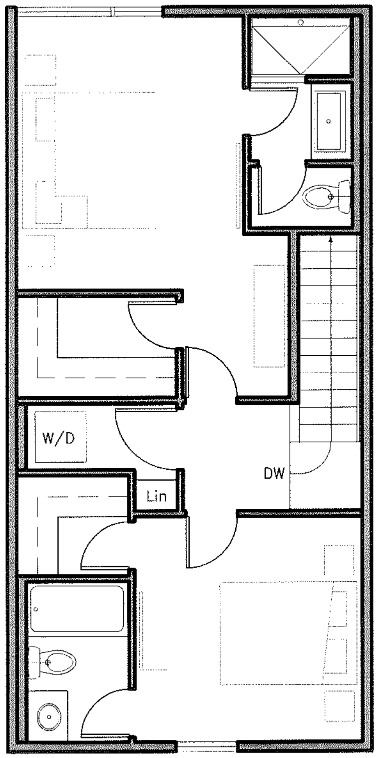


level 1:
670 gross sf living
590 gross sf garage

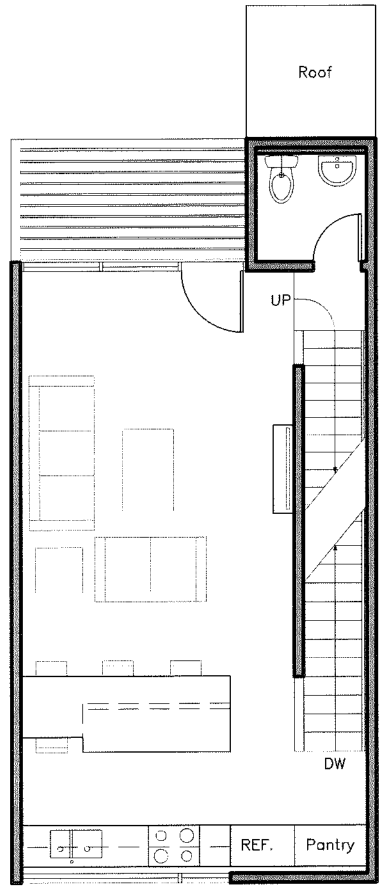
unit B plans

10'

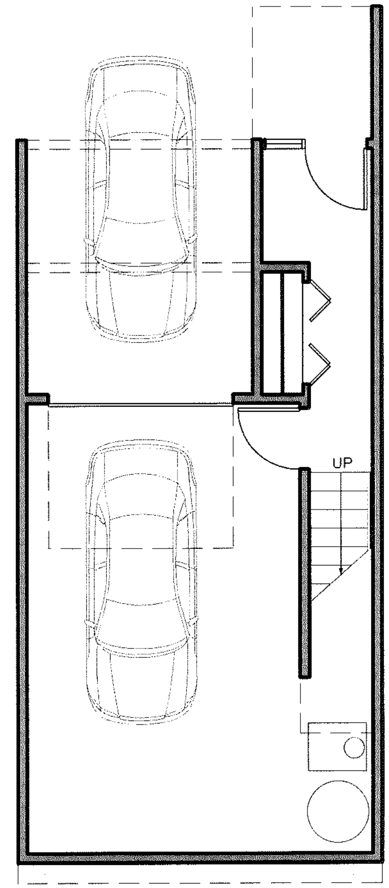
arrc lumen design 12 15 2016



level 3:
625 gross sf living



level 2:
542 gross sf living
72 gross sf deck



level 1:
141 gross sf living
351 gross sf garage

unit C plans 10'

arrc lumen design 12 15 2016

MUNICIPALITY OF ANCHORAGE

Project Management & Engineering
4700 Elmore Road
Anchorage, AK 99517

Ph: 907.343.8135
Fax: 907.343.8088

Mayor Ethan Berkowitz

December 2, 2016

Mr. Brandon Marcott, P.E.
Triad Engineering
1300 East 68th Avenue, Suite 210
Anchorage, AK 99511

Re: 2nd & Christensen – Street Section Modifications

Dear Mr. Marcott:

We have reviewed your request for our concurrence of alternate street sections for the private streets in this proposed development. For the lower street, you are requesting approval for a 26-foot wide street, measured to edge of pavement. You also propose to eliminate curb and gutter, and install an inverted crown on the lower street. You propose to treat the upper street as a private driveway with a width of 24 feet measured to edge of pavement. You propose to eliminate curb and gutter on one side of this street.

Lower Road:

We have no objection to your proposal to install an inverted crown section.

Regarding elimination of curb and gutter on this street; the site plan you provided shows 1.5-foot wide concrete edging on both sides of the street. The concrete edging is not shown on the typical section included in your letter. We have no objection to eliminating the curb and gutter as long as the concrete edging is installed. This configuration would be in general conformance with AMCR 21.90.003.F.1.a which requires that private streets have a total width of 30 feet to back of curb.

Upper Road/Driveway:

The Fire Marshal will need to approve the proposal to treat this street as a private driveway. Presuming the Fire Marshal has no objection, we would have no objection to your proposal to install a tipped section and eliminate curb and gutter on one side of the street.

Our concurrence for these reduced street sections is contingent on a minimum driveway length of 22 feet at the center of each driveway, measured to the back of the concrete edging.

Your letter includes proposed structural sections for the upper and lower road. You have not requested a variance from the requirements for structural section. Upon initial review, the sections shown in the letter do not comply with the requirements of the DCM, and a variance request would be required.

Our concurrence of your proposed street sections is contingent upon approval by the Fire Marshal. We have not reviewed the submittal for compliance with applicable codes and policies related to fire access.

No other aspects of this project have been reviewed for variances and none are granted. This variance is based on the information submitted and ultimate responsibility for the adequacy of the design solution continues to reside with the design engineer.

Should you have questions regarding this response or wish to discuss it further, please call Kent at 343-8159 or Stephanie at 343-8070.

Sincerely,

Kent Kohlhase

Kent Kohlhase, P.E.
Acting Municipal Engineer

Stephanie Mormilo

Stephanie L. Mormilo, P.E.
Municipal Traffic Engineer

cc: J.W. Hansen, Director – PM&E
Jason Moncrieff, P.E., Private Development Manager
Cleo Hill, Fire Marshal

TRIAD
ENGINEERING

PHYSICAL
1300 E. 68th Ave., Suite 210
Anchorage, AK 99518

MAILING
P.O. Box 110890
Anchorage, AK 99511

OFFICE
907-561-6537

June 7th, 2016

Municipality of Anchorage
Project Management & Engineering
4700 Elmore Rd.
Anchorage, Alaska 99519-6650

Attention: Stephanie Mormilo P.E., Municipal Traffic Engineer
Subject: 2nd & Christensen, Driveway Lengths
Master Fill and Grade Permit Number TBD

Ms. Mormilo,

This letter is a request for your concurrence of the driveway length for the 2nd and Christensen development. The proposed development resides within Alaska Railroad Corporation property located north of 2nd Avenue and west of Christensen Drive in Anchorage, Alaska. The subject development proposes to construct 24 single family units with two associated private drives that provide access. Nearly 40 feet of vertical relief occurs across the site with a high elevation of 56 in the SE corner down to elevation 18 along the NW row of buildings.

As shown in the attached site plan, the topographic relief creates challenges for building placement. The northerly tier of buildings is located along the bluff which consists of 20 to 40% grades. In an effort to keep the units from being placed too far down the slope driveway depth is kept as short as feasible. The proposed orientation of the driveway is roughly 70 degrees to the road centerline as shown in Figure 1.

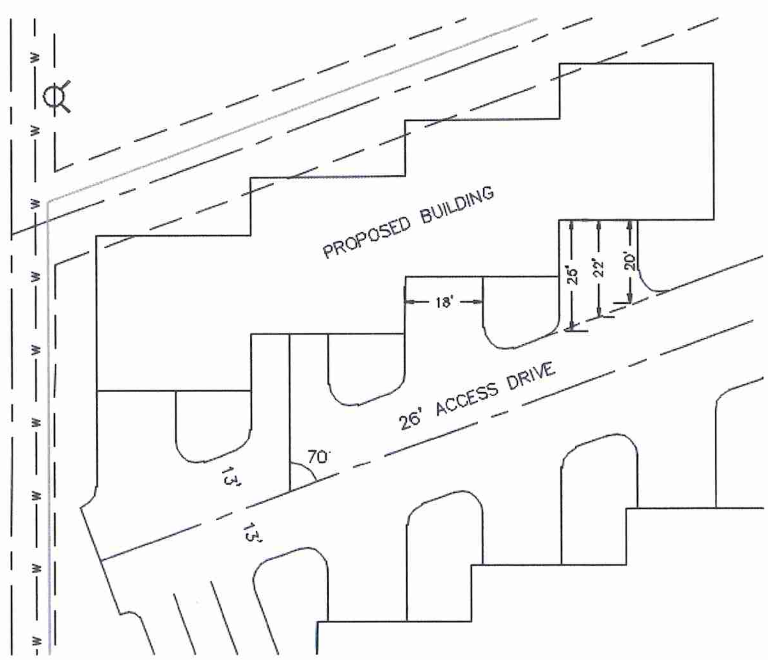


Figure 1

Centerline of the drive will be a minimum length of 22' in compliance with code requirements. The short side of the driveway will maintain a minimum length of 20'. Please note that this side of the driveway is not needed for parking calculations although it is anticipated that it will be used for parking.

In summary the proposed driveway configuration meet the required parking calculations and the intent of code while minimizing negative impacts to the site plan. Triad Engineering respectfully requests your concurrence of the above design approach prior to submittal of the site plan to Planning with the understanding that a formal variance request will follow as the design plans are finalized.

Thank you for your time and consideration in this matter. If you have any questions or require additional information, please call 344-3114 or email me at brandonmarcott@triadak.com.

Sincerely,
TRIAD ENGINEERING

Brandon Marcott, P.E.

Concur

Stephanie Mormilo

Stephanie Mormilo, P.E.
Municipal Traffic Engineer

6/21/16
Date

Cc: Dave Grenier, P.E., Triad Engineering
Tony Hoffman, P.L.S., The Boutet Company, Inc.
Petra Sattler-Smith, Lumen Design
John McGrew
Glenn Gellert
Trevor Edmondson
Francis McLaughlin

Authorization Certificate

Date: December 14, 2016

Current Project Legal Description:

A parcel of land located within the Alaska Railroad Anchorage Reserve situated in the Anchorage Recording District, Third Judicial District, State of Alaska and further described as follows:

A portion of Lots 1 and 2, Block 122, East Addition to Anchorage Original Town Site, U.S. Survey 408, Anchorage, Alaska, and;

A portion of Lot 2 U.S. Survey 1170, Anchorage, Alaska.

Proposed Legal: Same

Type of Authorization: Preliminary Plat and Site Plan Applications

Statement:

I hereby authorize Tony Hoffman of The Boutet Company Inc. to represent The Alaska Railroad Corporation in the Municipality of Anchorage Platting and Site Plan Applications of the above described property.

Thank you,

Jim Kubitz 12/15/2016

James Kubitz date
Vice President, Real Estate
Alaska Railroad Corporation

WMS Archive File Name: 16 APP Reserve

WMS WATERCOURSE MAPPING SUMMARY

Per the requirements for watercourse verification outlined in Project Management and Engineering Operating Policy and Procedure #8 and Planning Department Operating Policy and Procedure #1 (*effective June 18, 2007*), MOA Watershed Management Services has inspected the following location for the presence or absence of stream channels or other watercourses, as defined in Anchorage Municipal Code (21.35).

- Project Case Number or Subdivision Name: 2nd and Christensen Site Plan Application
- Project Location, Tax ID, or Legal Description: 002-071-27-000, 001-021-07-000
- Project Area (if different from the entire parcel or subdivision): Project Area includes only portion of parcel # 00102107 shown on the attached figure. -KBC

In accordance with the requirements and methods identified, WMS verifies that this parcel, project area, or application:

________ **<u>DOES NOT</u>** contain stream channels and/or drainageways, as identified in WMS field or archival mapping information.*

X KBC **<u>DOES</u>** contain stream channels and/or drainageways ***<u>AND</u>*** these are located and identified on submittal documents in general ~~congruence with~~ WMS field and archival mapping information.
*New or additional mapping **<u>IS NOT REQUIRED</u>**.**

________ Contains stream channels and/or drainageways ***<u>BUT</u>*** one or more streams or other watercourses:

- are ***<u>NOT</u>*** shown on submittal documents, or
- are ***<u>NOT</u>*** depicted adequately on submittal documents for verification, or
- are ***<u>NOT</u>*** located or identified on submittal documents in general congruence with WMS field and archival mapping information.

*New or additional mapping **<u>IS REQUIRED</u>** and must be re-submitted for further review and verification.**

________ Presence of stream channels and/or drainageways is unknown ***<u>AND</u>*** field verification is not possible at this time. WMS will verify as soon as conditions and prioritized resources allow.

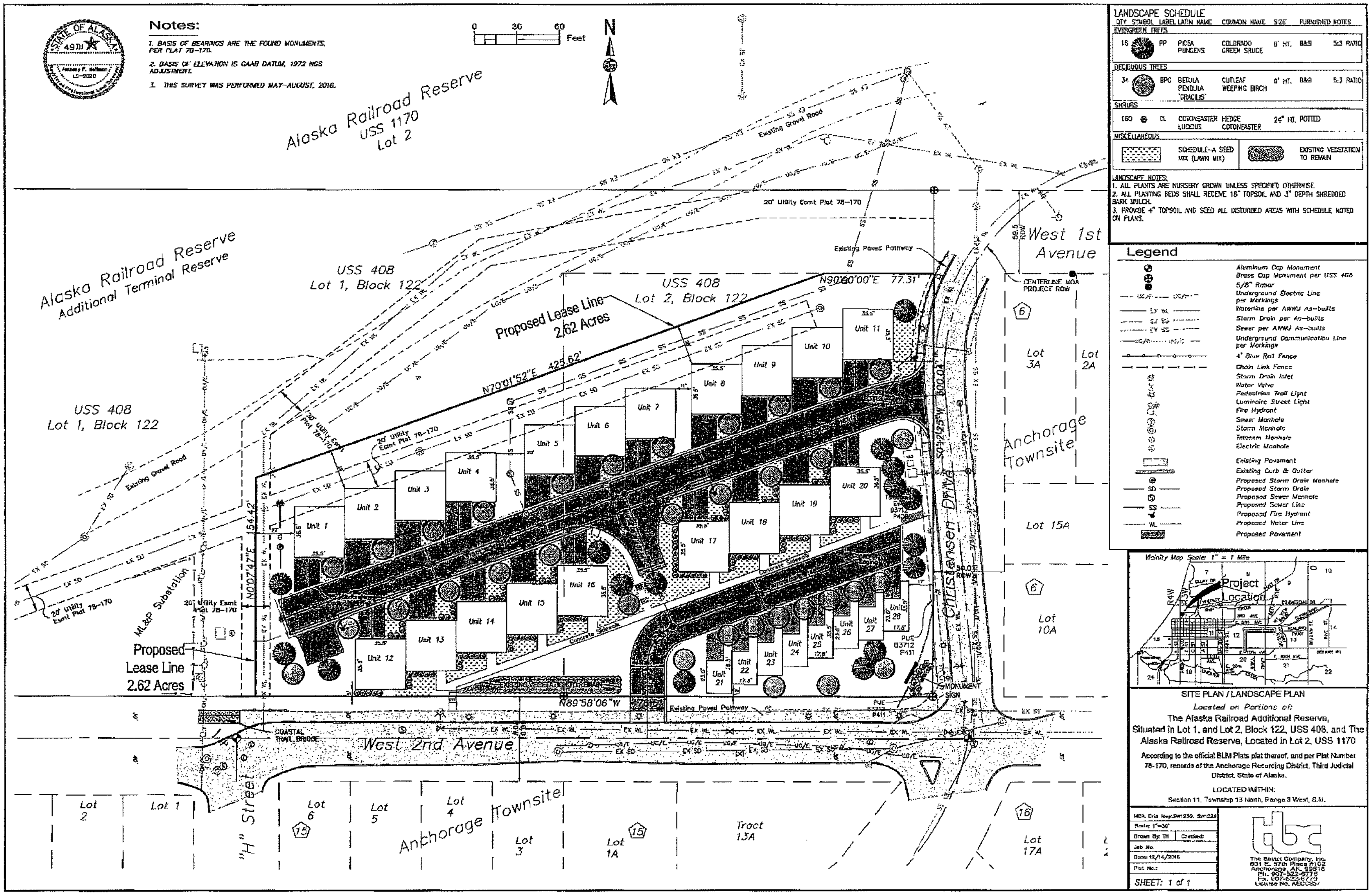
** Streams omitted in error by WMS or others remain subject to MOA Code and must be shown in new mapping upon identification of the error.*

ADDITIONAL INFORMATION:

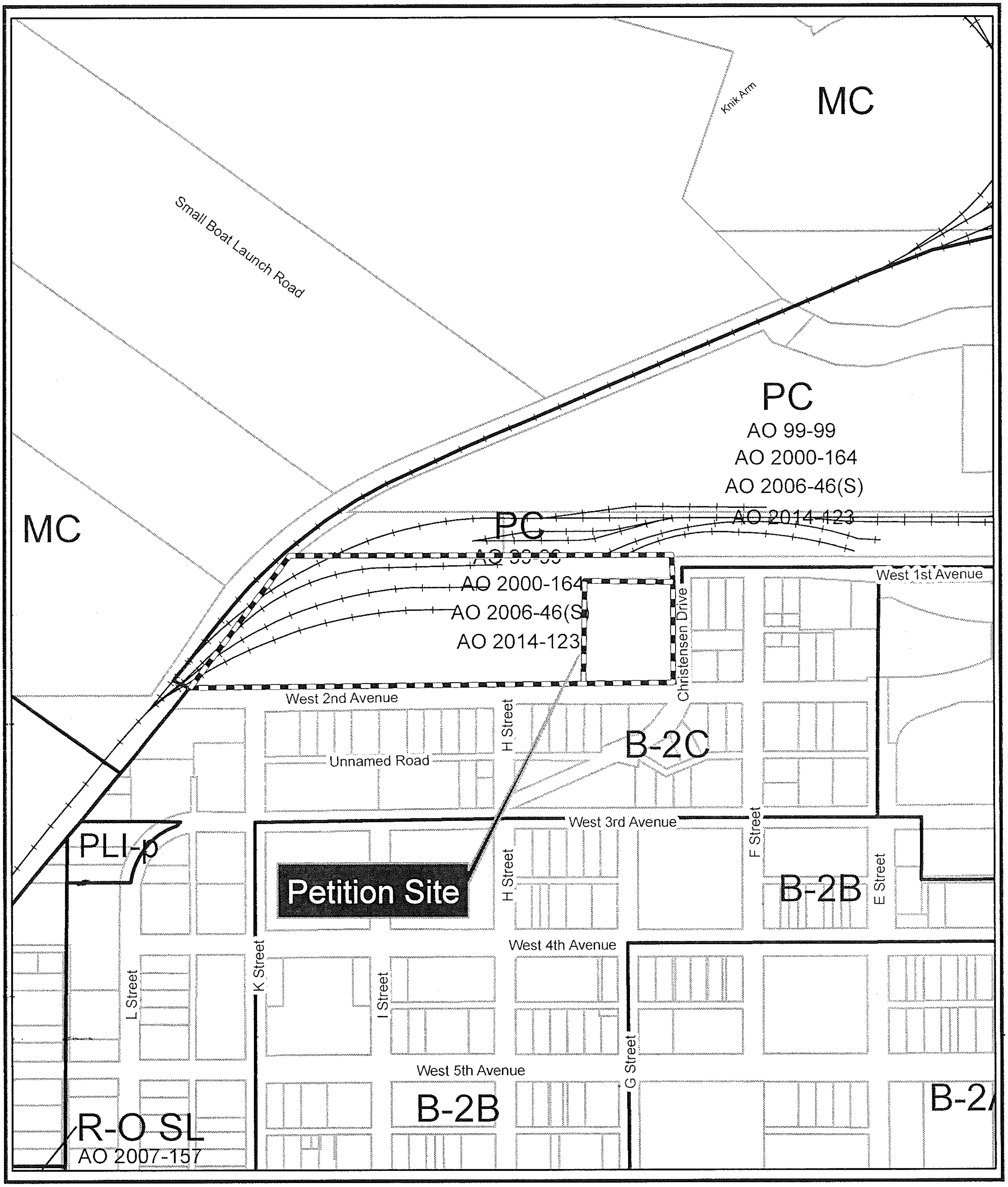
☐Y	☒N	WMS written drainage recommendations are available.	☐Preliminary	☐Final
☐Y	☒N	WMS written field inspection report or map is available.	☐Preliminary	☐Final
☐Y	☒N	Field flagging and/or map-grade GPS data is available.		

Inspection Certified By: [signature]

Date: 12/19/16


Notes:
1. BASIS OF BEARINGS ARE THE FOUND MONUMENTS PER PLAT 78-170.
2. BASIS OF ELEVATION IS GAAB DATUM, 1972 NGS ADJUSTMENT.
3. THIS SURVEY WAS PERFORMED MAY-AUGUST, 2016.
STATE OF ALASKA 49th
Feet
N
Alaska Railroad Reserve USS 1170 Lot 2
Alaska Railroad Reserve Additional Terminal Reserve
USS 408 Lot 1, Block 122
USS 408 Lot 2, Block 122
Proposed Lease Line 2.62 Acres
N70°01'52"E 425.62'
N90°00'00"E 77.31'
N00°07'47"E 154.42'
N89°58'06"W
S01°12'25"W 300.04'
20' Utility Esmt Plat 78-170
Existing Gravel Road
Existing Paved Pathway
ML&P Substation
Unit 1
Unit 2
Unit 3
Unit 4
Unit 5
Unit 6
Unit 7
Unit 8
Unit 9
Unit 10
Unit 11
Unit 12
Unit 13
Unit 14
Unit 15
Unit 16
Unit 17
Unit 18
Unit 19
Unit 20
Unit 21
Unit 22
Unit 23
Unit 24
Unit 25
Unit 26
Unit 27
Unit 28
Christensen Drive
West 1st Avenue
West 2nd Avenue
"H" Street
COASTAL TRAIL BRIDGE
CENTERLINE MOA PROJECT ROW
Anchorage Townsite
Lot 3A
Lot 2A
Lot 15A
Lot 10A
Lot 17A
Tract 13A
Lot 1A
Lot 3
Lot 4
Lot 5
Lot 6
Lot 1
Lot 2
LANDSCAPE SCHEDULE
QTY SYMBOL LABEL LATIN NAME COMMON NAME SIZE FURNISHED NOTES
EVERGREEN TREES
16 PP PICEA PUNGENS COLORADO GREEN SRUCE 6' HT. B&B 5:3 RATIO
DECIDUOUS TREES
34 BPC BETULA PENDULA 'GRACILIS' CUTLEAF WEEPING BIRCH 6' HT. B&B 5:3 RATIO
SHRUBS
160 CL COTONEASTER LUCIDUS HEDGE COTONEASTER 24" HT. POTTED
MISCELLANEOUS
SCHEDULE-A SEED MIX (LAWN MIX)
EXISTING VEGETATION TO REMAIN
LANDSCAPE NOTES:
1. ALL PLANTS ARE NURSERY GROWN UNLESS SPECIFIED OTHERWISE.
2. ALL PLANTING BEDS SHALL RECEIVE 18" TOPSOIL AND 3" DEPTH SHREDDED BARK MULCH.
3. PROVIDE 4" TOPSOIL AND SEED ALL DISTURBED AREAS WITH SCHEDULE NOTED ON PLANS.
Legend
Aluminum Cap Monument
Brass Cap Monument per USS 408
5/8" Rebar
Underground Electric Line per Markings
Waterline per AWWU As-builts
Storm Drain per As-builts
Sewer per AWWU As-builts
Underground Communication Line per Markings
4' Blue Rail Fence
Chain Link Fence
Storm Drain Inlet
Water Valve
Pedestrian Trail Light
Luminaire Street Light
Fire Hydrant
Sewer Manhole
Storm Manhole
Telecom Manhole
Electric Manhole
Existing Pavement
Existing Curb & Gutter
Proposed Storm Drain Manhole
Proposed Storm Drain
Proposed Sewer Manhole
Proposed Sewer Line
Proposed Fire Hydrant
Proposed Water Line
Proposed Pavement
Vicinity Map Scale: 1" = 1 Mile
Project Location
SITE PLAN / LANDSCAPE PLAN
Located on Portions of:
The Alaska Railroad Additional Reserve,
Situated in Lot 1, and Lot 2, Block 122, USS 408, and The Alaska Railroad Reserve, Located in Lot 2, USS 1170
According to the official BLM Plats plat thereof, and per Plat Number 78-170, records of the Anchorage Recording District, Third Judicial District, State of Alaska.
LOCATED WITHIN:
Section 11, Township 13 North, Range 3 West, S.M.
Scale: 1"=30'
Drawn By: TH
Checked:
Job No.
Date: 12/14/2016
Plat No.:
SHEET: 1 of 1
The Boutet Company, Inc.
601 E. 57th Place #102
Anchorage, AK 99518

2017-0017

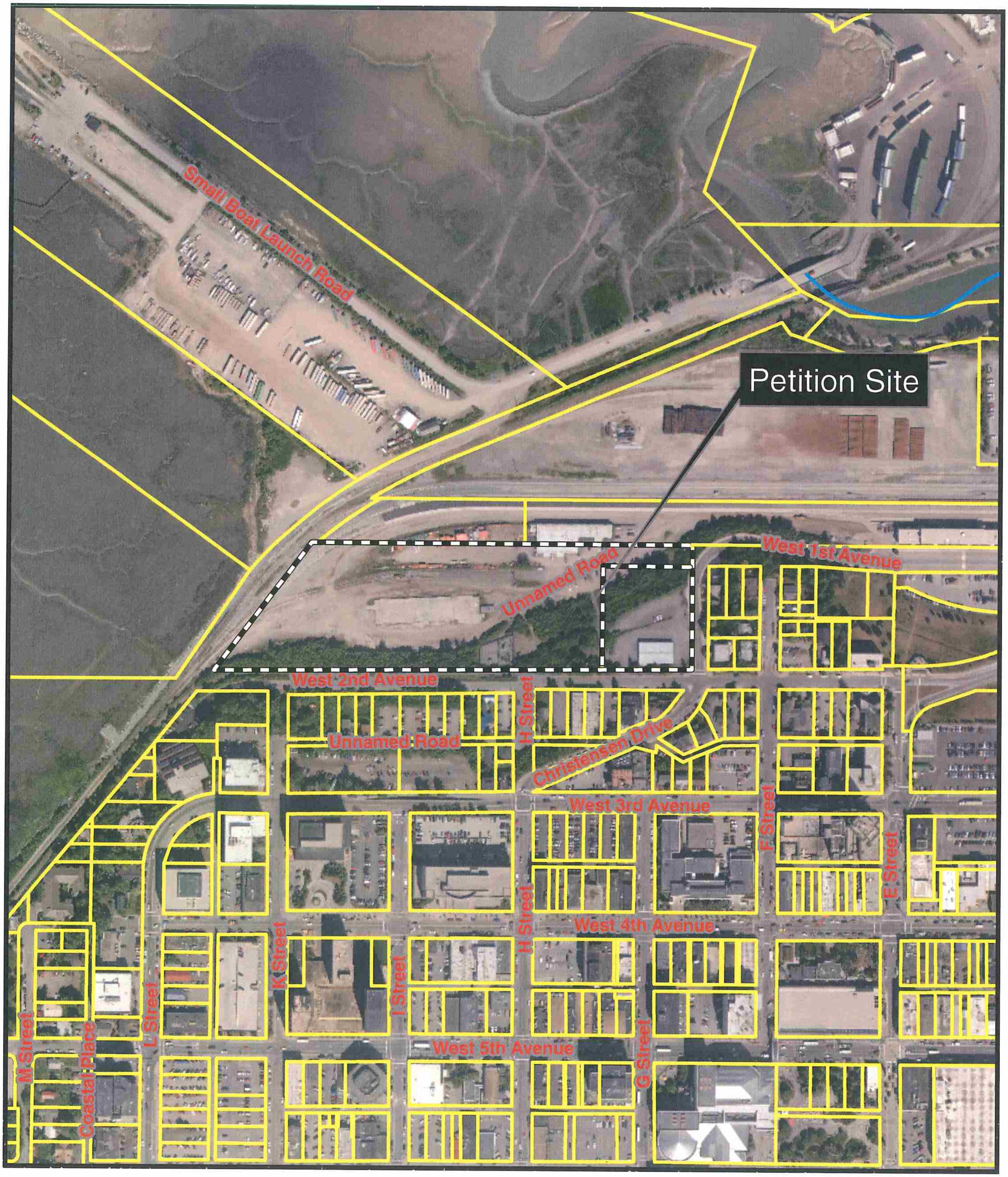


Municipality of Anchorage
Planning Department

Date: January 13, 2017

N

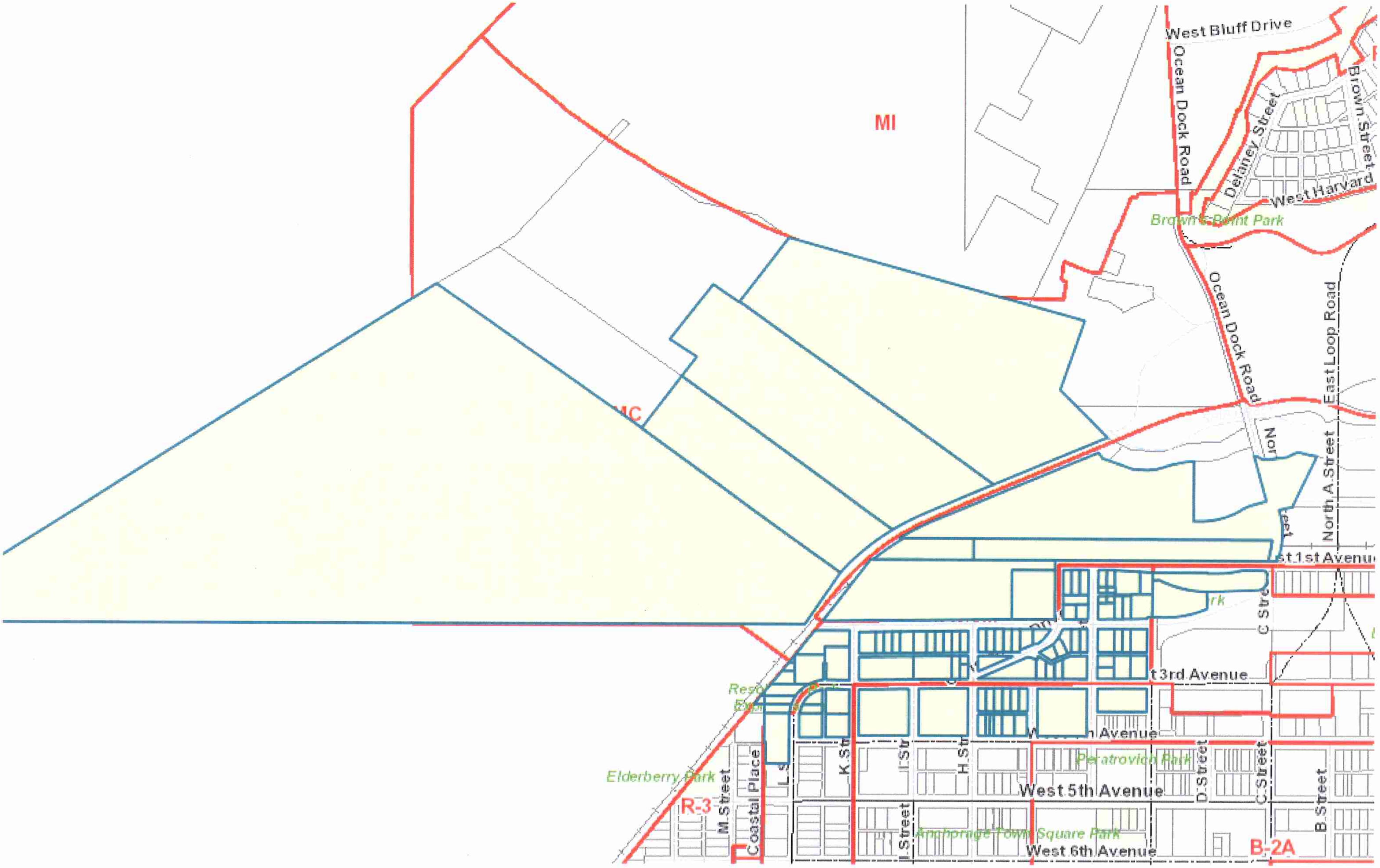
2017-0017



Municipality of Anchorage
Planning Department
Date: January 13, 2017

N

Anchorage



2017-0017 PHN map
Distance = 600' (98 pcls)