TOWN OF JACKSON PLANNING & BUILDING DEPARTMENT

TRANSMITTAL MEMO

Town of Jackson
☒Public Works/Engineering
☒Building
☐Title Company-*Wyoming Title & Escrow*
☒Town Attorney
☒Police

Joint Town/County
☒Parks and Recreation
☐Pathways

Teton County
☐Planning Division
☐Engineer
☐Surveyor- *Nelson*
☐Assessor
☐Clerk and Recorder
☐Road and Levee
☒Housing Authority

State of Wyoming
☐Teton Conservation
☐WYDOT
☐TC School District #1
☐Game and Fish
☐DEQ

Federal Agencies
☐Army Corp of Engineers

Utility Providers
☐Qwest
☐Lower Valley Energy
☐Bresnan Communications

Special Districts
☒START
☒Jackson Hole Fire/EMS
☐Irrigation Company

Date: May 1, 2018

Item #: P18-135 & 136

Planner: Tyler Valentine

Phone: 733-0440 ext. 1305

Fax: 734-3563

Email: tvalentine@jacksonwy.gov

Owner:
FSD Investments, LLC
PO Box 9879
Jackson WY 83002
307-413-4088
Groverjh@wyom.net

Applicant
Jorgensen Associates
PO Box 9550
Jackson, WY 83002
307-733-5150
rarmijo@jorgensenassociates.com

REQUESTS:

The applicant is submitting a request for a Hillside CUP for Physical Development on a parcel with slopes in excess of 10%. The property is located on 1255 W. Highway 22. In addition, the applicant seeks to amend their previously approved Development Agreement.

Please note: The applicant's previously approved Hillside CUP expired and all of the information pertaining to this new CUP is the same.

For questions, please call Tyler Valentine at 733-0440, x1305 or email to the address shown below. Thank you.

Please respond by: **May 15, 2018 (for Sufficiency)**
May 22, 2018 (with Comments)

RESPONSE: For Departments not using Trak-it, please send responses via email to:
tstolte@jacksonwy.gov

04/28/2018

FSD LLC

P.O. box 9879

Jackson, WY 83002

Charlie Schwartz & Eric Grove 307-413-4902

To Planning Dept. & Town Council:

FSD LLC is submitting a new hillside CUP for the Westview townhomes as the previously approved CUP has expired. All the info from the old CUP is still relevant and we are re-submitting all the documentation. In addition we are requesting that the approved development agreement be amended as needed regarding this CUP, the development plan etc.

As a reminder Westview is a 20 unit project with 16 deed restricted units.

Please see the attached documents for site plan and floor plan etc.

We apologize for this project taking longer than we had hoped and we appreciate your time needed to help us get this project off the ground. We also appreciate the water and sewer work the town did last fall of which we are still bonded for all the work that was done.

Thank you again for everyone's extra time on this,

Charlie Schwartz & Eric Grove

PLANNING PERMIT APPLICATION
Planning & Building Department
Planning Division

150 E Pearl Ave. | ph: (307) 733-0440
P.O. Box 1687 | fax: (307) 734-3563
Jackson, WY 83001 | www.townofjackson.com

For Office Use Only

Fees Paid_____________

Check # _____________ Credit Card_____________ Cash_____________

Application #s _____________ _____________ _____________

PROJECT.

Name/Description: Westview Townhomes

Physical Address: 125 West Highway 22

Lot, Subdivision: _____________ PIDN: 22-41-16-32-1-00-008

OWNER.

Name: FSD, Investments, LLC Phone: (307)413-4088

Mailing Address: P.O. Box 9879, Jackson, Wyoming ZIP: 83002

E-mail: groverjh@wyom.net

APPLICANT/AGENT.

Name: Jorgensen Associates, P.C. Phone: (307)733-5150

Mailing Address: P.O. Box 9550, Jackson, Wyoming ZIP: 83002

E-mail: rarmijo@jorgensenassociates.com

DESIGNATED PRIMARY CONTACT.

_______Owner X_______Applicant/Agent

TYPE OF APPLICATION. *Please check all that apply; see Fee Schedule for applicable fees.*

Use Permit
_______Basic Use
X_______Conditional Use
_______Special Use

Physical Development
_______Sketch Plan
_______Development Plan

Interpretations
_______Formal Interpretation
_______Zoning Compliance Verification

Relief from the LDRs
_______Administrative Adjustment
_______Variance
_______Beneficial Use Determination
_______Appeal of an Admin. Decision

Development Option/Subdivision
_______Development Option Plan
_______Subdivision Plat
_______Boundary Adjustment (replat)
_______Boundary Adjustment (no plat)

Amendments to the LDRs
_______LDR Text Amendment
_______Zoning Map Amendment
_______Planned Unit Development

PRE-SUBMITTAL STEPS. *Pre-submittal steps, such as a pre-application conference, environmental analysis, or neighborhood meeting, are required before application submittal for some application types. See Section 8.1.5, Summary of Procedures, for requirements applicable to your application package. If a pre-submittal step is required, please provide the information below. If you need assistance locating the project number or other information related to a pre-submittal step, contact the Planning Department.* ***If this application is amending a previous approval, indicate the original permit number.***

Pre-application Conference #: P15-084 Environmental Analysis #: ____________

Original Permit #: ____________ Date of Neighborhood Meeting: ____________

SUBMITTAL REQUIREMENTS. *Twelve (12) hard copies and one (1) digital copy of the application package (this form, plus all applicable attachments) should be submitted to the Planning Department.. Please ensure all submittal requirements are included. The Planning Department will not hold or process incomplete applications. Partial or incomplete applications will be returned to the applicant.*

Have you attached the following?

X **Application Fee.** Fees are cumulative. Applications for multiple types of permits, or for multiple permits of the same type, require multiple fees. See the currently adopted Fee Schedule in the Administrative Manual for more information.

X **Notarized Letter of Authorization.** A notarized letter of consent from the landowner is required if the applicant is not the owner, or if an agent is applying on behalf of the landowner. If the owner is a partnership or corporation, proof that the owner can sign on behalf of the partnership or corporation is also required. Please see the Letter of Authorization template in the Administrative Manual for a sample.

X **Response to Submittal Checklist.** All applications require response to applicable review standards. These standards are outlined on the Submittal Checklists for each application type. If a pre-application conference is held, the Submittal Checklists will be provided at the conference. If no pre-application conference is required, please see the Administrative Manual for the applicable Checklists. The checklist is intended as a reference to assist you in submitting a sufficient application; submitting a copy of the checklist itself is not required.

FORMAT.

The main component of any application is demonstration of compliance with all applicable Land Development Regulations (LDRs) and Resolutions. The submittal checklists are intended to identify applicable LDR standards and to outline the information that must be submitted to sufficiently address compliance with those standards.

For some submittal components, minimum standards and formatting requirements have been established. Those are referenced on the checklists where applicable. For all other submittal components, the applicant may choose to make use of narrative statements, maps, drawings, plans and specifications, tables and/or calculations to best demonstrate compliance with a particular standard.

Note: Information provided by the applicant or other review agencies during the planning process may identify other requirements that were not evident at the time of application submittal or a Pre-Application Conference, if held. Staff may request additional materials during review as needed to determine compliance with the LDRs.

Under penalty of perjury, I hereby certify that I have read this application and associated checklists and state that, to the best of my knowledge, all information submitted in this request is true and correct. I agree to comply with all county and state laws relating to the subject matter of this application, and hereby authorize representatives of Teton County to enter upon the above-mentioned property during normal business hours, after making a reasonable effort to contact the owner/applicant prior to entering.

____________ JANUARY 6, 2016

Signature of Owner or Authorized Applicant/Agent Date

Reed Armijo Principal

Name Printed Title

LETTER OF AUTHORIZATION

FSD Investments, LLC ,"Owner" whose address is: ____________

P.O. Box 9879, Jackson, WY 83002

(NAME OF ALL INDIVIDUALS OR ENTITY OWNING THE PROPERTY)

Charlie Schwartz and Eric Grove, as the owner of property more specifically legally described as: 1255 West Hwy 22, Jackson, WY

(If too lengthy, attach description)

HEREBY AUTHORIZES Jorgensen Associates, P.C. as agent to represent and act for Owner in making application for and receiving and accepting on Owners behalf, any permits or other action by the Town of Jackson, or the Town of Jackson Planning, Building, Engineering and/or Environmental Health Departments relating to the modification, development, planning or replatting, improvement, use or occupancy of land in the Town of Jackson. Owner agrees that Owner is or shall be deemed conclusively to be fully aware of and to have authorized and/or made any and all representations or promises contained in said application or any Owner information in support thereof, and shall be deemed to be aware of and to have authorized any subsequent revisions, corrections or modifications to such materials. Owner acknowledges and agrees that Owner shall be bound and shall abide by the written terms or conditions of issuance of any such named representative, whether actually delivered to Owner or not. Owner agrees that no modification, development, platting or replatting, improvement, occupancy or use of any structure or land involved in the application shall take place until approved by the appropriate official of the Town of Jackson, in accordance with applicable codes and regulations. Owner agrees to pay any fines and be liable for any other penalties arising out of the failure to comply with the terms of any permit or arising out of any violation of the applicable laws, codes or regulations applicable to the action sought to be permitted by the application authorized herein.

Under penalty of perjury, the undersigned swears that the foregoing is true and, if signing on behalf of a corporation, partnership, limited liability company or other entity, the undersigned swears that this authorization is given with the appropriate approval of such entity, if required.

OWNER: ____________

(SIGNATURE) (SIGNATURE OF CO-OWNER)

Title: ____________

(if signed by officer, partner or member of corporation, LLC (secretary or corporate owner) partnership or other non-individual Owner)

STATE OF Wyoming)
)SS.
COUNTY OF Teton)

The foregoing instrument was acknowledged before me by Charlie Schwartz this 5 day of January, 2016.

WITNESS my hand and official seal.

(Seal)

____
(Notary Public)
My commission expires: 6/11/19

Amy Glasgow - Notary Public
County of Lincoln
State of Wyoming
My Commission Expires: 06/11/2019

150 E Pearl Ave.
P.O. Box 1687
Jackson, WY 83001
ph: (307) 733-0440
fax: (307) 734-3563
www.townofjackson.com

Application Submittal Checklist for a
CONDITIONAL USE PERMIT (CUP)
Planning & Building Department
Planning Division

APPLICABILITY. *This checklist should be used when submitting an application for a **Conditional Use Permit.***

When is a Conditional Use Permit required?
Section 6.1.1 of the LDRs contains the Use Schedule for all zones. Allowed uses that require a Conditional Use Permit are denoted with a "C." You can also determine whether a Conditional Use Permit is required by referencing Subsection C of the applicable zone.

Do I need a Pre-Application Conference first?
Yes, a Pre-Application Conference is required.

FINDINGS FOR APPROVAL. *The application shall include a narrative statement addressing each of the applicable Findings for Approval, found in **Section 8.4.2, Conditional Use Permit.***

A conditional use permit shall be approved upon finding the application:

1. Is compatible with the desired future character of the area; and
2. Complies with the use specific standards of Division 6.1: Allowed Uses and the zone; and
3. Minimizes adverse visual impacts, and;
4. Minimizes adverse environmental impacts; and
5. Minimizes adverse impacts from nuisances; and
6. Minimizes adverse impacts on public facilities; and
7. Complies with all other relevant standards of these LDRs and all other Town Ordinances; and
8. Is in substantial conformance with all standards or conditions of any prior applicable permits or approvals.

GENERAL INFORMATION.

X _____ **Response to Pre-Application Conference Summary Checklist.** During the pre-application conference, you will be provided with a summary and checklist of applicable LDR standards and requirements that must be addressed for a sufficient application.

PO Box 9550 · 1315 HWY 89 S., Suite 201
Jackson, WY 83002
PH: 307.733.5150
www.jorgeng.com

Westview Townhomes
Conditional Use Permit
Findings for Approval
January 6, 2016

This Conditional Use Permit (CUP) is required for the proposed Westview Townhomes projects located at 1255 West Highway 22 in Jackson, Wyoming. This CUP is required based upon of the Town of Jackson Land Development Regulations (LDR) *Article 2.3.4 Urban Residential (R) B. Physical Development 9. Natural Hazards to Avoid – Steep Slopes, Lots with average cross slopes in excess of 10%* as this parcel has slopes that will be impacted by the development in excess of 10%.

Section 8.4.2 Conditional Use Permit, C. Findings of Approval indicates that a CUP shall be approved based upon a set of findings for approval. The findings for each item listed in the set of findings of approval included in the LDR's are summarized below.

1. ***Is consistent with the desired future character of the area:***
The proposed Westview Townhomes are in Character District 4 – Midtown, Subarea 4.2 – Northern Hillside of the Comprehensive Plan. It meets the Complete Neighborhood definition of defined character and high quality design for the future of providing 2 to 3 stories with single family townhomes. The location offers access to multi-modal choices including START and pathways and close access to grocery shopping, restaurants and bars, banking, and the post office.

2. ***Complies with the specific standards of Div. 6.1:***
The Westview Townhomes are Attached Single Family Residential Units in the AC Zone. The zoning is being changed to UR-PUD. Attached Single Family Residential Units are an allowed use requiring a Basic Use Permit according to Div. 6.1 of the LDR's.

3. ***Minimizes adverse visual impacts:***
The site presently consists of an older metal building and an exposed, man-made slope. The project site as recently been used as a rental car facility with a significant number of vehicles parked on the upper and lower lots and previously has been a gas station and convenient store. The project will remove the building and incorporate aesthetically pleasing residential townhomes with a landscaped berm along the frontage of WY22 and landscaping interspersed throughout the residences. This development is designed to improve upon the current use and will complement the existing landscape. It will not block or interfere with any views and will improve upon the existing visual impacts from neighboring properties and WY22.

4. ***Minimizes adverse environmental impact:***
Biota Research and Consulting has prepared Environmental Analysis for the parcel and no negative impacts to wildlife are expected to result from the proposed action. Potential adverse impacts have been considered and addressed with the proposed site plan.

5. ***Minimizes adverse impacts from nuisances:***
As a planned single family residential neighborhood, there are not anticipated to be any nuisances. Noise and other impacts are anticipated to be far less than the adjacent commercial

operations (rental car business, fuel storage yard, etc.).

6. ***Impact on Public Facilities:***
It is not anticipated that the Westview Townhomes will have adverse impacts on public facilities. The site is served by Town sewer. The applicant is coordinating with the Town Engineer to ensure adequate downstream capacity. The applicant is coordinating with the Town Engineer to identify the necessary water service improvements to ensure available capacity to serve the development. Storm water will be managed in accordance with Town requirements. The location of the development will minimize traffic impacts as the site is served by pathways and is a walkable distance from START service and several basic services such as grocery store, restaurant and bar, banks, etc. One of the existing accesses is being eliminated to reduce impacts on WY 22 from the project site. Structures will meet all required codes and will not have adverse impacts on police, fire, and EMT facilities.

7. ***Complies with other relevant standards of these Land Development Regulations:***
The proposed Westview Townhomes project complies with all other applicable provisions of the Land Development Regulations for use, layout, and general development characteristics.

8. ***Is in substantial conformance with all standards or conditions of any prior applicable permits or approval:***
The applicant is not aware of any prior standards or conditions of any prior applicable permits or approvals for this parcel.

JORGENSEN
It's About People, Trust and Know How
PO Box 9550 · 1315 HWY 89 S., Suite 201
Jackson, WY 83002
PH: 307.733.5150
| www.jorgeng.com

Westview Townhome Traffic Impact Statement
Jackson, Wyoming

Prepared by: Jorgensen Associates, PC
Project No. 09040

I. INTRODUCTION

The proposed Westview Townhomes development will be a 1.1 acre residential development located on U.S. Highway 22 within the Jackson town limits. The parcel is approximately 1,030 ft. from the U.S. Highway 89 and Wyoming Highway 22 intersection. The site will consist of twenty residential units in 6 buildings. Four of the six buildings will have 4 units-3 bedrooms per unit in each building and the upper two buildings will consist of 2 units each and have 3 bedrooms per unit. Access to the site will be provided in two existing locations; one on Wyoming Highway 22 and the other using the Search and Rescue road.

This statement focuses on the proposed project, previous use comparison, estimated traffic generation for pervious uses vs. proposed use, and mitigation measures. This report will identify and discuss any upgrades to the study area that may be necessary due to the impacts of the development.

All data, calculations, and worksheets can be found using the Institute of Transportation Engineer's (ITE) *Trip Generation, 7th Edition.*

II. EXISTING LAND USE AND TRANSPORTATION SYSTEM

EXISTING LAND USE

The existing land of the Westview Townhome development proposal consists of one single lot currently occupied by a rental car company building and fleet parking for [xx vehicles or xx square feet of fleet parking].. It has two frontage accesses to Wyoming Highway 22 and a third access to the Batch Plant Road that services the Search and Rescue Facility. This property is surrounded by a variety of land uses including residential, commercial and state owned lands.

Previously, the land has been used as a Gas Station/Convenience Store, Small Grocery Store (Choice Meats), and currently the Rental Car facility.

EXISTING ROADWAYS AND PATHWAYS

Wyoming Highway 22 is a State Primary Highway in the Wyoming Department of Transportation (WYDOT) system. Wyoming Highway 22 serves as Jackson's main connector to Wilson, Teton Village, and the Victor and Driggs area of Idaho. Along the frontages of the Westview Townhomes development site Highway 22 has a four lane section with two lanes in each direction, curb and gutter on both sides, and no shoulders.

The Batch Plant Road that services the Search and Rescue facility is a two lane 24 ft. wide gravel road with a paved approach to the highway.

There is a 6' pathway on both sides of the highway in the area of this study so bicycles do not share the roads with other vehicles. The pathway along the westbound lane currently ends near Spring Gulch Road, which is 2,250 ft north of the closest ingress/egress to the site. There is little pedestrian activity and the closest pedestrian crossing is at the U.S. Highway 89 and Highway 22 intersection. This crossing allows direct convenient access to Cutty's (a popular restaurant/bar), Albertsons, Wells Fargo Bank, a nearby Post Office, Pizza Hut, Lucky's (grocery store), and various other local customer friendly businesses.

EXISTING TRAFFIC

The existing traffic in this analysis is estimated from previously existing land uses on this parcel. Trip generations from these land uses are compared to the proposed development using the Institute of Transportation Engineer's (ITE) *Trip Generation, 7th Edition.*

Jorgensen would need to perform a physical count of traffic to obtain an understanding of existing traffic conditions but we feel this analysis will give an accurate feel for how traffic would flow with the proposed conditions. The AM and PM peak hours for each day were then averaged to find the Background Design Hour Volumes.

III. PROPOSED CONDITIONS

The Westview Townhomes development will be comprised of twenty residential units in 6 buildings. 1 of the buildings containing 4 units will be deed restricted employee or affordable housing with the remainder of the buildings being market rate units. Access to the site will is proposed from two directions.

Currently the proposed site has two accesses to U.S. Highway 22, one approximately 1100 feet from the Highway 89/Highway 22 intersection and a second approximately 1230 feet from the same intersection. The Westview Townhomes project will consolidate these accesses by eliminating the one nearer to Highway 89 and formalizing the further one. This upgraded access will be as far from the intersection as feasible.

The second access to this site will be from Batch Plant Road (Search and Rescue Road) and its intersection with Highway 22. This access will be used to reach the top two buildings, buildings 5 and 6.Improvements will be made to the Batch Plant Road intersection with Highway 22 to improve its functionality. The purpose of eliminating one of the Highway 22 accesses and using the existing Batch Plant Road will control traffic onto a public roadway while maintaining safety, capacity, and function of the roadway as stated in Division 7.6 Transportation Facility Standards of the 2015 Town of Jackson Comprehensive Plan.

BACKGROUND TRAFFIC

Background traffic refers to the current existing traffic and the future traffic that is anticipated without the proposed development and using the previous land uses. For this study the background traffic is calculated using the size of the existing building and the uses described in the Institute of Transportation Engineer's (ITE) *Trip Generation, 7th Edition.*

TRIP GENERATION

This report uses the Institute of Transportation Engineer's (ITE) *Trip Generation, 7th Edition,* to calculate the traffic generated by the proposed Westview Townhomes. *Trip Generation* provides trip generation rates for a myriad of land uses and is considered the standard for trip generation calculations in the traffic professions. To estimate the traffic generated by the development, the proposed use is matched to a Land Use type in *Trip Generation*. Table 1 shows the best matched ITE Land Uses.

Table 1 – Land Use

Proposed Land Use	**ITE Land Use**	**ITE Code**
Westview Townhome	Residential Condominium/Townhouse	230
Gas Station/Convenience Store	Convenience Market with Gasoline Pumps	853
Small Grocery Store	Supermarket	852
Rental Car Facility	None	None

The table below show the anticipated trips generated by the 20 residential units in Westview Townhome development.

Table 2 – Westview Townhome Trip Generation- Per Dwelling Unit

			Directional Distribution			
Analysis Period		*Calc'd Trips*	*Entering*	*Exiting*	*Entering*	*Exiting*
ADT	Weekday	59	50%	50%	29	29

Portions of these generated trips were assigned to each building for the potential of assigning each building's trips to one of the accesses. The trip allocation was made based on the percentage of total units in each building. For example, if Building 1 had 10% of the total residential units in the development it was assigned 10% of the generated trips. The trip allocation calculations were then rounded up to ensure no building was responsible for a fraction of a trip and to add a level of conservatism to the analysis.

Table 3 – Westview Townhome Adjusted Trip Generation

Analysis Period		*Entering*	*Exiting*
ADT	Weekday	3	3

This study uses the traffic volumes presented in Table 3 as the traffic generated by Westview Townhome.

Table 4 displays the projected traffic generated by The Gas Station/Convenience Store based on the existing size of the building that is currently on the property of 3,200 sqft

Table 4 – Gas Station/Convenience Store Trip Generation – Per 1,000 sqft

			Directional Distribution			
Analysis Period		*Calc'd Trips*	*Entering*	*Exiting*	*Entering*	*Exiting*
ADT	Weekday	2538	50%	50%	1269	1269

Table 5 displays the projected traffic generated by The Small Grocery Store.

Table 5 – Small Grocery Store (Butcher Shop)-Per 1,000 sqft

			Directional Distribution			
Analysis Period		*Calc'd Trips*	*Entering*	*Exiting*	*Entering*	*Exiting*
ADT	Weekday	1733	50%	50%	867	867

Table 6 displays the projected traffic generated by the Rental Car Facility.

Table 6 – Rental Car Facility-Vehicles currently on site

			Directional Distribution			
Analysis Period		*Calc'd Trips*	*Entering*	*Exiting*	*Entering*	*Exiting*
ADT	Weekday	160	50%	50%	80	80

TRIP DISTRIBUTION

The traffic generated by Westview Townhomes will use the access point off Highway 22 for buildings 1-4 and Batch Plant Road to access buildings 5 & 6 to enter and exit the site. Buildings 5 & 6 will be on a tier and no access from below, i.e. the Highway 22 access, will be available for these buildings. The distribution of traffic will be approximately 20% onto the Batch Plant Road access onto Highway 22, which is about 620 ft from the proposed direct access onto Highway 22 to which the other 80% will use as an access point.

The traffic generated with the Westview Townhome is the least amount of traffic when compared to all of the existing/past uses for the site.

TOTAL TRAFFIC

Typically the total traffic for the study intersections is found by adding the generated and distributed trips to the background Design Hourly Volume (DHV). This study requires additional steps and is not covered in this analysis.

IV. Conclusions and Mitigation Measures

The effects of the proposed Westview Townhome development will not increase traffic volumes over existing uses or most previous uses on the site. Expected traffic volumes for the proposed development fall below the existing and previous uses. Improvements to the existing accesses will be incorporated in to the design to allow for stacking of vehicles leaving the site and the smooth entrance and exit of vehicles.

The location of this development allows for the use of alternative transportation methods. The site is located within walking distance of two grocery stores, a popular bar/restaurant, convenience/liquor store, two banks, and other shopping possibilities. The site is also within walking distance of START bus stops going in both the east (in to Town) and west (towards Wilson and Teton Village) directions. The site's proximity to the Main Jackson post office will help eliminate the single occupancy vehicle trips usually associated with going to pick up the mail.

As stated previously, the existing east access onto Highway 22 will be eliminated thus easing traffic flow onto the highway.

The existing traffic signals at the Spring Gulch Road/ Highway 22 intersection as well as the U.S. Highway 89/ Highway 22 will be useful in providing gaps in the flow of traffic on Highway 22 for the proposed traffic from the townhomes to enter the highway.

Westview Townhomes
Town Of Jackson
Jackson, Wyoming

PROPOSED LAND USES

Land se: Westview Townhouse-Cars and Trucks
ITE Land se Category: **Residential Condominium/Townhouse**
ITE Land se Code: **230**
Independent Variable: Dwelling nits
Value: 6

Analysis Period		*Average Rate or Equation*	*Calc'd Trips*	*Directional Distribution* *Entering*	*Exiting*	*Entering*	*Exiting*
ADT	Weekday	Ln(T)= .85 (Ln( ))+2.55	59	50%	50%	29	29

EXISTING LAND USES

Land se: as Station/Convenience Store - 1000 sqft ross Floor Area
ITE Land se Category: **Retail**
ITE Land se Code: **853**
Independent Variable: 1000 sqft ross Floor Area
Value: 846

Analysis Period		*Average Rate or Equation*	*Calc'd Trips*	*Directional Distribution* *Entering*	*Exiting*	*Entering*	*Exiting*
ADT	Weekday	Not iven so = 3( )	2538	50%	50%	1269	1269

Land se: Small rocery Store
ITE Land se Category: **Retail**
ITE Land se Code: **852**
Independent Variable: 1000 sqft ross Floor Area
Value: 5 Ad usted based on single item sold (Butcher Shop)

Analysis Period		*Average Rate or Equation*	*Calc'd Trips*	*Directional Distribution* *Entering*	*Exiting*	*Entering*	*Exiting*
ADT	Weekday	T = 66.95( ) + 1391.58	1733	50%	50%	867	867

Land se: Rental Car Facility
ITE Land se Category: **None**
ITE Land se Code: **None**
Independent Variable: Vehicles
Value: 80

Analysis Period		*Average Rate or Equation*	*Calc'd Trips*	*Directional Distribution* *Entering*	*Exiting*	*Entering*	*Exiting*
ADT	Weekday	T = 2 ( )	160	50%	50%	80	80

-ENVIRONMENTAL ANALYSIS-

WESTVIEW TOWN HOMES PROJECT,

JACKSON, WYOMING

Prepared For

JORGENSEN ENGINEERING

P.O Box 9550, Jackson, WY 83001

Prepared By

research & consulting inc.

Biota

P. O. Box 8578 • 140 E Broadway, Suite 23, Jackson, WY 83002; voice: (307) 733-4216 • fax: (307) 733-1245

January 6, 2016

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TABLES

ENVIRONMENTAL ANALYSIS

WESTVIEW TOWN HOMES PROJECT, JACKSON, WYOMING

INTRODUCTION AND BACKGROUND

Biota Research and Consulting, Inc. (Biota) prepared an Environmental Assessment (EA) of proposed development within the Westview Town Homes property. The EA was requested by Jorgensen Engineering, agent for the landowner. Information provided in this document is required by the Town of Jackson Planning Department per Section 5.4.1 of the Jackson Land Development Regulations, Natural Hazard Protection Standards, because the project area is located within a designated Steep Slope area. The EA documents the extent of wildlife use occurring on the property and potential adverse impacts to wildlife and habitat resulting from the project.

LOCATION, PHYSIOGRAPHY, AND HISTORIC LAND USES

The property is located within the Town of Jackson in Teton County, Wyoming (T41N, R116W, Section 32; Appendix 1- Exhibit 1). The 1.1 acre project area is situated on the lower slopes of East Gros Ventre Butte approximately 1,000 feet north of the "Y" intersection of US Highway 89 (West Broadway) and Wyoming Highway 22. The terrain of the property is in a largely disturbed condition, although a narrow strip of native vegetation persists along the upper sloped area. Elevations range between 6,160 and 6,240 feet, and drainage is generally north to south. Most of the project area show evidence of historic land altering activities associated with historic development and commercial uses.

SURFACE HYDROLOGY AND WETLANDS

No surface hydrologic features or wetlands are present within the project area.

VEGETATIVE COVERTYPES

Vegetative covertypes consist of primarily existing disturbed areas along with a small area of the xeric shrub covertype (Appendix 1-Exhibit 2). The Land Development Regulations ranked the relative values of mesic and non-mesic covertypes by assigning each an ordinal value ranging from 1 (lowest value) to 10 (highest value). These criteria include wildlife species diversity, abundance and distribution of habitats, wildlife species using given habitats, and the degree of alteration associated with the habitats. Disturbed areas are not ranked under the relative wildlife habitat value criteria. Acreages, percent occurrence, and relative habitat values of each covertype are summarized in Table 1.

Table 1. Acreages, percent occurrence, and ordinal ranking of vegetative covertypes within the Westview Town Homes project area.

Vegetative Covertypes	Acres	%	Ranking
Xeric Shrub	0.06	5	3
Disturbed - Grassland	0.33	30	
Disturbed - Impervious Surface	0.71	65	
Total	**1.1**	**100**	**NA**

Xeric Shrub

The xeric shrub covertype comprises 0.06 acres of the project area, and is located on the upper slopes in the only location that has not experienced historic land disturbance activities. Scattered low-growing sagebrush and rabbitbrush shrubs occur here in combination with invasive plant species. The xeric shrub covertype has been given an ordinal ranking of 3.

Disturbed

Disturbed land comprises 95% (1.04 acres) of the project area and includes 0.71 acres of impervious surface or areas lacking vegetative cover, and areas revegetated in grass and noxious weed (0.33 acres). Disturbed areas appear primarily associated with the actions taken to flat areas for commercial uses. The Land Use Regulations assigned no ordinal ranking to disturbed areas because of their typical lack of foraging and cover habitat for wildlife.

WILDLIFE SPECIES OF SPECIAL CONCERN

Vegetative communities within the project area represent habitat for a several species of birds and mammals, some of which have been classified as species of special concern (SSCs) in the Jackson-Teton County Comprehensive Plan and Land Use Regulations (2015). In addition, migratory birds and amphibians are addressed in this section because they are considered sensitive species and are often used as ecological indicators by management agencies. Wildlife species of special concern that are or might be present within the project area are discussed below.

Bald Eagle

Teton County Land Development Regulations protect nesting bald eagles by prohibiting development within 660 feet of standing/occupied, active, or inactive nests, and also protects known perch and roost trees regarded as crucial winter habitat (Section 5.2.1 G6a & b). No bald eagle nests are within 660 feet of the project area. The High School Hill bald eagle nest is located on the wooded north face of High School Butte, approximately 2,900 feet west of the project area. Although these nesting birds, their offspring, and perhaps other bald eagles can be expected in the vicinity, they are not expected to use the project area itself due to the high percentage of disturbed ground, the high volume transportation corridor neighboring the project area, and surrounding land uses. Observations of eagles in this area are primarily linked to their movements to and from foraging habitat associated with Spring Creek, Flat Creek, or nearby mule deer winter ranges when carrion from winter-killed animals may be present. There are no important bald eagle habitat features present within the project area.

Teton County Land Development Regulations protect nesting bald eagles by prohibiting development within 660 feet of standing/occupied, active, or inactive nests, and also protects known perch and roost trees regarded as crucial winter habitat (Section 5.2.1 G6a & b). No bald eagle nests are within 660 feet of the project area.

Raptors

One general group of raptors involving shrub-grassland species is expected to be present along the undeveloped slopes adjacent to the project area. Shrub-grassland raptors primarily exploit open shrub- and grass-dominated communities, and use trees for perching and nesting. It is likely that red-tailed hawks, great horned owls, and American kestrels use the project area in a very limited capacity, and in conjunction with adjacent areas. No evidence of raptors presently or historically nesting within the project area was discover, and there are no natural roosting or perching structures located on the property.

MULE DEER

The entire project area has been generally mapped as crucial mule deer winter range by the Wyoming Game and Fish Department (Appendix 1-Exhibit 3). The mapping depicts the entire south end of East Gros Ventre Butte as crucial winter range including West Broadway, regardless of whether or not development is present. In reality, however, most of the project area represents little, if any, habitat to wintering mule deer due to the absence of browse species, the lack of thermal cover, the disturbed nature of the site, and its location amidst surrounding development. Past land uses have resulted in the site being largely denuded of native shrubs and replaced primarily with bare ground, or grasses and invasive species. Relatively high levels of commercial use have and continue to occur within and in the vicinity of the project area.

Teton County Land Development Regulations protect crucial mule deer winter range and migration corridors (Section 5.2.1 G2a & b) and state:

> *No physical development, use, development option, or subdivision shall occur within crucial mule deer migration routes, unless the applicant can demonstrate that if can be located within the mule deer migration route in such a way that it will not detrimentally affect the ability of mule deer to migrate from their summer ranges to their crucial winter ranges.*
>
> *No physical development, use, development option, or subdivision shall occur on crucial mule deer winter range, unless the applicant can demonstrate that it can be located within the mule deer crucial winter range in such a way that it will not detrimentally affect the food supply and/or cover provided by the crucial winter range to the mule deer, or detrimentally affect the potential for survival of the mule deer using the crucial winter range.*

Direct evidence of mule deer presence observed within the project area included approximately a dozen sets of tracks of animals moving across the project area; no evidence of bedding, resting or foraging were observed. Mule deer were observed foraging on the native vegetation that persists on the cut slope below the Teton County Search and Rescue facility7, and nearly all of the tracks across the project area orginated or terminated in this area. This suggests that individual deer can be expected to move through the project area in route to more favorable habitat located in the vicinity.

Thirteen years of data collected during a winter mule deer study on East Gros Ventre Butte by Biota (1979-1994) and additional data collected by the Conservation Research Center (Teton Science School) showed that deer were not observed within the project area (Appendix 1-Exhibit 3). Three mule deer groups were observed in proximity to the project area at higher elevations during nearly 20 years of data collection.

The Teton County Search and Rescue Facility Mule Deer Monitoring Report prepared by Alder Environmental in 2011 reported no mule deer within or in proximity to the project area during 33 observation events from December 2010 through March 2011. The closest mule deer group observed during the TECO SAR Facility mule deer monitoring study was approximately 250 feet east of the project area. It is a unique circumstance where data over such a time frame, and with this level of effort, are available to substantiate the findings that the proposed development area is not providing crucial winter habitat or vital movement corridors for mule deer.

Moose

The project area does not represent either crucial or non-crucial moose winter range, but has been mapped as non-crucial spring-summer-fall habitat by the Wyoming Game and Fish Department. Moose presence within the project area is expected to be a rare event where individual moose are moving between areas of more suitable habitat.

Teton County Land Development Regulations protect moose winter range (Section 5.2.1 Subsection G.3) and state:

> *No physical development, use, development option, or subdivision shall occur within crucial moose winter habitat, unless the applicant can demonstrate that it can be located within the moose crucial winter habitat in such a way that it will not detrimentally affect the food supply and/or cover provided by the crucial winter habitat to the moose, or detrimentally affect the potential for survival of the moose using the crucial winter habitat.*

Elk

The project area does not represent either crucial or non-crucial elk winter range, but has been mapped as non-crucial spring-summer-fall habitat by the Wyoming Game and Fish Department. No elk sign was observed within the project area. However, an expanding elk population on the Gros Ventre Buttes suggests that a small number of elk may forage in areas proximate to the project area during early green-up, but elk use of the parcel is not expected.

Teton County Land Development Regulations protect crucial elk winter range and migration corridors (Section 5.2.1 Subsection G.1.a & b) and state:

> *No physical development, use, development option, or subdivision shall occur within crucial elk migration routes, unless the applicant can demonstrate that it can be located in such a way that it will not detrimentally affect the ability of elk to migrate from their summer ranges to their crucial winter ranges.*
>
> *No physical development, use, development option, or subdivision shall occur on crucial elk winter range, unless the applicant can demonstrate that it can be located in such a way that it will not detrimentally affect the food supply and/or cover provided by the crucial winter range to the elk, or detrimentally affect the potential for survival of the elk using the crucial winter range.*

Migratory Birds

Migratory birds include raptors, passerines, and shorebirds that breed in North America but migrate to Mexico, and Central and South America for the winter. In Wyoming, 162 bird species are considered neotropical migrants (Cerovski et al. 2001) with peak migration periods occurring May through early October. Nesting is typically initiated in May and June and potential nesting habitat includes native grasslands, shrublands, and cottonwood and coniferous forest stands. In general, deciduous forest communities with cottonwood, willow, and aspen have been found to have higher avian species abundance and richness than any other vegetative community in the western U.S. (Smith and Wachob 2005). Riparian areas often serve as migration corridors for migratory birds and conserving these areas is believed to be essential to maintaining healthy population structures of birds in this region.

A total of 7 oranamental deciduous shrubs and a single conifer are present within the project area, and these plants, at best, represent low quality migratory bird nesting and foraging habitat. Existing development within the property and its associated high level of disturbance may allow generalist avian species such as house sparrows, European starlings, black-billed magpies and pigeon species to inhabit the site. The remaining disturbed portions of the project area offer little or no habitat to migratory birds.

THREATENED AND ENDANGERED SPECIES

In addition to SSCs, the Teton County's Land Development Regulations require that all animals and plants listed under the Endangered Species Act as threatened or endangered be analyzed as part of this EA. Below is a list of threatened, endangered, or recently delisted species that have been documented in Teton County and could potentially occur within the project area. Although 4 listed plant species occur in Wyoming, these plants (i.e., Ute Ladies'-tresses, Colorado butterfly plant, blowout penstemon, and desert yellowhead) have very specific habitat requirements and ranges outside of Teton County.

Species name	Classification/Status
Grizzly bear	Threatened
Gray wolf	Experimental/Non-essential
Canada lynx	Threatened
Yellow-billed Cuckoo	Threatened

No species listed under the Endangered Species Act are present within the project area.

DEVELOPMENT IMPACT ASSESSMENT

PROPOSED DEVELOPMENT

The property previously had a one-story commercial building, and provided storage for a bus fleet and rental vehicles. Future development includes residential townhomes and parking, as provided by Jorgensen Engineering. The approximate area of the proposed site plan includes 0.25 acres of structural development, and 0.25 acres of parking (Exhibit 4).

IMPACT DEFINITIONS

The assessment of environmental consequences of the proposed development on wildlife and fish used the following impact measure, duration, and intensity definitions.
Impact Measures - Four impact measures are examined for wildlife. These include habitat loss, mortality, habitat fragmentation, and human-caused disturbance.

- Habitat Loss - Implementation and perpetuation of all or part of the project would result in a direct loss of habitat.
- Mortality - Implementation and perpetuation of all or part of the project would result in the death(s) of individuals.
- Habitat Fragmentation - Implementation and perpetuation of all or part of the project would result in the fragmentation of habitat.
- Human-caused Disturbance - Implementation and perpetuation of all or part of the project would result in the displacement of individual animals.

Duration of Impact - A short-term impact would have a duration less than or equal to 3 years and a long-term impact would have a duration greater than 3 years following implementation.

Intensity of Impact - Impact thresholds are defined in Table 2.

Table 2. Impact threshold definitions				
Measures	**Negligible**	**Minor**	**Moderate**	**Major**
Habitat Loss **Mortality** **Habitat Fragmentation** **Human-caused Disturbance**	A small number of individual animals and/or a small amount of their respective habitat may be adversely affected via direct or indirect impacts associated with a given alternative. Populations would not be affected or the effects would be below a measurable level of detection. Mitigation measures are not warranted.	Adverse impacts to individual animals and/or their respective habitats would be more numerous and detectable. Populations would not be affected or the effects would be below a measurable level of detection. Mitigation measures may be needed and would be successful in reducing adverse effects.	Effects to individual animals and their habitat would be readily detectable, with consequences occurring at a local population level. Mitigation measures would likely be needed to reduce adverse effects and would likely be successful.	Effects to individual animals and their habitat would be obvious and would have substantive consequences on a regional population level. Extensive mitigation measures would be needed to reduce any adverse effects and their success would not be guaranteed.

IMPACTS TO SURFACE HYDROLOGY

The proposed development action will not impact any surface water feature.

IMPACTS TO WETLANDS

The proposed development action will not impact any wetlands.

IMPACTS TO VEGETATIVE COVERTYPES

There will be no impacts to native vegetative covertypes as a result of proposed development. Impacts to vegetative covertypes total approximately 0.5 acres, and are constrained to disturbed areas.

IMPACTS TO WILDLIFE

Bald Eagles

Proposed development will not adversely impact bald eagle nesting areas or crucial winter foraging habitat. The nearest active bald eagle nest is located approximately 2,900 feet from the western project area boundary, and therefore, proposed development occurring within the project area complies with LDRs pertaining to bald eagles. The eagle nest is not visible from the project area because of its location on the north side of High School Butte and the surrounding vegetation that visually screens it. No precautions associated with the current project need to be taken to protect this nest or bald eagle habitat.

Mule Deer

Proposed development is not expected to adversely impact mule deer or their habitat. The location of proposed development is on a site that has experience numerous iterations of commercial development, and land disturbing activities that have impacted approximately 95% of the surface area over time. A narrow strip of slopeside xeric shrub remains with remnant native cover, howerver, this area too is impacted by noxious weeds that are prevalent on the site. Development is proposed within a largely disturbed area, with very little evidence of mule deer use with the exception of a low incidence of momement between areas of higher habitat quality. No important winter range or crucial habitat is present within the project area, therefore, proposed development will not adversely impact the mule deer population. The proposed action will not inhibit mule deer movements in the vicinity of the project area.

For these reasons proposed development is in full compliance with Section 5.2.1 G2a & b of the Land Development regulations.

MOOSE

Proposed development is not expected to adversely impact moose and, therefore, is compliant with Section 5.2.1 Subsection G.3 of the Land Development Regulations.

ELK

Proposed development is not expected to adversely impact elk and, therefore, is compliant with Section 5.2.1 Subsection G.1.a &b of the Land Development Regulations..

RAPTORS

Proposed development is not expected to adversely impact raptors.

MIGRATORY BIRDS

Proposed development is not expected to result in the net loss of any migratory bird foraging or nesting habitat.

Threatened and Endangered Species

Proposed development on the property is not likely to adversely affect threatened or endangered species.

PROJECT VICINITY IMPACT STATEMENT

The project vicinity impact statement is meant to analyze cumulative adverse impacts on protected resources and critical wildlife habitat resulting from the proposed development and other existing development in the vicinity. The required geographical vicinity of analysis is a ½-mile radius around the project area. The cumulative impacts being analyzed are equivalent to the additive effects of the proposed development to existing residential development and human use in the project vicinity as outlined below.

The Westview Town Homes site is situated along the southern toe of slope of East Gros Ventre Butte, in the Town of Jackson Auto-Urban Commercial Zone. The 1/2-mile impact vicinity zone is comprised of Auto-Urban Commericial, Auto-Urban Residential, Urban Residential, NC Zones, Public Park, and areas zoned Rural within Teton Countyto the north and west. The proposed development density is consistent with development density occurring within the Auto-Urban zones and Urban Residential zones within the impact area.

Crucial mule deer winter range is the only critical wildlife habitat within the 1/2-mile vicinity of the tract. The proposed project is not expected to contribute to adverse cumulative impacts on mule deer in conjunction with other development in the vicinity. No cumulative impacts to elk or moose crucial winter ranges are expected. Development like the proposed and other development in the vicinity will continue to accommodate year-round and winter mule deer use that occurs in proximity to the urban, commercial zones so long as development avoids important habitats and leaves adequate open space for ungulate foraging and movement. There are no adverse cumulative impacts to bald eagles, raptors, migratory birds or Federally protected threatened or endangered species as a result of the proposed development given that no there will be no additive loss of productive habitat.

CONCLUSIONS

The Natural Hazard Protection Standards of the Jackson Land Development Regulations classify the Westview Town Homes project area as a qualifying "Steep Slope" and proposed development requires an assessment of wildlife use and potential adverse impacts to wildlife. The project area falls within mapped crucial winter range for mule deer. Elk and moose crucial winter ranges are absent. The project area occurs in the vicinity of an active bald eagle nest but outside of the 660-foot nest setback. The site has been almost entirely disturbed as a result of historic and existing commercial use and development; only about 5% of the land area supports native, xeric shrub vegetation.

The proposed development is confined almost exclusively to previously disturbed areas bordering Wyoming Highway 22, but falls within Wyoming Game and Fish Department mapped mule deer crucial winter range. The determination of potential impacts to mule deer involved both mapping and evaluating foraging opportunities, as well as reviewing several observational datasets that span the years from 1979 through 2011 (including 14 winter seasons). Review of each of these studies provided empirical support for a conclusion that no negative impacts to mule deer, their crucial habitat, or crucial movement corridors are expected to result from the proposed action. In addition, no negative impacts are expected to effect other protected natural resources including wetlands, watercourses or associated setbacks, wildlife species of special concern, or species with Federal protected status.

APPENDIX 1 – LIST OF EXHIBITS

WESTVIEW TOWN HOMES PROJECT AREA

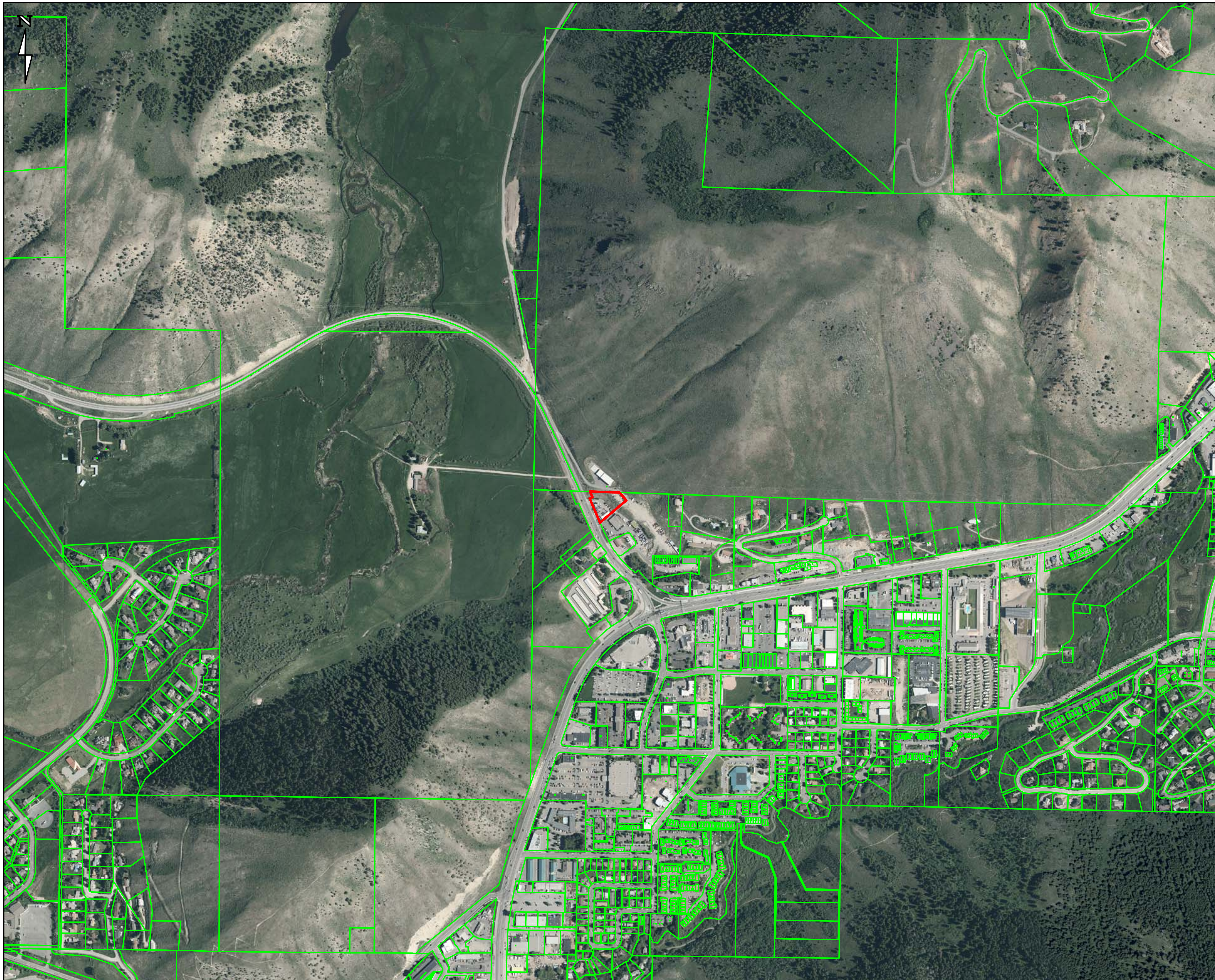
ENVIRONMENTAL ANALYSIS, TETON COUNTY, WYOMING

1) Aerial photograph depicting the location and site characteristics of the Westview Town Homes property in Jackson, Teton County, Wyoming.

2) Aerial photograph depicting vegetative covertypes within the Westview Town Homes property in Jackson, Teton County, Wyoming.

3) Aerial photograph depicting mapped mule deer habitat and historic observations on and in the vicinity of the Westview Town Homes property in Jackson, Teton County, Wyoming.

4) Aerial photograph depicting proposed development within the Westview Town Homes property in Jackson, Teton County, Wyoming.



Attachment 1
Aerial photograph depicting the location and site characteristics of the Westview Town Homes property in Jackson, Teton County, Wyoming.

January 6, 2016

Approximate Scale: 1 inch = 800 feet

Legend

Westview Town Homes Property

Platted Parcels

research & consulting inc.
Biota

PO Box 8578, 140 E. Broadway, Suite 23, Jackson, WY 83002

Attachment 2
Aerial photograph depicting vegetative covertypes within the Westview Town Homes property in Jackson, Teton County, Wyoming.

January 6, 2016

Approximate Scale: 1 inch = 50 feet

Legend

Westview Town Homes Property

Covertype Legend

Disturbed Grassland

Xeric Shrubland

Disturbed

research & consulting inc.
Biota
PO Box 8578, 140 E. Broadway, Suite 23, Jackson, WY 83002

Attachment 3
Aerial photograph depicting mapped mule deer habitat and historic observations on and in the vicinity of the Westview Town Homes property in Jackson, Teton County, Wyoming.

January 6, 2016

Approximate Scale: 1 inch = 400 feet

Legend

Westview Town Homes Property

Mule Deer Habitat Legend

Crucial Winter Range

Crucial Winter Yearlong Range

Number of Mule Deer Observations

Within Project Area
None

Adjacent To Project Area
★ 1982-83 = 20
★ 1987-88 = 10
★ 1987-88 = 1

research & consulting inc.
Biota

PO Box 8578, 140 E. Broadway, Suite 23, Jackson, WY 83002

Exhibit 4
Aerial photograph depicting proposed development within the Westview Town Homes property in Jackson, Teton County, Wyoming.

January 6, 2016

Approximate Scale: 1 inch = 50 feet

Legend

- Westview Town Homes Property
- Parking
- Townhomes
- Platted Parcels

research & consulting inc.
Biota

PO Box 8578, 140 E. Broadway, Suite 23, Jackson, WY 83002

Town Council
Town of Jackson
Jackson, Wyoming

January 6, 2016

RE: Design character and visual analysis for the Westview Town Homes Project.

Council Members,

I'm writing on behalf of the owners of the Westview Town Homes Project, a proposed development for1255 West Highway 22. I assisted the Owners with the design and sighting of the project, and was asked to provide some explanation for our thought process.

The topography of the site created 2 development areas – one at street level adjacent to highway 22, the other on the bench above – accessible from Batch Plant Road, north of the project. The lower four buildings (pods) each house 4 units, the upper two buildings: 2, for a total of 20 residential units.

By arranging the units around a central parking area, we minimized the amount of paving need to service the buildings. This also creates a village configuration around a semi-enclosed courtyard, which is desirable and especially appropriate for a residential development. On the open side of the courtyard, the side adjacent to highway 22, we created separation with a berm and trees. The resulting arrangement creates a sense of separation and security for the units and a natural but defined street edge for the highway.

We used neutral earth tones in the materials pallet to complement rather than contrast with the site. The units are pushed into the hillside to reduce their visual impact and preserve the natural flat area of the site for circulation, in turn eliminating a need for expressed retaining walls. We used low slope roofs to get the units stacked and under the height allowed, which allowed us to break the development up into smaller buildings. Finally, we're planning to reclaim and enhance the hillside with new trees and irrigation to further soften and tie the development to the site.

Sincerely,

Christopher Lee
Owner – Design Associates Architects.

Tyler Valentine

From: Josh Frappart
Sent: Thursday, September 15, 2016 2:35 PM
To: Tyler Valentine; Larry Pardee
Subject: RE: Westview Town Homes - Geotech

Tyler,

We are good with this final report. It appears they have revised their initial report to address the issue brought up by the third party review. Let me know if we can help with anything else. Thanks,

Josh P. Frappart
Associate Engineer
Town of Jackson - Public Works
450 Snow King Avenue
PO Box 1687
Jackson, WY 83001
Office: (307)733-3079 ext. 1413
Cell: (307)690-4295
Email: jfrappart@ci.jackson.wy.us

From: Tyler Valentine
Sent: Thursday, September 15, 2016 1:41 PM
To: Josh Frappart <JFrappart@ci.jackson.wy.us>; Larry Pardee <lpardee@ci.jackson.wy.us>
Subject: FW: Westview Town Homes - Geotech

Josh or Larry,

Reed just submitted a response to the third party review (attached). Is Engineering okay with this?

Thanks,

Tyler V

From: Reed Armijo [mailto:rarmijo@jorgensenassociates.com]
Sent: Thursday, September 15, 2016 11:49 AM
To: Tyler Valentine <TValentine@townofjackson.com>
Cc: Larry Pardee <lpardee@ci.jackson.wy.us>; Josh Frappart <JFrappart@ci.jackson.wy.us>; Colter Lane <clane@jorgensenassociates.com>
Subject: Westview Town Homes - Geotech

Tyler –

Enclosed please fond the Westview Town Home geotechnical report with an errata addressing George Machan/Landslide Technology comments. Please let me know if you have any questions.

Thank you,
Reed

Reed Armijo P.E.
Principal Engineer
PO Box 9550 · 1315 HWY 89 S., Suite 201
Jackson, WY 83002
TEL: (307) 733-5150 x314
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Jackson, WY · Pinedale, WY · Driggs, ID

Geotechnical Investigation Report
Westview Town Homes
1255 W. Highway 22
Jackson, Wyoming

ERRATA AND COMMENTARY

Report Background

Jorgensen Geotechnical (JG) prepared a Geotechnical Investigation Report for the proposed Westview Town Homes project at 1255 W. Highway 22 in Jackson, Wyoming dated July 27, 2016. Geotechnical analysis indicates the slope at the site was stable under static and seismic conditions. The primary geotechnical concern at the site is collapsible deposits of wind-blown silts and clays (i.e., loess).

The Town of Jackson requested a 3rd Party Review of the report by Landslide Technology (LT) of Portland, Oregon. A letter summarizing the review was submitted by George Machan, P.E. on August 30, 2016. LT acknowledges "the results of the stability analysis indicate relatively stable conditions." A majority of the review comments pertains to managing differential settlement and collapse potential of the loess soils observed at the site.

JG has prepared this Errata and Commentary (E&C) in response to the technical aspects of the review and letter from LT. This E&C is hereby incorporated into the Geotechnical Report and should accompany the report in all future submittals and correspondence.

Errata

1) Section 6.1.3, page 26
   Pressure Distribution under a Footing
   The correct distribution is "**1/2H:1V slope.**"

Commentary

Structural Design of Slabs

The approach recommended by this office reduces the risk of slab settlement by compacting native soil (see Section 7.4 Interior Slabs-on-Grade). The result is a layer of compacted fine-grained soil with reduced collapse potential that is also hydro-phobic. Our office has used this approach to improve the performance of interior slabs-on-grade for many years. Interior slabs are typically more protected from environmental effects (e.g., wetting, drying, freezing, etc.) than exterior slabs and are also usually very lightly loaded. We generally don't recommend structural slabs unless the anticipated movement is upward, such as with expansive soils. It is generally accepted that floor slabs are almost never free from cracks and cracking is caused by many factors other than differential settlement of underlying native soil. The additional expense of requiring a reinforced structural slab, in our opinion, is not justified, particularly if the owner is accepting of cracking within reasonable tolerances.

Limits of Excavation

Although assuming foundation pressures induced on underlying soil follow a linear distribution of 0.5H:1V, often referred to as the "2:1 Method", it is not the only approach to estimate pressures applied to soil by foundation elements. Boussinesq stress distributions, based on elasticity theory, have more of a "bulb" shape. The figure below shows the pressure distributions for a strip footing of width = b. As is shown in the figure, a pressure equipotential line equivalent to 30% of the foundation pressure (0.3p) extends to an approximate depth of 2b, commonly referred to as the foundation's "zone of influence", and only extends laterally to approximately 0.75b from the center of the footing. Therefore, in our opinion, requiring the excavation to extend the lateral distances suggested by LT is not necessary.

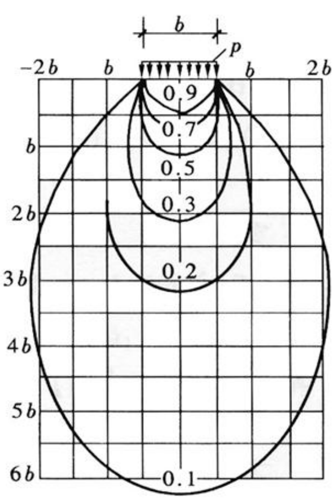

Figure 1: Boussinesq Vertical Stress Distribution below Continuous Footing

Temporary Cut Slope Stability

See Section 6.5 of the Geotechnical Report. Loess soil in the Jackson Hole region is most commonly classified as Type A soil according to OSHA regulation. However, the consistency of loess can change dramatically with changes in moisture, which often differs between the time of the investigation and construction, and it can also be fissured. We acknowledge softening of the soil with increasing moisture and fissuring can both downgrade the loess to Type B or Type C. We make an initial recommendation of excavation slopes in our reports to help contractors and designers plan the construction. Excavation slopes shown on Figures 10 and 11 are schematic in nature and are intended to help designers and contractors visualize the proposed excavation. They do not constitute a requirement of temporary slopes. As stated in the report, the "Contractor shall ultimately be responsible for adherence to OSHA and other safety regulations" by observing unconfined compressive strength and any fissuring structure <u>at the time of construction</u>.

Cuts and Fills Stability

We have made recommendations regarding the order in which excavation and fill placement should take place in order to reduce the risk of slope instability during construction. See Section 6.5 of the Geotechnical Report.

Subdrainage

Foundation and sub-slab drain options have been described in Section 7.3 of the Geotechnical Report and depicted in Figure 12. The Superior Wall® foundation system recommends a fairly robust sub-slab drainage system that we have incorporated into our typical recommendations. Drain layers will be placed on top of compacted native loess, forming a low permeable barrier between the drain layer and the underlying native loess. The Report notes that "management of water at this site is extremely important" and recommends JG review final plans to ensure that site drainage is properly accounted for.

Surface Water and Exterior Slabs-on-Grade

Final site grading and management of surface water from sources such as roof runoff and rainfall infiltration are extremely important. The Report recommends not using hardscapes or landscaping features that are sensitive to differential settlement. The Report also strongly recommends "landscapers and water feature designers should be provided the geotechnical report and formally briefed about the necessity to manage water and grades at the site."

LANDSLIDE
TECHNOLOGY

10250 S.W. Greenburg Road, Suite 111
Portland, Oregon 97223
Phone 503-452-1200 Fax 503-452-1528

August 30, 2016 2498

Mr. Tyler Sinclair
Town of Jackson
P.O. Box 1687
Jackson, Wyoming 83001

Geotechnical 3rd Party Review – Slope Stability
Proposed Westview Townhomes Project
1255 West Highway 22,
Jackson, Wyoming

Dear Mr. Sinclair:

In accordance with your request, we have performed a 3rd Party geotechnical slope stability review of the July 27, 2016 Geotechnical Report for this proposed residential development. The Geotechnical Report was prepared by Jorgensen Geotechnical, Jackson, Wyoming.

Background Information

The site is located near the intersection of West Highway 22 and West Broadway Avenue, at the toe of the East Gros Ventre Butte slope. The site has been regraded in the past to create two benches with a steep slope between. The preliminary project plan is to construct townhomes on both benches.

Geologic conditions are described on the Geologic Map of the Jackson Quadrangle, LMS-9, published by the State of Wyoming Geologic Survey (Love & Albee, 2004). Results of Jorgensen Geotechnical's subsurface investigation are presented in their July 27, 2016 Geotechnical Report. Additional subsurface conditions and geotechnical data are provided in the Womack report for the adjacent Clark property to the southeast (dated March 14, 2008) and the Landslide Technology reports for the nearby landslide at Budge Drive / West Broadway Avenue (June 2014).

Slope Stability

The stability of the hillside slopes was investigated by Jorgensen Geotechnical, which included relatively deep subsurface explorations to investigate if possible landslide conditions exist. Jorgensen Geotechnical performed geologic reconnaissances and reviewed site geomorphology, and did not observe or identify landslide features. Subsurface conditions primarily consist of loess overlying stony colluvium and layers of low-plasticity clay. Landslide conditions were not evident in the subsurface explorations. The results of the stability analysis indicate relatively stable conditions, and the geotechnical report addresses stability design issues when excavating or filling on this site.

Another geotechnical concern has been identified in the geotechnical report, associated with wind-blown loess deposits. The loess is potentially compressible, particularly when impacted by water, and differential settlement may result, as described in the geotechnical report.

Review Comments

The report describes several methods to reduce the potential impact of differential settlement/collapse of the loess. The option that completely removes the loess and replaces it with compacted structural fill is the preferred option since this removes the concern of soil collapse.

Another option described in the geotechnical report is the use of helical foundation piers; however, the concrete floor slab would be subject to differential settlement, which could cause cracking and unevenness of the floor slab. The report also describes an option of partial removal of loess (overexcavation and replacement), as shown in Figure 11, which treats the upper zone of loess but leaves the deeper portion of loess inplace, with the risk of differential settlement/collapse impacting foundations and floor slabs.

If the helical pier option is pursued further, measures to prevent differential settlement of the floor slabs should be evaluated, including concrete floor slabs that would be designed to span between pier foundations. The geotechnical report should also indicate the likely range of pier depths based on the subsurface materials that were encountered in the investigations.

If the partial overexcavation/replacement option is pursued further, the structure design should include structural engineering analysis and design to verify the structure would flexibly accommodate differential settlement without damage, or would span areas of differential settlement without damage. Structural design of both footings and floor slabs should be performed.

Lateral limits of foundation treatment (consisting of overexcavation of loess and replacement with structural fill) are likely to extend further than the dimension "B" shown on Figures 10 and 11 (Jorgensen geotechnical report). The lateral extent of bearing pressure as it propagates deeper to competent subsurface materials is typically assumed 0.5H:1V. Using this approach, the lateral extent of the base of the overexcavation area should be defined at the toe of the temporary subexcavation slope. For example, if the depth of overexcavation is 10 feet to reach firm competent subsurface material, the lateral extent to the toe of the temporary subexcavation slope would be 5 feet. In addition the slope angle needed for the temporary subexcavation slope would make the lateral limits of subexcavation larger (e.g., an additional 10 feet horizontal if the temporary cut slope angle is 1:1, which would require further evaluation to select the appropriate slope angle).

Temporary sideslopes for excavation are regulated by OSHA. The details in the geotechnical report show a temporary slope of "0.75H:1V" which the report states is OSHA's requirement for Type A soils (firm cohesive soils, with unconfined compressive strength of at least 1.5 tsf);

however, the selection of the angle of the temporary cut slope is typically the responsibility of the construction contractor and based on OSHA requirements for the actual soil conditions encountered. OSHA requires minimum 1:1 and 1.5H:1V cut slope angles for Type B and Type C soils, respectively. Type B soil is defined by OSHA as cohesive soils with an unconfined compressive strength of 0.5 to 1.5 tsf. Type C soil is defined by OSHA as cohesive soils with an unconfined compressive strength of 0.5 tsf or less. The geotechnical report states that the loess could be very soft to medium stiff. The temporary cut slope angle should be based on OSHA requirements for cut slopes being exposed greater than 24 hours. If the geotechnical engineer of record intends the temporary cut slope to be made at a specific angle, such as "0.75H:1V", then provide the rationale and supporting analysis for the recommendation in the geotechnical report.

There appears to be a typo on the bottom of page 26, where "0.5V:1H" probably was intended to be "0.5H:1V".

Cuts and fills should be minimized to avoid causing slope instability. Control of surface water and subsurface water should also be controlled to avoid causing slope instability.

Subdrainage systems installed to prevent groundwater from impacting basement walls and floor slabs are standard practice. Drain pipes in all cases need to have a continuous gradient to provide positive flow towards discharge points, which should be defined and labeled on all details. A special consideration for subdrains that are underlain by potentially-collapsible loess soil is to either remove all the underlying loess, or to prevent water in the drain pipes, sumps and free-draining backfill from infiltrating into the subsurface, or to design the structure, foundations and floor slab to tolerate potential differential settlement without damage. Prevention of water infiltration into the subsurface is also a prudent "best management practice" for developments constructed on slopes.

Exterior slabs, facilities ("hardscapes") and landscape areas that are underlain by loess might also experience differential settlement. The report describes some of the concerns, for example swimming pools, hot tubs, fountains and other water features, and sprinklers. In addition, roof runoff and rainfall infiltration could contribute to collapse and differential settlement of loess soils, or slope stability. The Town should consider whether to require advance mitigation measures or whether the risk can be acceptable according to the building codes, laws, and local practice.

Consider requiring a statement in the property deeds (covenant) acknowledging the risk of differential settlement, potential soil collapse, and slope stability, and explain that various sources of water can contribute to differential settlement/collapse/instability. In addition, such requirements can be incorporated in the townhouse "Covenants, Conditions and Rules (CC&R)" and homeowner bylaws to be binding on all property owners. A maintenance manual should also be considered for describing appropriate practices for managing and performing modifications to the buildings, paved areas, landscaped areas, pools, etc., in regards to risks associated with loess soils and water infiltration potentially reducing slope stability.

Summary

The review comments are provided from a geotechnical perspective, primarily addressing slope stability and the potential issues associated with loess soils. The recommendations and analyses described in the Jorgensen geotechnical report have been reviewed in a general manner to check relative consistency with slope stability practice and treatment of loess soils; however, independent site evaluations, geotechnical investigation/testing or analyses are not the responsibility of this third party review. In addition, this review does not include checking building code requirements, which is the responsibility of the Town's building department. The geotechnical designer of record is responsible for the accuracy and appropriateness of their investigation, analyses and recommendations for compliance with local building codes, and for geotechnical design and construction, and performing QA/QC of their work. In addition, there are concerns that should be addressed by site development and structural engineers.

If further clarification of the geotechnical comments is desired, please contact the undersigned.

Very truly yours,

LANDSLIDE TECHNOLOGY

George Machan, P.E.
Senior Associate Geotechnical Engineer

GEOTECHNICAL INVESTIGATION REPORT
WEST VIEW TOWNHOMES
1255 W HIGHWAY 22
JACKSON, WYOMING

Prepared for:

Mr. Eric Grove
F.S.D. Investments, Inc.
P.O. Box 9879
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Prepared by:

Jorgensen Geotechnical, LLC
P.O. Box 9550
Jackson, Wyoming 83002

July 27, 2016

JORGENSEN
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July 27, 2016

Mr. Eric Grove
F.S.D. Investments, Inc.
Transmitted via email: ericgrovemn@gmail.com

RE: GEOTECHNICAL INVESTIGATION REPORT, WEST VIEW TOWNHOMES, 1255 W. HIGHWAY 22, JACKSON, WYOMING (PROJECT NO: 09040.02)

Dear Eric,

We are pleased to present this geotechnical investigation report the proposed West View Townhomes located at 1255 W. Highway 22 in Jackson, Wyoming. The report describes site conditions and presents conclusions and recommendations to support design and construction of foundation elements.

Summary

Due to slope stability concerns at the site, the investigative and analytical level of effort has far exceeded what is typical for the residential project of this scale. However, the effort has been worthwhile as our investigation and analyses appear to indicate there is not a slope stability issue for the proposed project. Plastic clays present at the West Broadway Landslide and other nearby properties were not observed.

Loess (i.e., wind-deposited silt), which has been shown to collapse when wetted under load, is the primary geotechnical concern. Three different foundation options are presented to reduce the risk of settlement associated with building on loess soils. Water management, both during and post-construction, will be extremely important. Landscapers and other designers should be provided this geotechnical report and formally briefed about the necessity to manage drainage and grades at the site.

If you have any questions about this report, or if we may provide other services to you, please contact us. As the project progresses, we will be available to answer questions.

Respectfully submitted,

JORGENSEN GEOTECHNICAL, LLC

Professional Engineer (Civil)
WESLEY R. WOMACK
4958
7/27/2016
Date
WYOMING

Ray Womack, PE, PG

Colter H. Lane, EI, MS

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1.0 INTRODUCTION

The proposed West View Townhomes development at 1255 West Highway 22 in Jackson, Wyoming (Figure 1) is located approximately 2000 feet northwest of the West Broadway Landslide (WBL). Due to concerns about similar geology between the two sites along the toe of East Gros Ventre Butte, geotechnical investigative and analytical work at this site has exceeded that which would be typically employed for a residential development.

At the request of Mr. Eric Grove, Jorgensen Geotechnical performed a preliminary slope stability analysis for the proposed project. Results of the stability analysis were presented in a report dated September 29, 2015. The preliminary results indicated the slope at the site was likely stable under existing and seismic conditions. A site specific geotechnical investigation was recommended to verify assumptions regarding the underlying subsurface conditions.

A detailed geotechnical site investigation was performed on June 1-3, 2016. The purposes were to observe soil and groundwater conditions, evaluate soil-engineering properties, explore for weak, plastic clays associated with the WBL, and to provide recommendations to support design and construction of foundation and drainage elements. The scope of services included drilling and logging six exploratory borings, installing three vibrating wire piezometers, performing engineering analyses, and producing this geotechnical investigation report.

The primary geotechnical concern is plastic clay deposits observed to the southeast of the project site and found to underlie the slide block of the WBL. These clays were not observed in the investigation.

2.0 PROPOSED CONSTRUCTION

The proposed development will consist of twenty residential units in six buildings. Four of the six buildings will be located on the lower portion of the parcel and consist of four units with three bedrooms per unit. The remaining two buildings will be located on the upper portion of the parcel and consist of two units with three bedrooms per unit. Access to the site will be provided in two existing locations; one directly from WY 22 and the other using Batch Plant Road (County Road 22-14).

It is our understanding the proposed foundation system will comprise prefabricated Superior Walls® placed on a clean crushed stone footing with interior slabs-on-grade. Construction of the upper levels will use structural insulated panels (SIPs) and associated techniques.

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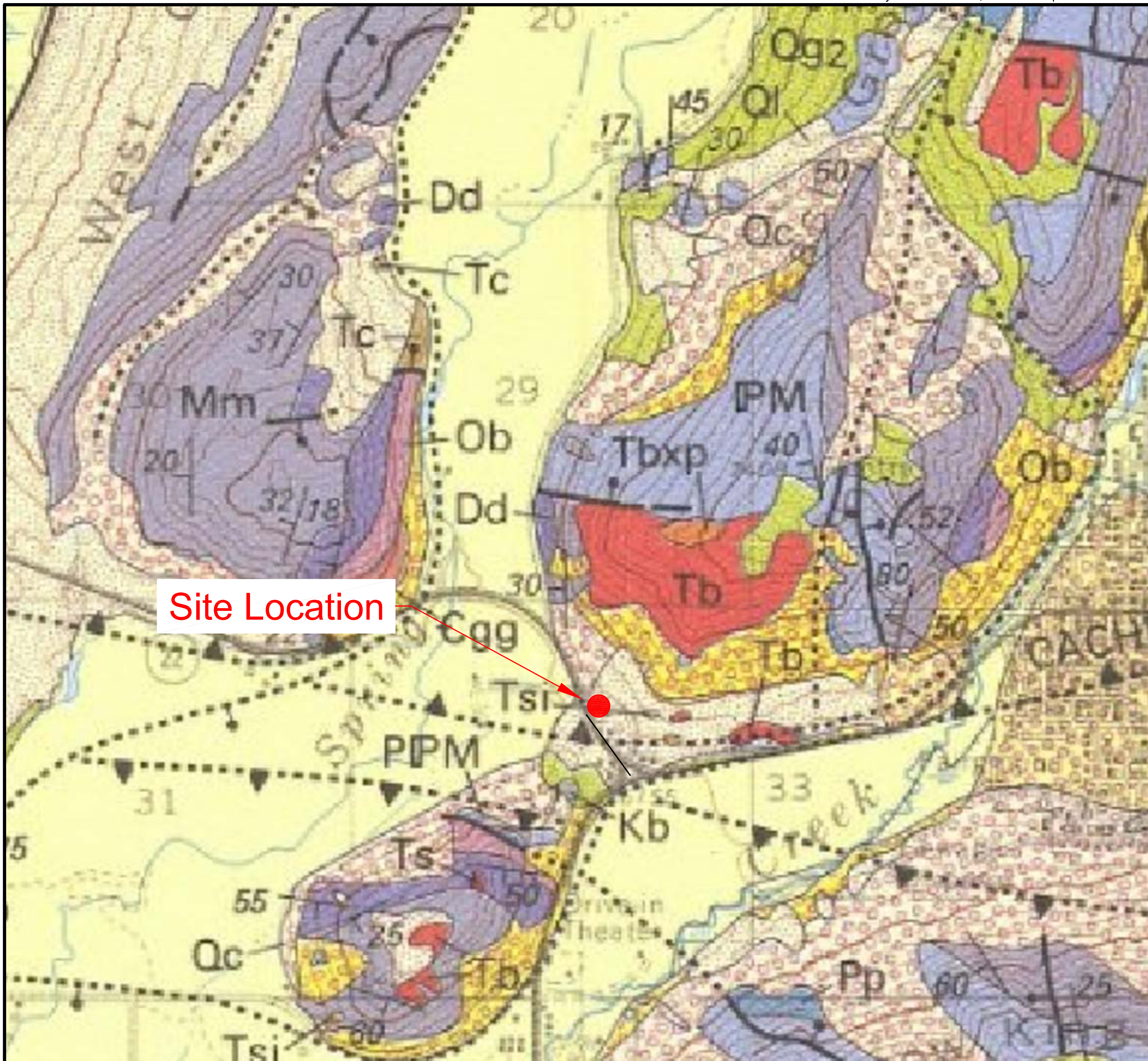

Map adapted from:
"Love et al, 1992, Geologic Map of the Grand Teton National Park, Teton County, Wyoming, Map I-2031"

Relevant Map Symbols:

Qc	Colluvium
Qt	Talus and Related Deposits
Ql	Loess
Tb	Basaltic Andesite
Tsi	Shooting Iron Formation
PPM	Wells/Amsden Formations
Tbxp	Flow-breccia and Pumice
PM	Tensleep Sandstone/Amsden Formation
Kb	Bacon Ridge Sandstone

0 2000 4000

SCALE: 1 INCH = 2000 FEET
THIS SCALE VALID ONLY FOR 8.5x11 PRINTS

DRAFTED BY: CHL REVIEWED BY: RW PROJECT NUMBER 09040.02	SHEET TITLE: FIGURE 1 SITE LOCATION AND GEOLOGICAL MAP	PROJECT TITLE: WEST VIEW TOWNHOMES 1255 W HIGHWAY 22 JACKSON, WY	JORGENSEN Geotechnical, LLC 307-733-5150 www.jorgeng.com

3.0 INVESTIGATION PROCEDURE

3.1 Field Investigation

A field investigation at 1255 West Highway 22 was conducted on June 1st through June 3rd, 2016. A staff geotechnical engineer from this office directed the drilling and sampling of six hollow-stem auger borings, designated JG-1 through JG-6 in the order in which they were drilled. Location and depth of each boring were chosen to explore potential slope instability, specifically plastic, lacustrine (i.e., lake-deposited) clays near elevations 6,150-ft to 6,160-ft. Depths of borings ranged from 31 to 71.5 feet below the ground surface (bgs), which exceed that which is typical of light, residential construction. Depths and location Soil type, thickness, consistency, and relative moisture content were observed and documented by the engineer.

Three vibrating wire piezometers (VWPs) from Durham Geo Slope Indicator were installed in borings to facilitate monitoring changes in groundwater levels during the weeks following the site investigation. One VWP was installed in JG-3 (JG-3-P1) and two VWPs were installed in JG-5 (JG-5-P2 and JG-5-P3). Each VWP was attached to the outside of a 1-inch PVC pipe and grouted in place using a bentonite-cement grout as recommended by the manufacturer. VWP serial numbers and installation depths are shown on the boring logs in Appendix A and calibration sheets of the VWPs are in Appendix B.

Surveyed borehole locations are shown on Figure 2 and borehole logs are presented graphically in Appendix A. Borehole locations were selected by the engineer to represent the proposed construction. Site conditions are variable and actual soil conditions encountered in the foundation excavation may differ from those represented in the borehole logs.

Standard penetration tests (SPT) were recorded and samples were obtained from all six borings at 2.5 to 5-foot intervals. Blow counts for the Standard Penetration Test (field N-values) were adjusted for hammer efficiency and overburden stress as suggested by Youd and Idriss (1997 and 2001) and Fang (1991). The blow counts were adjusted to a standard hammer efficiency of 60% and overburden pressure of one atmosphere, to obtain the standard adjusted $(N_1)_{60}$ value in blows per foot (bpf).

Data of a boring that Womack & Associates installed on the project site during a 2011 investigation for the Town of Jackson East Pathways Project were examined and incorporated into our analysis as part of this work.

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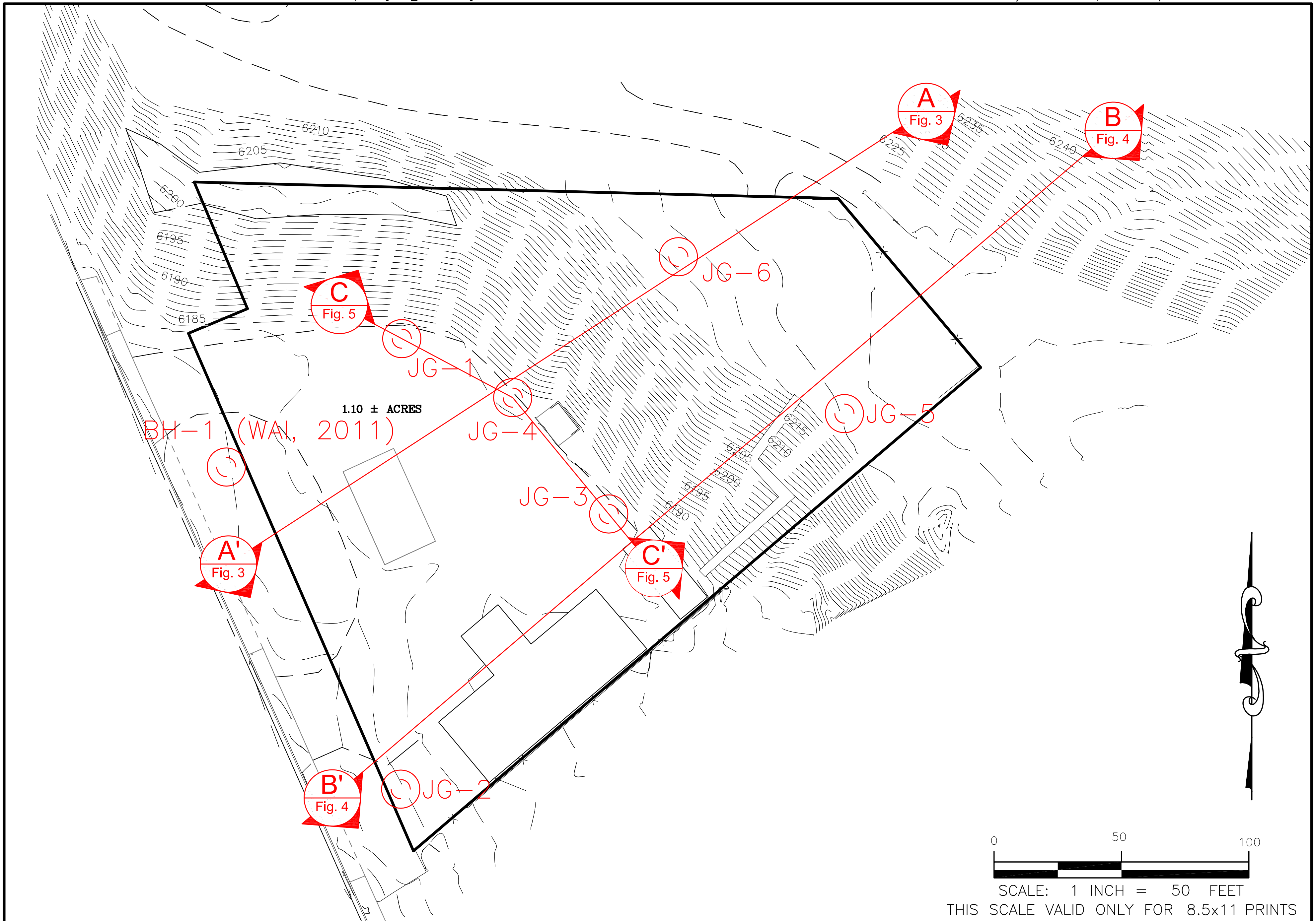

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REVIEWED BY:	RW
PROJECT NUMBER 09040.02	

SHEET TITLE:
FIGURE 2
BOREHOLE AND CROSS-SECTION LOCATION MAP

PROJECT TITLE:
WEST VIEW TOWNHOMES
1255 W HIGHWAY 22
JACKSON, WY

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3.2 Laboratory Analyses

Selected samples of fine-grained soils were sent to the soils laboratory of SK Geotechnical in Billings, MT, and were tested to classify the soil and to estimate engineering parameters. Classification tests included natural moisture content, Atterberg limits, and gradation. Relatively undisturbed specimens obtained with thin-walled Shelby tubes were tested for dry density, consolidation, collapse potential, and shear strength. Laboratory results are in Appendix C.

3.3 Report Preparation

The report describes the geological site conditions and includes a site location and geologic map, borehole logs, laboratory test results, and generalized geologic cross-sections. The report provides engineering analyses and recommendations for construction of foundation elements.

4.0 SITE CONDITIONS

4.1 Description

The project site of the West View Townhomes is located on a 1.1 acre property within the Town of Jackson limits along Wyoming Highway 22 (WY 22). The parcel is approximately 1,030 feet west of the U.S. Highway 89 and WY 22 intersection, at the southwestern toe of East Gros Ventre Butte (Figure 1). The parcel consists of a lower level area adjacent to WY 22 at an approximate elevation of 6,188 feet above mean sea level (AMSL) and an upper level area that is approximately 35 feet above the lower area.

Several buildings currently occupy the lot and will be removed as part of the proposed development. The majority of lower portion of the lot is paved while the upper portion is currently surfaced with imported aggregate.

4.2 Historical Information

It appears the “benched” topography observed at the site is not a result of soil or rock deposition but was instead created by excavation. There does not appear to be evidence that excavated soils were stockpiled or used as fills on the site. The original ground surface is estimated to be approximately 3H:1V and has been shown on the provided cross-sections (Figures 3 through 5).

According to the Teton County GIS Map Server, excavation began on the lower pad sometime between 1945 and 1955 and was expanded to approximately its current configuration by 1999. It appears the initial improvements included two accesses from WY 22, several small buildings, and a tank array on a small bench at the north end of the property. The upper pad or deck and Batch Plant Road were excavated sometime between 1955 and 1967 and also expanded to approximately its current condition by 1999.

The site has been used for a variety of commercial uses including a gas station and convenience store, a small market specializing in meat (Choice Meats), a rental car agency, and most recently a transit operation (Alltrans).

The project site was previously registered in the Underground Storage Tank (UST) program of the Wyoming Department of Environmental Quality (WYDEQ). Past work on the site included numerous monitoring wells, the majority of which have since been abandoned. The site gained "resolved" status in 2004 and soil or groundwater contamination is not anticipated to affect the proposed construction.

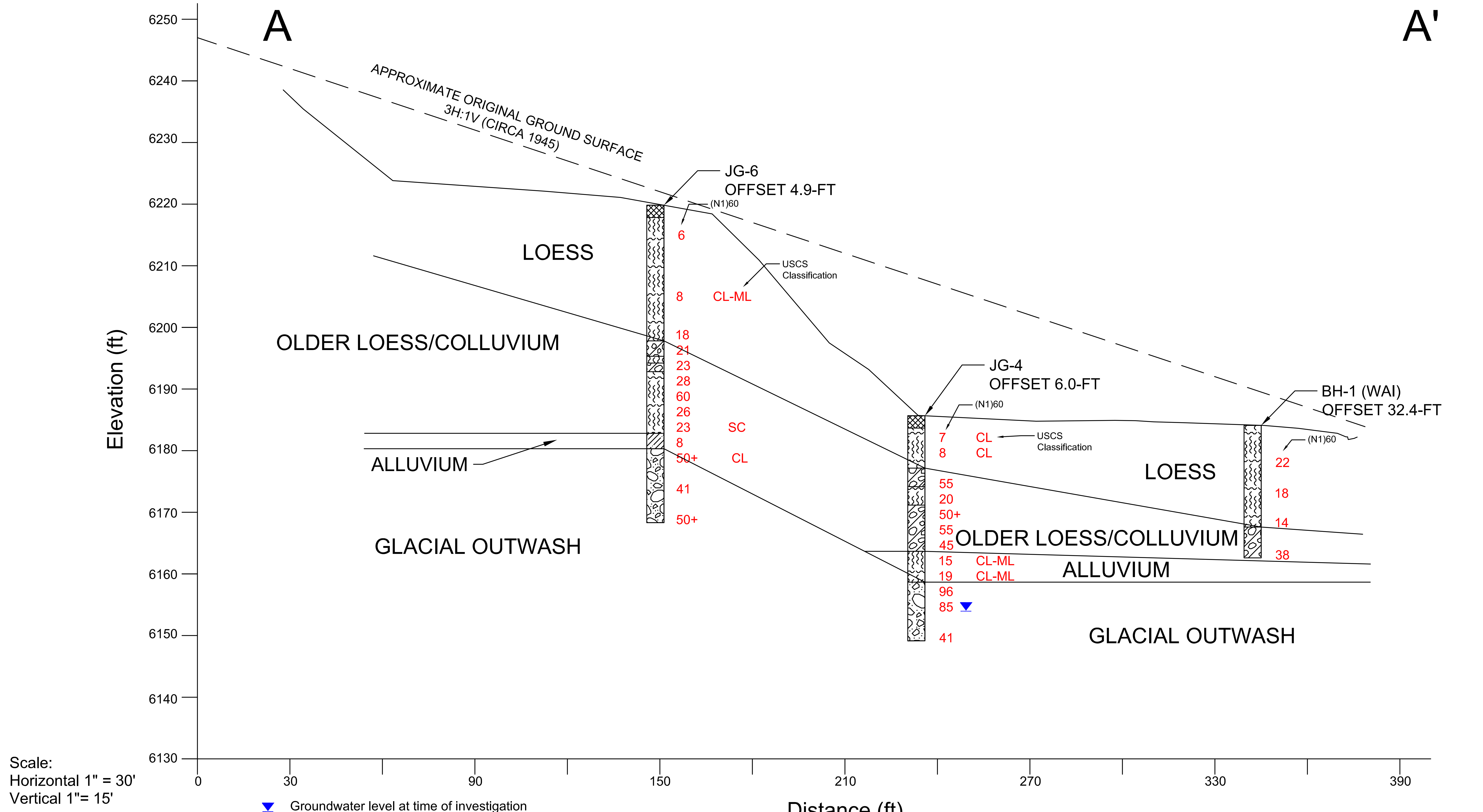
4.3 Geology

Figure 1 is a generalized geologic map of the project site adapted from the Geologic Map of the Grand Teton National Park (Love, et al., 1992), which shows the location and type of surface deposits, bedrock units, and geologic structures (i.e., faults and rock orientations). According to the map, the project site is at least partially covered by Quaternary loess deposits (Ql) which are windborne (aeolian) silt deposits, typically derived from glacial outwash sources. The west end of the site is mapped as colluvium (Qc), consisting of gravity deposits of limestone and "basalt" gravel and silt derived from outcrops upslope. Bedrock is not shown on the map, but small windows of Quaternary-aged clayey lakebeds of the Shooting Iron Formation appear just off the property to the south.

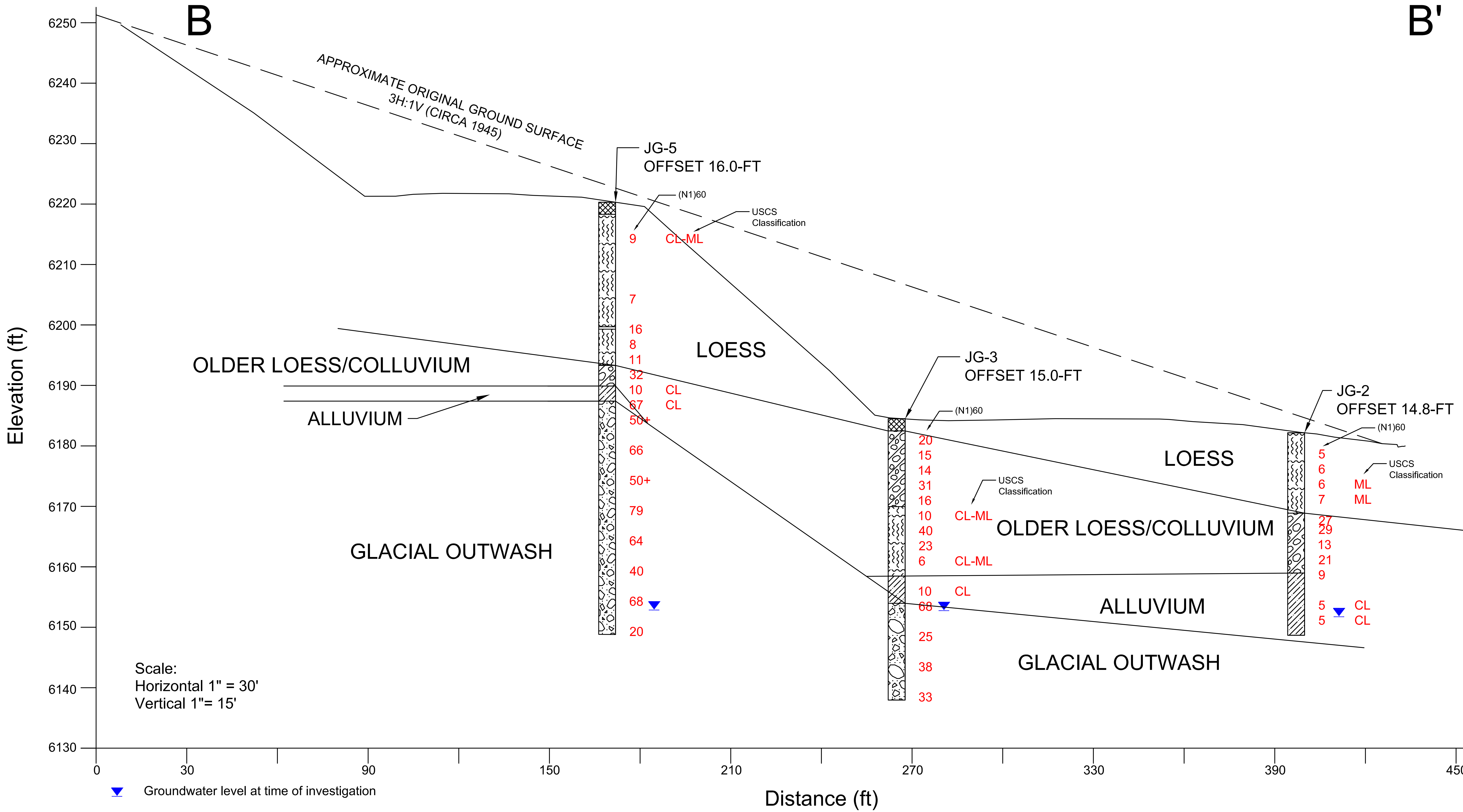
The geologic map depicts outcrops and surface soil deposits; subsurface conditions are usually more complex. The basic stratigraphy of the site consists of a variable layer of younger loess underlain by interbedded layers of stony colluvium and older loess, underlain in turn by stony glacial outwash (Qg2). In some locations, alluvial low-plastic clay was observed directly above the stony outwash. It is thought that these alluvial clays were deposited in a low-energy environment near the end of the glacial melt-out episode, possibly in discontinuous stream channels on the surface of the stony outwash.

As the geologic cross-sections illustrate, the stony glacial outwash at one time probably had the benched appearance of the terraces along the Snake River in Grand Teton National Park north of Jackson. These terraces were subsequently obscured by deposition of windblown loess and colluvium (gravity deposits from the face of East Gros Ventre Butte). Abrupt steps should be expected between the buried stony glacial outwash terraces. For example, the elevation of the glacial outwash on the upper bench varies by about 8-ft. The outwash was originally level and was subsequently eroded by lateral channel movement, creating a higher terrace. Later erosion and down-cutting lowered the gravel surface an additional 20 to 35 feet (see Figures 3 and 4).

Laminated lake bed deposits comprised of plastic clays, which are known to exist to the south and east of the project, were not observed in any of the borings. The most problematical material appears to be the loess (see Section 4.4.2). More detailed discussion of soil types encountered during the site investigation may be found in the following sections.



DRAFTED BY: CHL	SHEET TITLE:	PROJECT TITLE:	JORGENSEN Geotechnical, LLC
REVIEWED BY: RW	FIGURE 3	WEST VIEW TOWNHOMES	307-733-5150 www.jorgeng.com
PROJECT NUMBER 09040.02	GENERALIZED GEOLOGIC CROSS-SECTION A-A'	1255 W HIGHWAY 22 JACKSON, WY	



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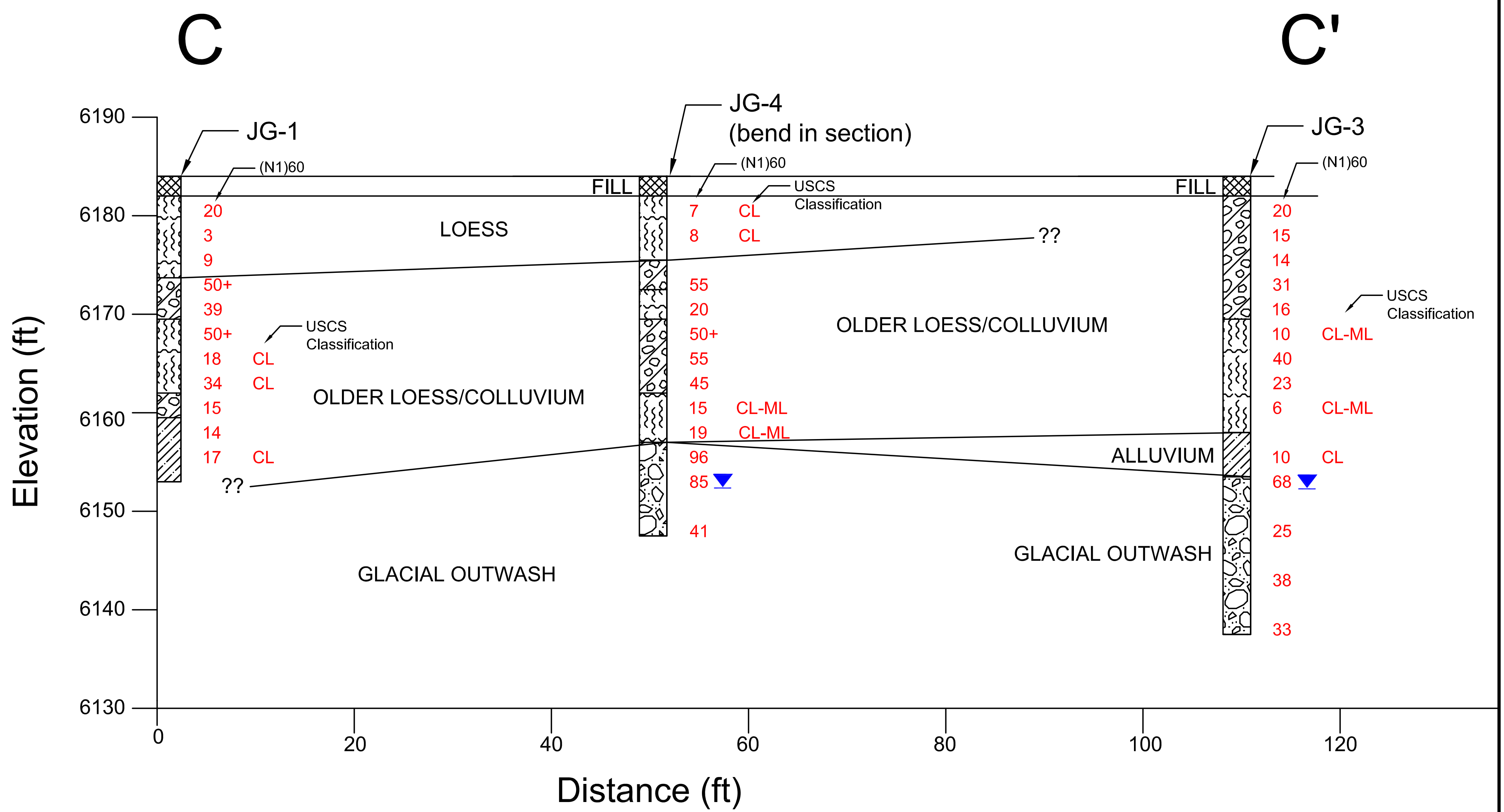

Scale:
Horizontal 1" = 15'
Vertical 1"= 15'

Groundwater level at time of investigation

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PROJECT NUMBER	09040.02

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FIGURE 5
GENERALIZED GEOLOGIC
CROSS-SECTION C-C'

PROJECT TITLE:
WEST VIEW TOWNHOMES
1255 W HIGHWAY 22
JACKSON, WY

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4.4 Soil Descriptions

As discussed above, the site stratigraphy is made up of wind-blown loess, gravel and clay colluvium interbedded with older loess, alluvial lean clays, and stony glacial outwash deposits. Generalized geologic cross-sections A-A', B-B', and C-C' (Figures 3 through 5) illustrate our interpretation of the contacts between soil layers. The cross-sections are a graphical representation of approximate stratigraphic relationships, and do not necessarily allow prediction of subsurface conditions at any location other than the borings and test pits themselves. Below is a summary of soil descriptions, standard penetration tests, and laboratory test results organized by material origin. Descriptive borehole logs are in Appendix A and complete laboratory test results are in Appendix C.

4.4.1 Fill

As described above, the upper level of the site is covered with aggregate surfacing while the lower level is paved with asphalt concrete. Where observed in the borings, fill was encountered to approximately 2-ft below the ground surface. No samples were taken of the fill and properties were estimated from material returned to the surface with the augers. The fill was described in the field as dry, gray, rounded to subrounded gravel in a silty sand matrix. All fill appears to be too shallow to affect the foundations. Since fill encountered at the site is relatively thin, it has not been incorporated into the stratigraphic model used for stability analysis (see Section 5.1.2).

4.4.2 Loess

Wind-blown loess was observed near the surface in all borings except in JG-3, where the grading of the site's lower level may have removed approximately 20-25 feet of material. Wind deposited clayey silt loess typically "blankets" the existing surface topography wherever it is deposited, in this case on top of layers of colluvium and older loess. In general, the younger loess was described in the field as moist, tan brown with white calcite deposition, very soft to medium stiff, and massive with pinhole voids. Occasional stones derived from rock types known to be located uphill were observed in samples. These are presumed to have rolled down slope and were incorporated into the loess as it was being deposited.

Adjusted SPT blow counts, or $(N_1)_{60}$ values (i.e., adjusted to an equivalent pressure of one atmosphere and standard hammer energy efficiency of 60%), are in the range 3 to 20 blows per foot (bpf). Higher blow counts (e.g., JG-1 D1, JG-5 D3, JG-5 D5, and JG-6 D3) are due to the influence of stones and if these results are excluded, the average $(N_1)_{60}$ value is 7 indicating the loess, on average, has a medium stiff soil consistency. Our experience has been that the silty loess typically is stiff, particularly when dry. Adjusted blow counts in the loess observed in BH-1 (WAI, 2011) were on average higher than observed during this investigation (range of 14 to 22). This may be due to drier soil conditions in October of 2011 than in June of 2016.

Laboratory tests of samples indicate in-situ moisture content of loess samples range from 12.7% and 29.0%. The fines content (silt and clay finer than the #200 sieve) of select specimens ranges from 79.7 to 96.6% with an average of 91.2%. Three hydrometer tests were performed indicating clay content (i.e., fraction of particles < 0.002mm) ranges from 20.9% to 31.1%. Samples have liquid limit (LL) values of 23 to 37 and plasticity indices (PI) of 3 to 17. Samples classify as CL (lean clay with sand), ML (lean silt with sand), or CL-ML (low plastic silt and clay with sand) in the Unified Soil Classification System.

Consolidation tests were conducted on three relatively undisturbed samples of silty loess. The specimen JG-4 U1 taken from 7.5-ft bgs had an in-situ moisture content of 22.3% and a dry density of 68.5 pcf. The specimen was saturated under a load of 2,000 psf with sudden settlement, or collapse, of 3.7%. Specimen JG-6 U1 taken from a depth of 7.5-ft bgs had a moisture content of 13.7% and a dry density of 77.8 pcf. Specimen JG-6 U2 taken from a depth of 10-ft bgs had a moisture content of 14.4% and a dry density of 73.1 pcf. The two specimens from JG-6 were subjected to a double oedometer type consolidation test. Specimen JG-6 U1 was consolidated at in-situ moisture while JG-6 U2 was consolidated under saturated conditions. The result is being able to estimate the collapse potential at a range of applied stresses, which is summarized in Table 4-1 below. For your convenience, we have attached an article regarding construction in loess soils as Appendix D.

Table 4-1: Collapse Potential Estimated from Double Oedometer Testing of JG-6 U1 and U2

Applied Stress (psf)	Estimated Collapse Potential
500	2.8%
1000	3.9%
2000	5.3%
4000	6.3%
8000	7.6%

4.4.3 Colluvium and Older Loess

Underlying the younger loess deposit in most of the borings are interbedded layers of colluvium and older loess deposits. In general, colluviual deposits observed at the site are dominated by gravel in a matrix of sandy clay whereas the loess was observed to be massive deposits of clays and silts. In many of the borings, it was difficult to distinguish between gravity and wind-blown deposits as even the mostly fine-grained, massive deposits of loess contain stones. As such, we have chosen to treat these two as one layer within the site's stratigraphic model (see Figures 3, 4, and 5).

Most colluvial-type soil samples were described in the field as moist, brown, medium dense to dense, and intact comprising limestone, andesite ("basalt" on the geologic map), and sandstone gravel in a sandy clay matrix. The rock types in the collvium are consistent with the geology upslope on East Gros Ventre Butte. $(N_1)_{60}$ values ranged from 14 to 60 bpf with an average of 31 bpf. Several SPT tests met refusal on stones. The minimum adjusted blow count is from a sample of clayey sand with gravel, likely deposited at lower energy near the distal end of a debris flow.

The older loess was generally described in the field as moist, reddish brown, soft, and massive, often containing pinhole voids and calcite stringers. $(N_1)_{60}$ values ranged from 6 to 34 bpf with an average of 17 bpf. Many of the SPT tests may have been skewed upward by gravel in some of the samplers. Samples had fine contents within the range of 61 to 87%. Tested samples had in-situ moisture contents ranging from 10.3% to 26.4% with an average of 18.7%. Older loess specimens had LL values of 22 to 33 and PI values of 6 to 13. The presence of pinhole voids indicate this deposit have a very low density and is likely collapsible, as discussed for the younger loess above.

4.4.4 Alluvium

A relatively thin layer of, fine-grained clay deposits were observed in most borings immediately above the stony glacial outwash deposits. These deposits were in some cases logged as soft, but are generally massive and lack the laminations usually associated with lake beds. The origin of these materials is uncertain, though we have conjectured they might be alluvium associated with deposition of fine-grained clays and sands following the melt out of the Qg2 glaciers. Some clay deposits, such as observed in the bottom of JG-2, may have originated as overbank deposits from Flat Creek, the channel of which may have formerly wrapped around the hillside above Broadway and Highway 22, but appear to pinch out to the northwest. In the stratigraphic model of the site (see Figures 3, 4, and 5), we have assumed these deposits to be continuous though it is possible they are confined to discontinuous channels cut into the stony outwash. Adjusted SPT blow counts are in the range 5 to 17 blows per foot (bpf), with an average of 10 indicating soft to medium stiff consistencies.

In-situ moisture contents of alluvium samples from the borings range from 19.0% to 32.9%, in some cases (JG-2 D10 and D11) very near or exceeding the tested liquid limit of the specimen. Though not observed during the investigation and follow-up groundwater monitoring, it is possible there exists a perched groundwater table within these fine-grained deposits during the spring runoff season. This possibility has been incorporated into the stability analyses. Further discussion is in Section 5.1.3. The fines content of select specimens ranges from 55.9% to 93.0% with an average of 77.7%. Tested samples have liquid limit (LL) values of 26 to 43 and plasticity indices (PI) of 11 to 22. In general, samples classify according to the Unified Soil Classification System as CL (lean clay with sand or sandy lean clay, depending on the fraction of sand-sized particles).

During the investigation, we attempted to obtain a thin-walled tube of the material in JG-6. However, the sampler impinged on stony outwash and only 4-5 inches of fine-grained soil was recovered. In the lab, the soil was extruded and consolidated back to an estimated in-situ density and subjected to a three point direct shear test. The resulting drained strength parameters of the tested soil are $\phi' = 25.7°$ and $c' = 883$ psf.

4.4.5 Stony Glacial Outwash (Qg2)

The site is underlain at depth by stony glacial outwash deposits (Qg2), identified by the presence of quartzite roundstones. As can be seen in the geologic cross-sections, there appears to be at least one large step in the outwash deposit from borings in the upper level of the site (JG-5 and JG-6) to where it is observed lower level borings (JG-3 and JG-4). Borehole JG-5 was drilled to 70-ft bgs and JG-6 to 50-ft bgs and encountered continuous glacial outwash below an elevation of 6,187.2-ft and 6,179.1-ft, respectively. JG-3 was drilled to 46.5-ft bgs and JG-4 was drilled to 36-ft bgs with outwash was observed at an elevation of 6,153.3-ft and 6,157.2-ft, respectively. Stony outwash is assumed to underlie the alluvial deposits observed in JG-1 and JG-2, but the borings did not encounter outwash. As discussed in Section 4.3, steps in the surface of represent erosional features similar to the terraces of the Snake River floodplain north of Jackson.

4.5 Surface Observations

Signs of actual or potential slope instability including, but not limited to, cracks, subsidence, seepage, excessive moisture, ponding, and/or slumping were not observed at the site during the field investigation.

4.6 Groundwater

Groundwater was encountered in all but two of the borings at an approximate elevation of 6,152-ft AMSL at the time of the investigation. Three VW piezometers were installed to monitor groundwater fluctuations in the weeks following the site investigation. Water surface elevations measured in JG-3-P1 and JG-5-P2 ranged from approximately 6,145-ft to 6,147-ft with approximately 0.5-ft between instruments indicating level groundwater conditions across the site. Maximum levels were 6,146.6-ft and 6,147.0-ft in JG-3-P1 and JG-5-P3, respectively. Piezometer JG-5 P2, installed within the clay alluvium on top of the stony outwash at 32-ft bgs, did not measure a water surface. Groundwater appears deep enough to not pose an issue with the proposed construction. Complete monitoring data and a representative graph are included as Appendix E.

4.7 Earthquakes and Seismicity

Jackson Hole is located within the Intermountain Seismic Belt, a zone of seismicity that extends from southern Utah through eastern Idaho and western Montana and encompasses western Wyoming including the Teton Range (Smith and Arabasz, 1991). The Teton Fault is considered an important structural element of the Intermountain Seismic Belt. The fault trace is believed to end at Teton Pass. Machette suggested that the “active” portion of the Teton fault terminates north of Wilson near Phillips Canyon and estimates that slip rates along the active fault north of Phillips Canyon are less than 0.2 mm/yr (i.e., very low). Ancient faults such as the Jackson Thrust and the Cache Creek Thrust have been mapped very near the project site but are very old and not considered active.

Ground motion accelerations should be derived for the project site in accordance with the general procedure defined in the International Building Code (IBC). The IBC references ASCE 7 to determine the ground motion accelerations. Based on the subsurface soils, the site should be classified as Seismic Site Class D ("Stiff Soil") with a risk category of I/II/III. For your convenience, USGS Seismic Design Maps Summary and Detailed Reports were produced and are attached as Appendix F. Structural designers will be responsible for ensuring seismic loads are applied to the structure according to the appropriate codes.

The site (Latitude: N 43.5°, Longitude: W 110.8°) is in an area of moderate seismic activity. The current peak horizontal acceleration (PGA) with 10% probability of exceedance in 50-years is approximately 0.198g, according to the USGS National Seismic Hazard Maps (2008). This has been applied in this report for analysis of seismic lateral loading on retaining walls (see Section 6.3) and for pseudo-static seismic slope stability analysis (see Section 5.1.4).

The provisions of the IBC are intended to provide uniform levels of performance for structures, depending on their occupancy and use and the risk inherent to their failure. The approach adopted in the IBC is intended to provide a uniform margin of safety against collapse at the design ground motion. The design earthquake ground motion is selected at a ground shaking level that is 2/3 of the maximum considered earthquake (MCE) ground motion, which has a likelihood of exceedance of 2 percent in 50 years (a return period of about 2,500 years). The owner should be aware that the IBC is not intended to prevent damage or loss of function during a major earthquake. It is intended to reduce the risk of loss of life.

4.8 Geologic Hazards and Liquefaction

The owner should be aware that in the event of a large magnitude earthquake, there are several geologic hazards that could potentially cause damage to structures (Smith et al, 1993). Potential hazards at this site might include strong ground shaking, ground cracking, and surface rupture along a concealed fault trace. The owners may wish to consider the option of carrying earthquake insurance in addition to homeowner's insurance.

Loose, saturated sands and silty sands, and in some cases, silts and gravels, may liquefy when exposed to seismic shaking. The gravel at depth encountered at this project site appears too stony to liquefy in a seismic event. There is a relatively small risk that liquefiable sands occur at greater depth. Groundwater appears too deep to affect the clays and silts above the outwash gravels.

5.0 SLOPE STABILTIY ANALYSIS

5.1 Stability Analysis Methodology

Slope stability analyses were performed using GEO-SLOPE International's SLOPE/W limited equilibrium program (GeoStudio 2012, V8.15). The following methodology was performed in order to develop the stability model:

5.1.1 Geometry

Two cross-sections were selected to be representative of the site. Cross-section locations may be seen on Figure 2. External geometry (i.e., ground surface) of the cross-sections were developed using topographic data from a survey performed by this office in June 2016 and historical aerial photography from the Teton County GIS website. Internal geometry (i.e., subsurface conditions) was developed using the borehole data collected from the site investigation. Contacts between material types were interpreted so as to create a reasonably conservative model based on our predictions of soil origin and understanding of local geology. Figures 3 and 4 show the modeled cross-sections and predicted external and internal geometry.

The surface of the stony glacial deposits is assumed to be made up of two to three outwash terraces. We connected the terraces assuming an angle of repose of 35° from the surface of the outwash observed in the upper borings (JG-5 and JG-6). There is also a step about 8-ft high between JG-5 and JG-6, which is not represented in the 2-dimmensional stability but probably does not adversely impact the slope. Alluvial clays deposited on the stony outwash are also assumed to have been originally level. It is expected the clays were eroded from the face of the terrace during the development of the lower terrace and were not continuously modeled from the upper level to the lower level.

Slip surfaces were developed using an "Entry-Exit" definition with a circular slip surface. The program creates hundreds of slip surfaces by connecting points of the blocks and selects the critical slip surface as the one with the lowest Factor of Safety (FS). FS is the ratio of forces resisting slope failure divided by forces tending to cause failure. A FS of 1.0 indicates imminent slope failure. FS < 1.0 implies failure and FS > 1.0 implies stability.

5.1.2 Materials

Effective stress shear strength parameters pertaining to a Mohr-Coulomb strength model were estimated for the site soils. Shear strength consists of two parameters: cohesion (c'), which expresses the shear strength at zero overburden pressure, and friction angle (φ'), which expresses the relationship between overburden pressure and shear strength (i.e., that shear strength increases with loading, from a minimum of c'). Unit weight is a measure of the soil's density or weight per unit volume.

The stratigraphic model is simplified into four different material models and soil parameters were applied using a combination of field estimates, direct lab testing, and correlations between SPT blow counts and index tests. A summary of the soil parameters applied to each material is shown in Table 5-1.

Table 5-1: Modeled Soil Parameters

Layer Name	Strength Model	Unit Weight (pcf)	Cohesion (c',psf)	Friction Angle (φ', degrees)
STONY OUTWASH	Mohr-Coulomb	135	0	35
ALLUVIUM	Mohr-Coulomb	120	100-800	25-30
OLDER LOESS/COLLUVIUM	Mohr-Coulomb	115	100	32
SILTY LOESS	Mohr-Coulomb	85	100	30

Theoretically, most soils in a drained condition do not have cohesion. However, apparent cohesion from soil matric suction and cementation are often present. When the material models of loess and older loess/colluvium are considered cohesionless, the critical slip surface found in the model tends to approach the "infinite slope" case.

A remolded sample of alluvial clay was tested for effective shear strength parameters using direct shear. The testing yielded $\phi' = 25.7°$ and $c' = 883$ psf. A correlation between the alluvial clay's plasticity index (PI) and peak effective friction angle (Ladd et al, 1977) indicates the soil is fairly strong. Using the maximum PI (22 from sample JG-5 D7) yields $\phi' = 30°$. As discussed in Section 5.2 below, critical slip surfaces (those with the lowest factors of safety, shown on Figures 6, 7, 8, and 9) did not extend deep enough to be affected by the shear strength of the clay.

However, in order to consider all cases, a deeper slip surface was manually selected and the shear strength of the clay was modeled parametrically using $\phi' = 25°$ while varying $c' = 100$ psf to 800 psf, FS values of Cross-Section A-A' ranged from 2.7 to 3.1 in a static analysis and from 2.0 to 2.3 when applying seismic conditions. Similarly, FS values in Cross-Section B-B' ranged from 3.3 to 3.6 and 2.3 to 2.6 in static and seismic analyses, respectively. These results indicate the changes in FS values of less than 15%.

5.1.3 Phreatic Surface

Groundwater at this site was observed at an approximate maximum elevation of 6,147-ft on June 20, 2016, within the stony glacial outwash. It is probable that the site investigation occurred early enough to capture the groundwater peak. However, it is likely that water surface elevations within the cross-sections may be higher during the spring snowmelt or heavy precipitation.

Samples of the older loess and alluvial clay near depths of 25 to 30 feet were tested to have moisture contents approaching the soils' liquid limits. It is possible a perched groundwater condition exists during snow runoff or following heavy precipitation. As a "worst case" condition we have added a phreatic surface to the models 5-ft above the surface of the alluvial clay.

5.1.4 Seismicity

The site (Latitude: N 43.5°, Longitude: W 110.8°) is in an area of moderate seismic activity. The current peak horizontal acceleration (%) with 10% probability of exceedance in 50-years is 0.198g, according to the USGS National Seismic Hazard Maps (2008). Seismicity is assessed in the slope stability models using a pseudo-static method with half the horizontal seismic load, or approximately k_h = 0.1g.

5.1.5 Building Loads

To model conditions after project completion, the geometry was altered to account for anticipated excavation. Foundation loads were modeled by averaging an assumed footing load over the length and width of the building and applying it as a 1-ft thick soil layer with a unit weight 500 pcf. It is our understanding Superior Wall® foundation walls, buried approximately 4-ft deep, will be backfilled in preparation of the floor slab. Thus the backfill was also added into the model as a soil with a unit weight of 110 pcf. For the building at the toe of the existing slope, a point load was added to estimate the effect of the foundation wall. This load was positioned ⅓H above the bottom of the wall with a magnitude equal to the active lateral pressure resultant uphill of the building (see Section 6.3.1). The modeled height of retained soil (H) of Cross-Section A-A' is approximately 8.5-ft and the calculated resultant force (R) is 2,100 lb. In Cross-Section B-B', H = 8.0-ft and R is approximately 1,888 lb.

5.1.6 Analyses

The slope stability analyses were performed using the SLOPE/W stability module of GeoStudio 2012 version 8.15.1.11236, produced by GEO-SLOPE International, Ltd. The Morgenstern-Price limit equilibrium method, which takes into consideration moment and force equilibrium, was used to analyze slope stability. Schematic cross-sections are shown on Figures 3 and 4 and SLOPE/W output figures are presented in Figures 6 through 9.

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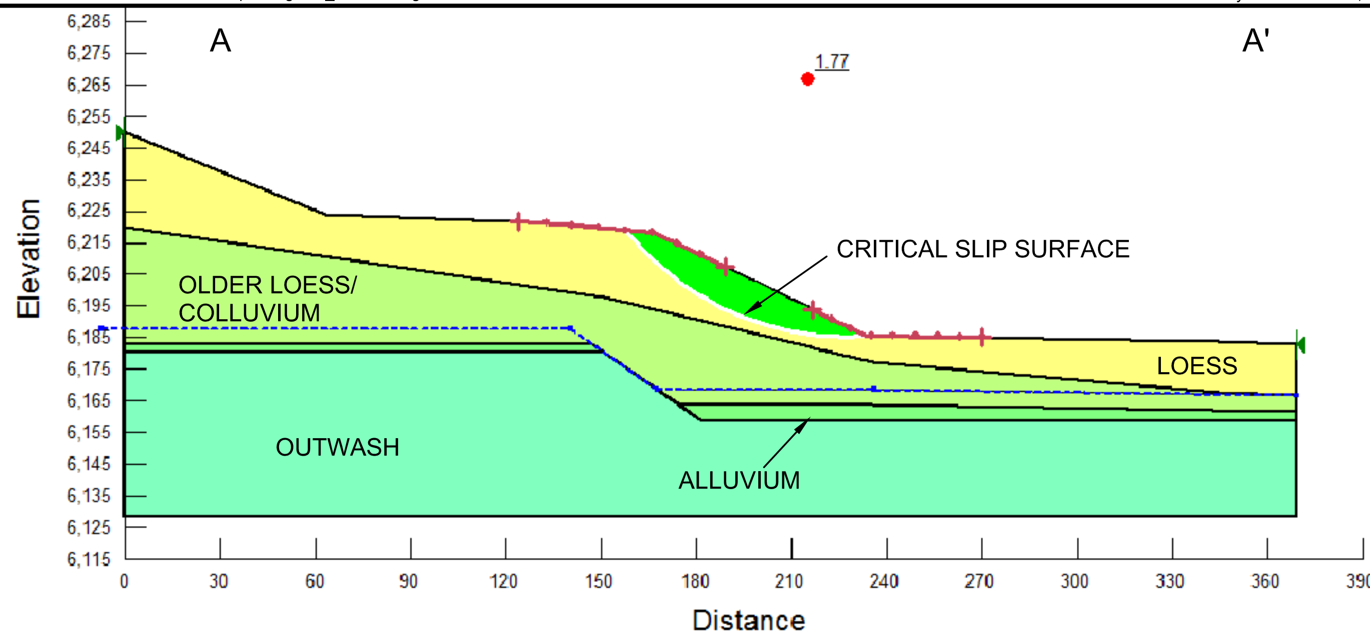
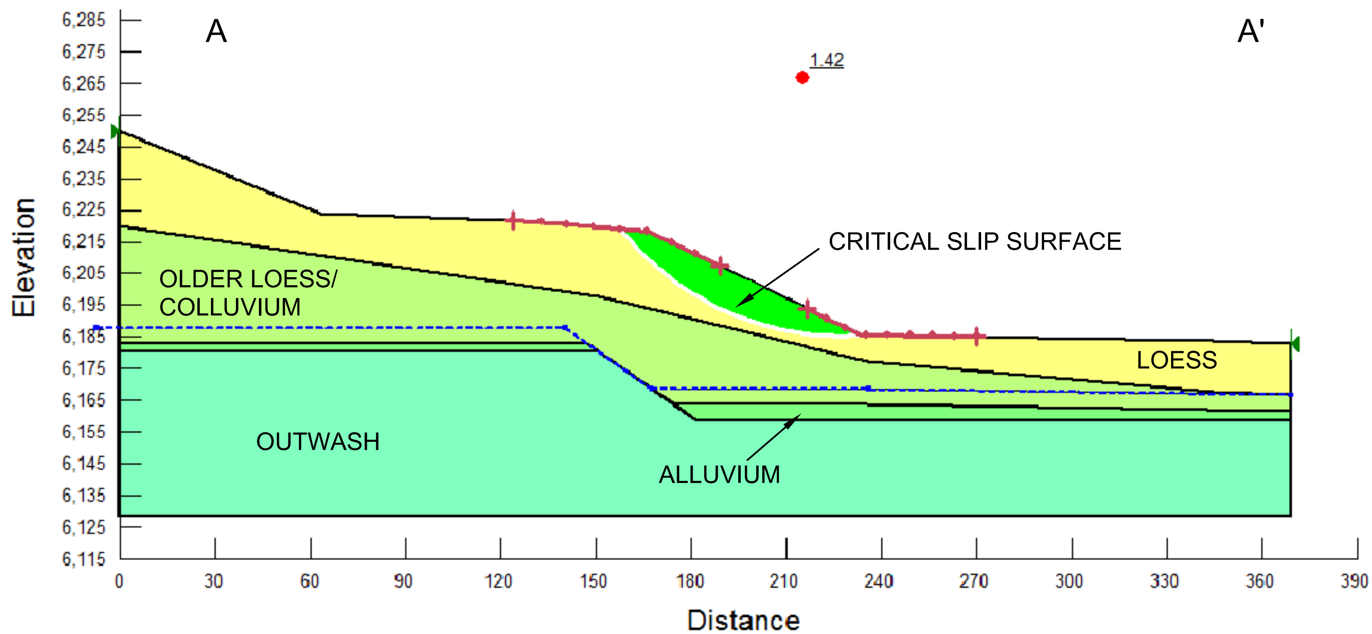
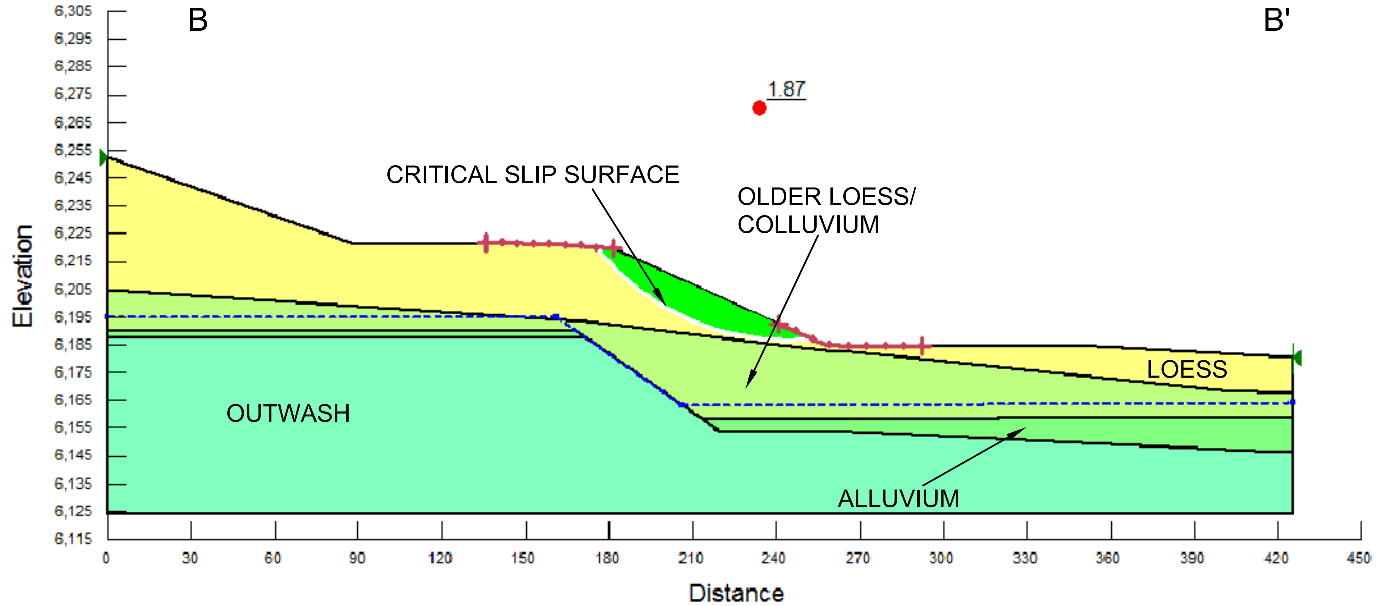


Cross-Section A-A' Existing Conditions - Static Analysis

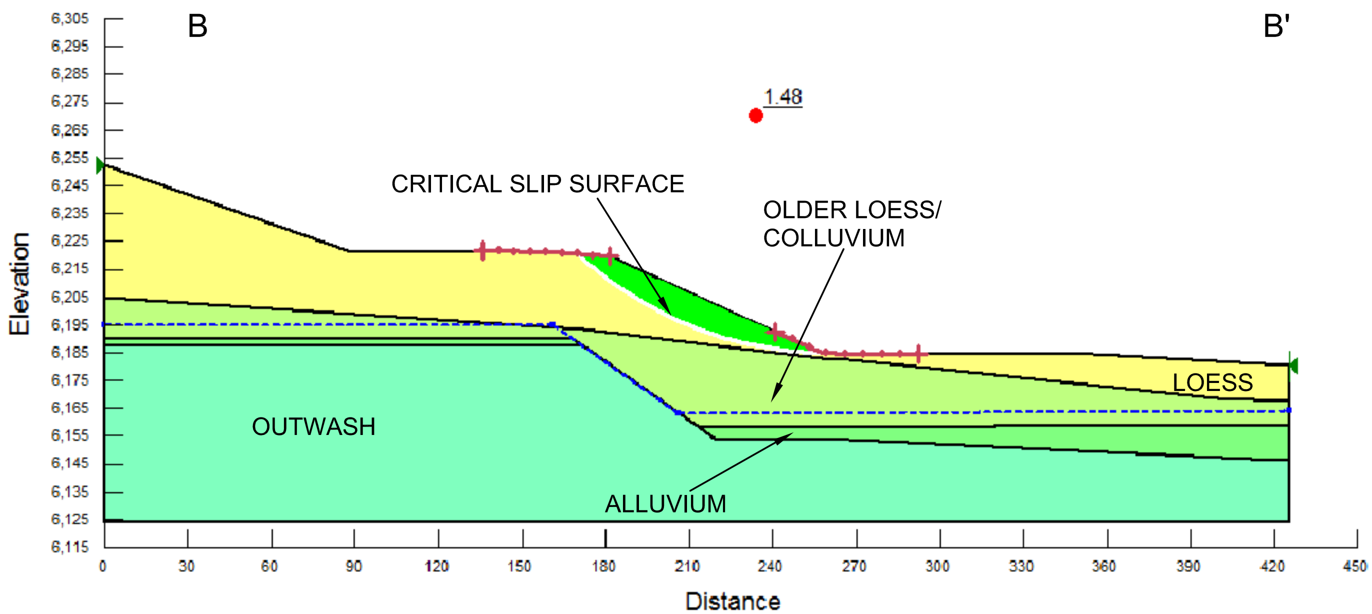


Cross-Section A-A' Existing Conditions - Seismic Analysis

DRAFTED BY: CHL REVIEWED BY: RW PROJECT NUMBER 09040.02	SHEET TITLE: FIGURE 6 STABILITY CROSS-SECTION A - A' EXISTING CONDITIONS	PROJECT TITLE: WEST VIEW TOWNHOMES 1255 W HIGHWAY 22 JACKSON, WY	JORGENSEN Geotechnical, LLC 307-733-5150 www.jorgeng.com



Cross-Section B-B' Existing Conditions - Static Analysis



Cross-Section B-B' Existing Conditions - Seismic Analysis

DRAFTED BY:	CHL
REVIEWED BY:	RW
PROJECT NUMBER	09040.02

SHEET TITLE:
FIGURE 8
STABILITY CROSS-SECTION B - B'
EXISTING CONDITIONS

PROJECT TITLE:
WEST VIEW TOWNHOMES
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JACKSON, WY

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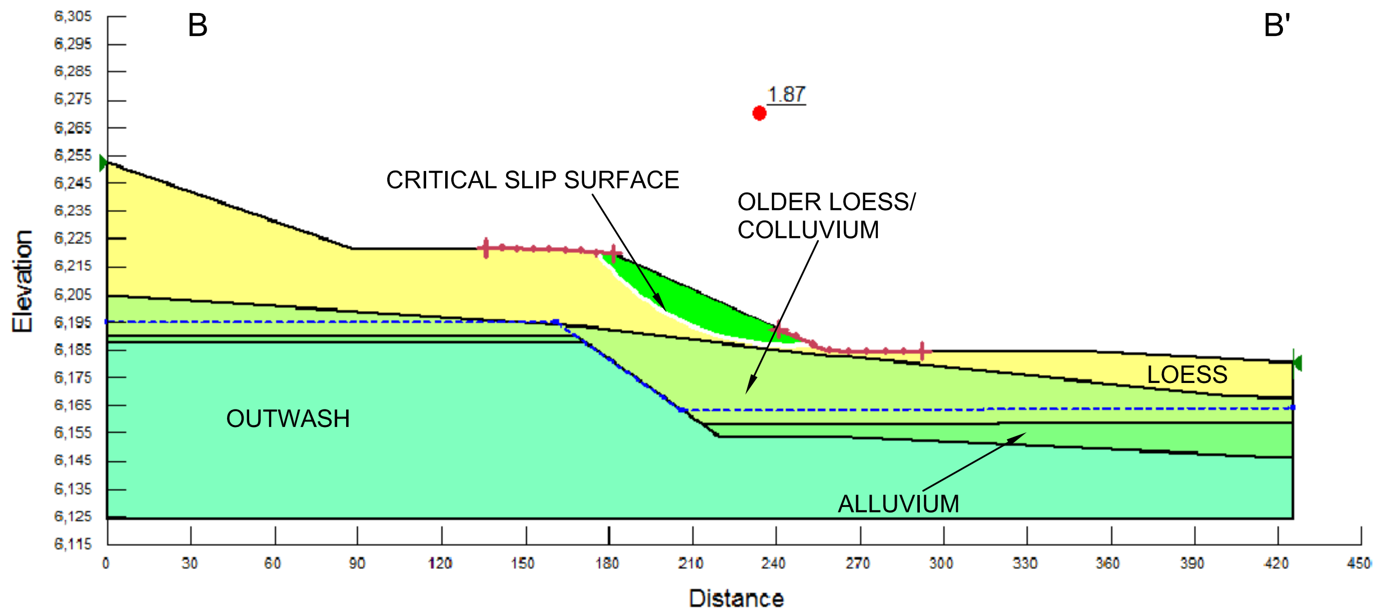
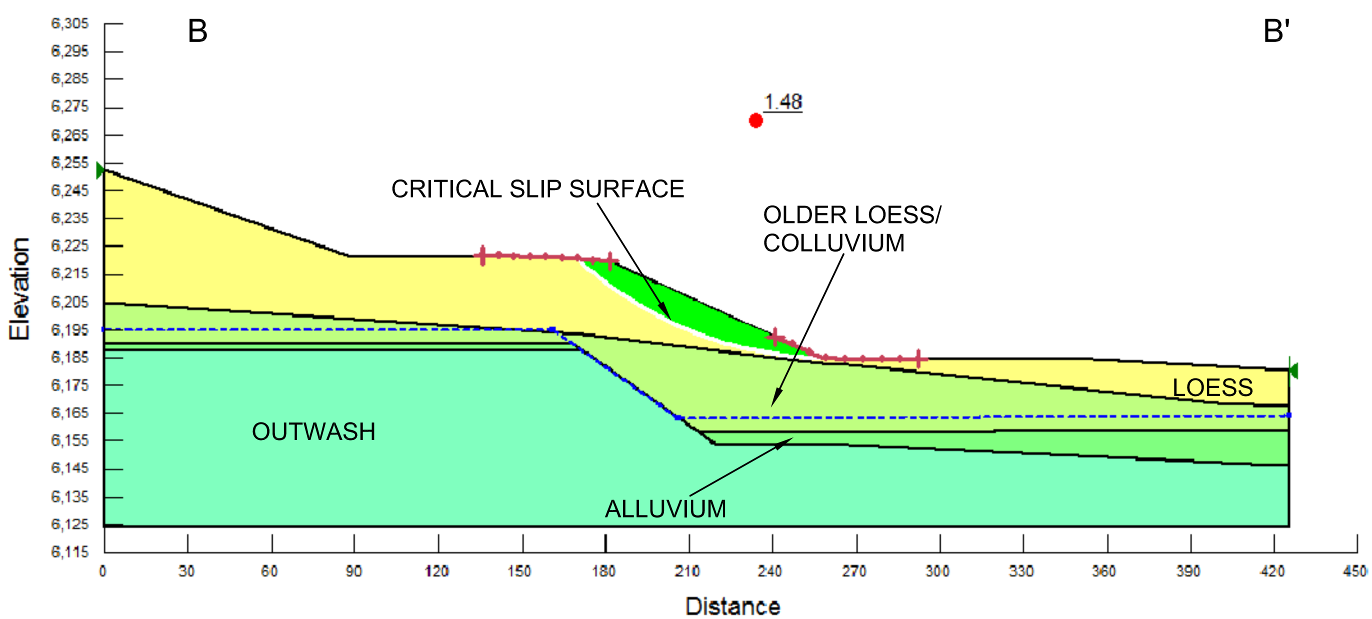


Cross-Section B-B' Existing Conditions - Static Analysis



Cross-Section B-B' Existing Conditions - Seismic Analysis

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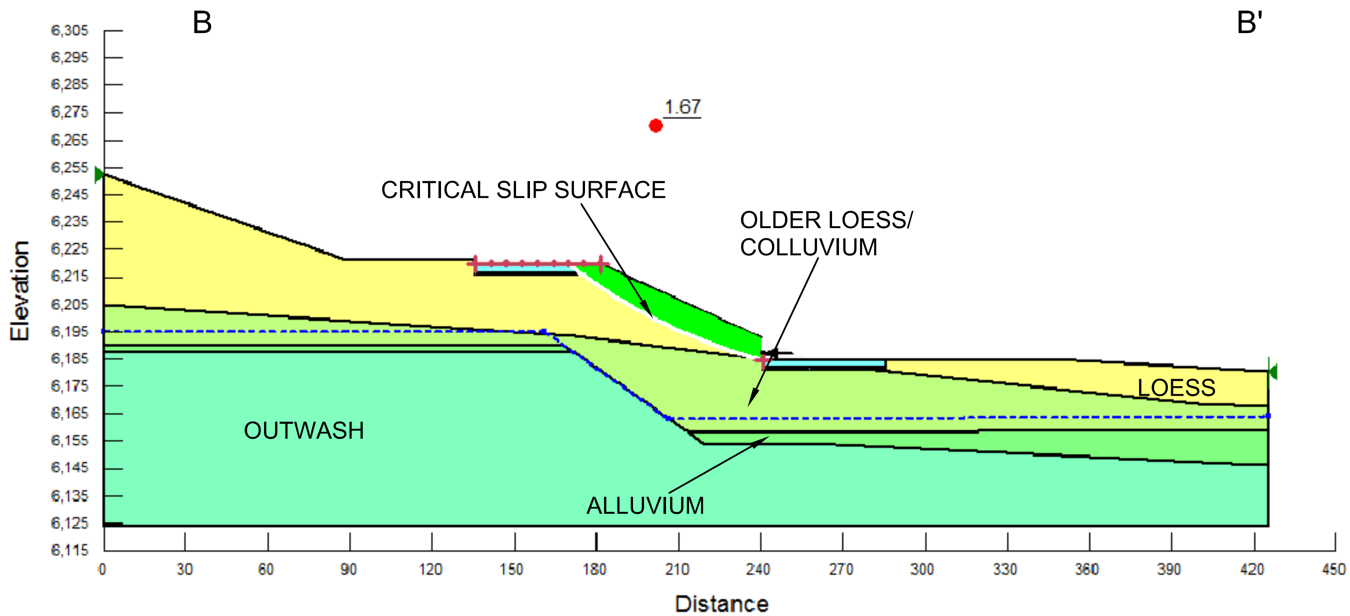
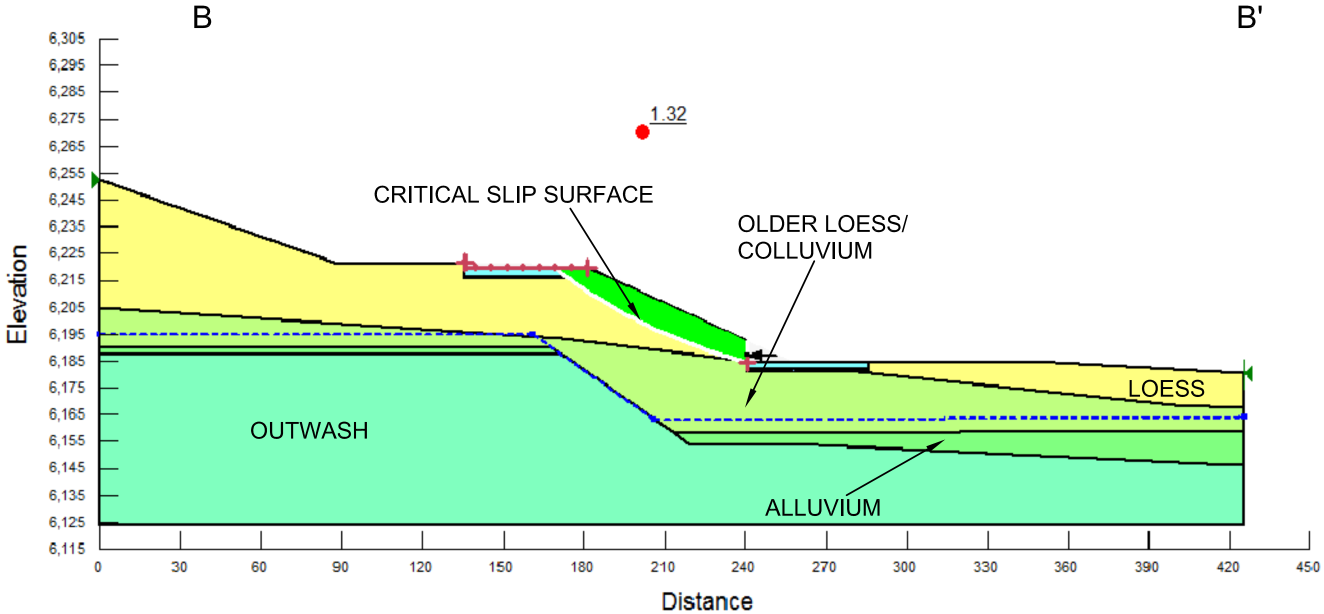


Cross-Section B-B' Post-Construction Conditions - Static Analysis



Cross-Section B-B' Post-Construction Conditions - Seismic Analysis

DRAFTED BY: CHL REVIEWED BY: RW PROJECT NUMBER 09040.02	SHEET TITLE: FIGURE 9 STABILITY CROSS-SECTION B - B' POST CONSTRUCTION CONDITIONS	PROJECT TITLE: WEST VIEW TOWNHOMES 1255 W HIGHWAY 22 JACKSON, WY	JORGENSEN Geotechnical, LLC 307-733-5150 www.jorgeng.com

5.2 Stability Analysis Results

Figures 6 through 9 show the modeled output of the slope stability analyses with the critical slip surface highlighted. Table 5-2 presents factors of safety for each condition analyzed.

Table 5-2: Summary of Stability Analyses Results

Cross-Section	Analysis Condition	Modeled Factor of Safety
A-A' Existing Conditions	Static	1.77
	Seismic	1.42
A-A' Proposed Project	Static	1.56
	Seismic	1.24
B-B' Existing Conditions	Static	1.87
	Seismic	1.48
B-B' Proposed Project	Static	1.67
	Seismic	1.22

In summary, the stability analyses indicate the analyzed cross-sections are stable under static and seismic conditions. Critical slip surfaces generated by the modeling software do not appear to extend deep enough to be affected by the modeled phreatic surface (i.e, groundwater) or to encounter the alluvial clay. When deep slip surfaces are extended to the weakest soil layer encountered during the investigation (i.e., alluvial clays), factors of safety are high. Soils at the site appear stiff (i.e., strong) and the site investigation did not encounter any underlying structure that would indicate unstable conditions.

5.3 Stability Modeling Limitations

This analysis has been performed to assess the global stability of the site and the impacts of the proposed project after completion only. Depending on construction plans and details, further stability analysis may need to be performed. For instance, excavation for the buildings at the toe may require temporary construction shoring. This office is prepared to perform follow-up modeling, slope stability analysis, and shoring design to support construction, if requested.

6.0 ADDITIONAL ENGINEERING ANALYSIS

6.1 Settlement

Loess is the most problematic material at the proposed West View Townhomes site and was encountered at proposed footing elevations in nearly every investigative boring, the exception being JG-3 on the lower level of the site.

The wind-blown deposit has a very low density and may collapse when wetted. As described in Section 4.4.2 above, consolidation tests performed on soils sampled from this site indicate collapse potential ranging from approximately 4 to 6.5% over the range of anticipated foundation loads. To put this in terms of settlement, consider the following. The zone of influence from a typical spread footing extends to an approximate depth of twice the footing width (2B, where B = footing width). For a 2-ft strip footing, the depth of influence is then 4-ft below the bottom of footing. If the soil within the zone of influence were to become saturated, settlement on the order of 2 to 3 inches may be expected.

Collapse settlement tends to occur locally, as a result of unusual moisture events, such as broken sprinkler or water service lines, or concentration of surface water adjacent to foundations due to poor surface runoff control. Collapse settlement is usually highly differential and therefore particularly damaging. In our opinion, it should be assumed that any loess encountered at the site is collapsible and should be addressed accordingly.

We recommend three alternatives, depending on the thickness of loess, to prepare the foundation subgrades to reduce the risk of excessive differential settlement: over-excavation and replacement of the native loess, deep foundation elements (such as helical piers), or over-excavation and re-compaction of the silty soil.

6.1.1 Over-Excavation and Replacement of Native Loess

It appears the historical grading of the site removed a considerable amount of overlying younger loess and it may be possible to remove the remaining deposits down to the surface of stony deposits of colluvium for portions of the structures proposed along the toe of the existing slope. Loess was observed to depths of 10.3-ft bgs in JG-1 and 8.5-ft bgs in JG-4. If the depth of foundations near the front of the proposed units are installed at a depth of 3.5-ft below the existing ground surface, additional excavation to reach the surface of the colluvium will be approximately 5 to 7 feet. This approach may not be feasible for the entire structure due to the constraints of the existing slope, but could represent a time or cost savings by not requiring moisture conditioning and re-compacting the native soil (Section 6.1.2).

Excavation of the native loess option should extend a footing width (B) beyond the edge of the footing to the surface of the underlying stony layer and structural fill should contact directly with the colluvium, as illustrated on Figure 10. Replacement material shall be approved structural fill, such as locally sourced sandy gravel and cobble (i.e., “pit-run”). Significant settlement of the stony colluvium, or structural fill in contact with the colluvium, is not anticipated. Pit-run is easy to compact, but requires very careful drainage control to prevent storage of water in contact with underlying native soil ("bathtub effect"). Careful observation by a qualified observer is critical to performance of engineered fills.

Prior to fill placement, pre-roll the surface to compact materials that have been disturbed during excavation using a smooth drum vibratory roller (in vibratory mode) with a minimum of three passes. The actual number of passes should be determined by observing whether the surface is yielding after each pass. If the surface appears to be yielding, the number of passes should be increased until a non-yielding condition is observed. A representative of this office should observe the surface of the native soil prior to the placement of fill.

Place the structural fill in lifts and compact using the method specification described in Table 6-1. Pit-run or other clean, stony material will compact into a dense, strong, well-drained structural fill, and tight moisture control is usually not required. A vibrating roller-compactor is required for adequate compaction of granular material. Compaction of stony material with a sheepsfoot roller is not recommended. Pit-run gravel usually requires minimal compactive effort, and due to the stony nature of the materials, nuclear density testing can yield variable compaction results. If reasonable compactive effort is made on the lifts of pit-run, compaction testing is not necessary.

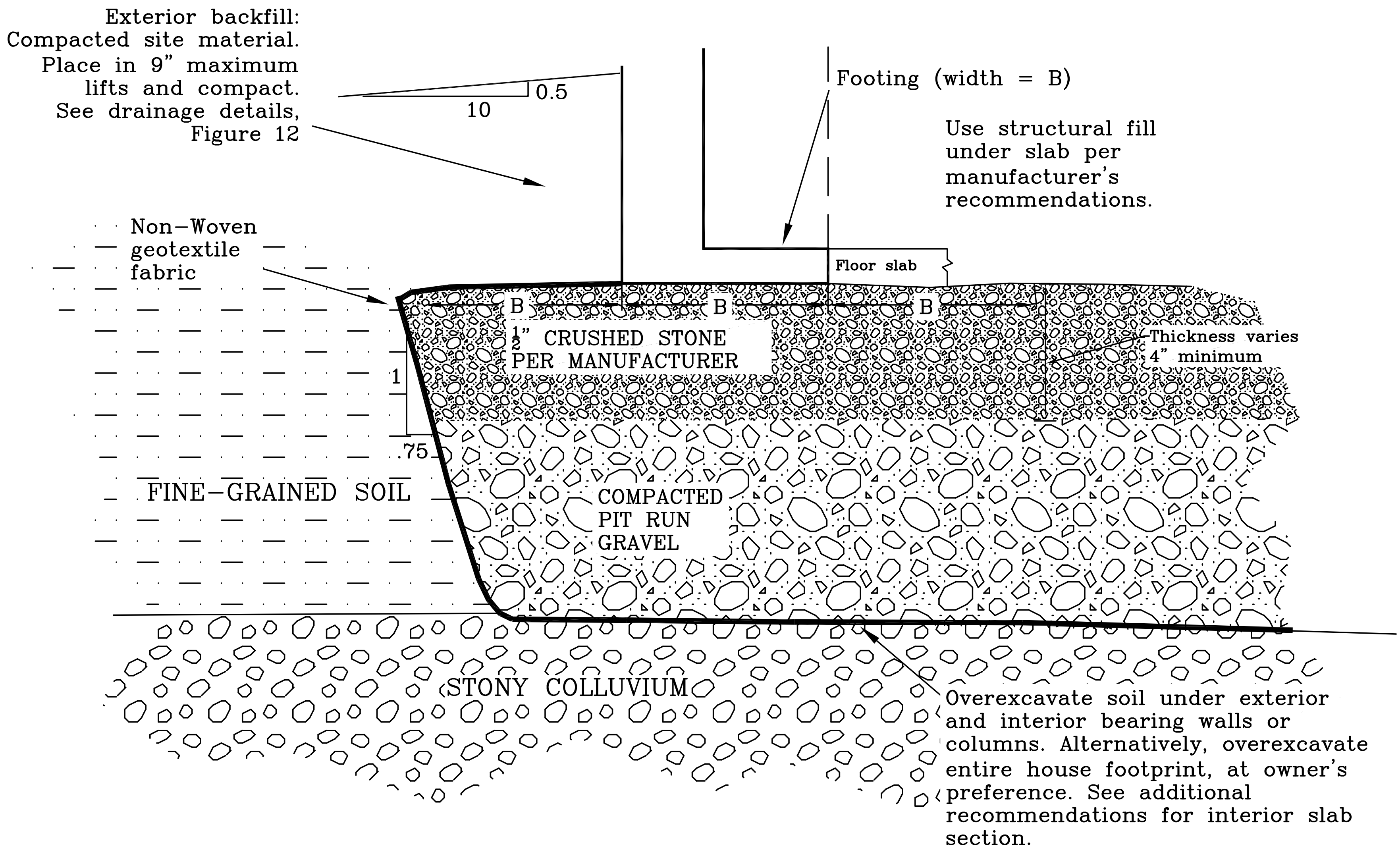
Table 6-1: Compaction Method Specification for Stony Materials

Compactor Type	Lift Thickness	Number of Passes*	Maximum Particle Size
Hand held "whacker"	6-inches	5	4-inches
1.5 ton static weight	9-inches	5	6-inches
5 ton static weight	12-inches	3	9-inches **

*The actual number of passes should be determined by observing whether the surface is yielding after each pass. If the surface appears to be yielding, the number of passes should be increased until a non-yielding condition is observed.

** Occasional 12-inch stones are allowable, but avoid nesting.

Pit-run fill should be placed in a maximum loose lift thickness of 9-inches, unless a large roller is available, in which case a 12-inch loose thickness would be acceptable. A minimum of three passes with the vibratory roller should be applied to each lift. The actual number of passes should be determined by observing the compaction after each pass to determine if the surface is non-yielding. If the fill surface appears to be yielding, the number of passes should be increased until a non-yielding condition is observed. Fill should be placed in horizontal lifts. Moisture conditioning is usually not critical, but may enhance the process.



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Note: Sub–slab drainage may not be required depending on structural fill chosen. Geotechnical Engineer approval required.

DRAFTED BY: MW REVIEWED BY: CHL PROJECT NUMBER 09040.02	SHEET TITLE: FIGURE 10 OVER-EXCAVATION AND REPLACEMENT SCHEMATIC	PROJECT TITLE: WEST VIEW TOWNHOMES 1255 W HIGHWAY 22 JACKSON, WY	JORGENSEN Geotechnical, LLC 307-733-5150 www.jorgeng.com

6.1.2 Deep Foundation Elements

A majority of the site is covered by thick deposits of loess where over-excavation and replacement is not a viable alternative. Deep foundation elements such as helical piers bearing on the stony colluvium or stony glacial outwash will dramatically reduce the risk of settlement associated with collapse of the loess. Helical piers are commonly recommended in this region as they are easy to install and down-drag forces anticipated in the loess are negligible due to the slenderness of the shaft. Depth of helical piers may be significant, particularly for the units on the project's upper level. If this option is selected, test piers should be installed to determine anticipated depth and allowable capacities.

6.1.3 Over-Excavation and Re-Compaction of Native Loess

As an economic alternative to deep foundation elements, this office recommends over-excavating the fine-grained soil and re-compacting with careful moisture-density control. ***Please note that this method is not without risk*** since collapsible material remains below the improved material and there is a possibility that moisture could affect this remaining soil. This option is not bad practice and we have successfully constructed numerous projects using this technique; it just comes with more settlement risk than helical piers. The risk is difficult to quantify, as settlement events in collapsible soils tend to be episodic. However, it is important that the owner/contractor understand that choosing this option over deep foundation elements is choosing a higher risk of settlement over the life of the building.

When all of the loess is not removed from beneath footings, it is preferable to compact the natural soil because it is compatible with the remaining subgrade material and less vulnerable to collection of fugitive water. Many excavation contractors prefer to use pit-run as replacement fill because pit-run is usually easier to compact and less sensitive to moisture content. However, the pit-run may act as a moisture sink (i.e., "bathtub effect") and cause wetting of the adjacent fine-grained soil.

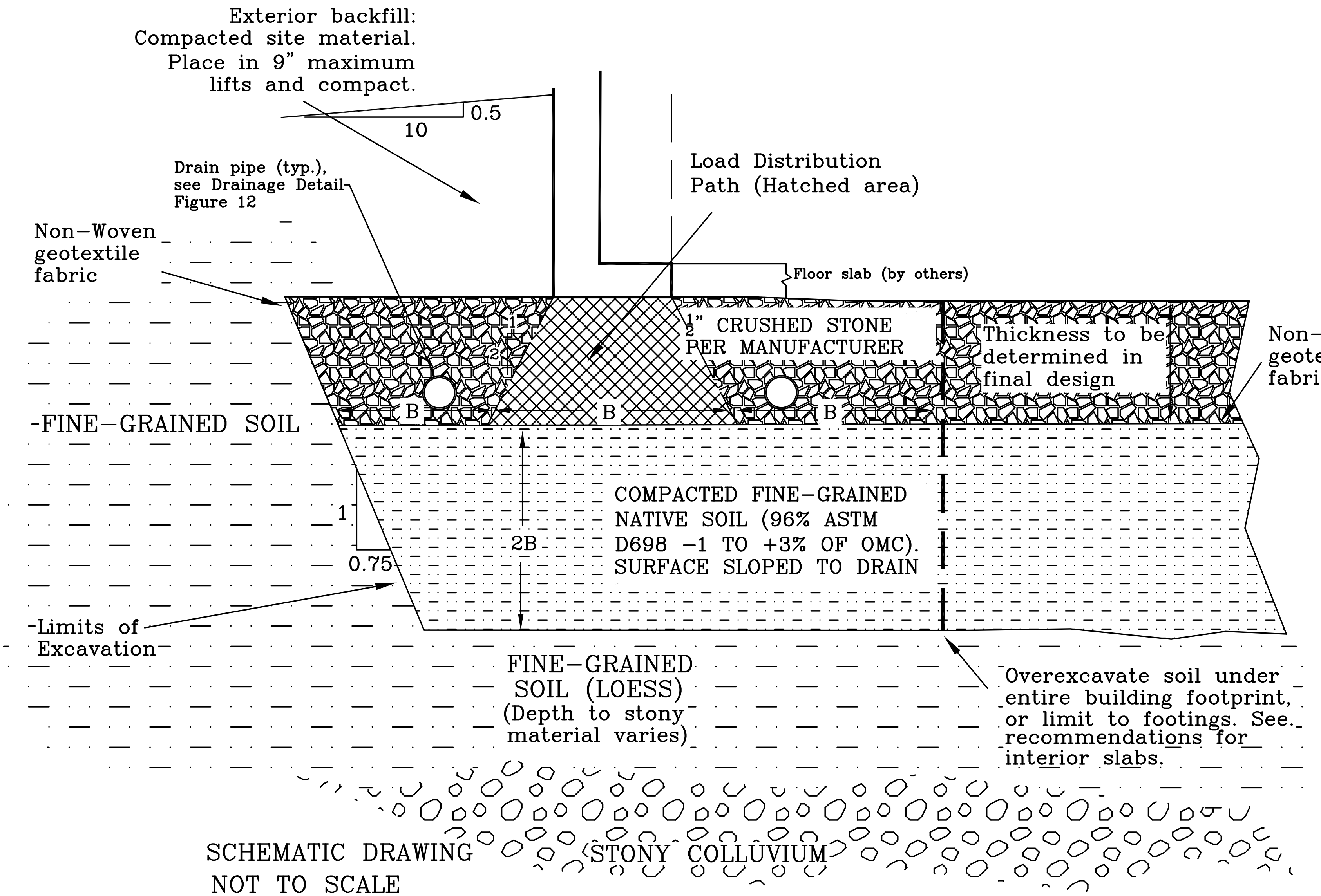
It should be noted that this method should only be performed with great care as moisture control and compaction are very difficult. It is our understanding that construction will begin toward the end of the summer or beginning of the fall. This is typically a drier time of year in Jackson. However, if plans change and construction begins in the spring or early summer, snowmelt and surface water runoff may be problematical. Freezing temperatures in the late fall or winter also pose problems with moisture control. The most common cause of foundation failure is wetting of soils below foundations during construction. Therefore, temporary drainage diversions may be necessary to divert water from the foundation excavations. Careful planning of foundation construction is required to maintain positive drainage across the site and subgrades must be protected from freezing.

The Superior Wall® foundation system uses aggregate to transfer load to the underlying soil subgrade. It is standard practice to assume the pressure distribution under a footing spreads out at a 1/2V:1H slope. The width of the pressure distribution at the bottom of the aggregate has been considered the width of footing for analysis and recommendations and will depend on the final thickness of the aggregate. The thickness of crushed stone will depend on what is required to reduce the bearing pressure to the allowable pressure of the re-compacted loess (see discussion in Section 6.2).

Loess should be excavated at least two footing widths (2B) from beneath the aggregate and at least one footing width (B) on either side of the modified pressure distribution, as shown in Figure 11. This volume is often described as a footing's zone of influence, as foundation loads are estimated to be low enough outside this region to not affect the soil. It may be easier and certainly safer to excavate below the entire footprint of the building (i.e., below both footings and slabs). If the excavation is not extended to the entire footprint of the building, loess under interior slabs-on-grade must be improved as described in Section 7.4.

Native loess soils must be compacted to a minimum dry density of 96% ASTM D698. The surface of the compacted loess should be graded at a minimum of 0.5% toward the pipes of the drainage system. Natural soils should be compacted near or slightly wet of optimum moisture content, between -1% and +3% of optimum. If the material is compacted dry of optimum it may still be collapsible. It is also very important to follow proper procedures for moisture blending and compaction. Soils must be thoroughly mixed with water at the surface and turned several times using a grader or disk. **It is unacceptable to place fill lifts and spray the material in the excavation.** The water will penetrate only a short distance into the lift and the material will compact poorly.

A sample of the soil should be obtained as early in the construction process as possible and submitted to Proctor compaction testing, per ASTM D698. In the test, soil at a range of moisture contents will be compacted using the same effort. The result is a curve relating moisture content to dry density, allowing us to determine optimum moisture and maximum dry density. It will also be important to provide density testing with a nuclear density gauge and supervision during fill placement. Testing should occur in each compacted lift for quality control. This office is available to provide these services.



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DRAFTED BY:	MW	SHEET TITLE:	PROJECT TITLE:	JORGENSEN Geotechnical, LLC 307-733-5150 www.jorgeng.com
REVIEWED BY:	CHL	FIGURE 11	WEST VIEW TOWNHOMES	
PROJECT NUMBER 09040.02		OVER-EXCAVATION AND RE-COMPACTION SCHEMATIC	1255 W HIGHWAY 22 JACKSON, WY	

6.2 Bearing Capacity

Bearing capacity of soil refers to its ability to resist shear failure under load. Bearing capacities have been calculated using Terzaghi's bearing capacity equation for isolated and strip footings (Bowles, 1996) for two soil conditions: 1) stony colluvium or stony structural fill in contact with the colluvium and 2) re-compacted loess. Bearing capacity values for re-compacted loess have also been calculated for footings on a slope for the upper two proposed structures. See Table 6-2. This office should inspect exposed foundation subgrade soils in to verify assumptions made during design.

Table 6-2: Summary of Bearing Capacity Calculations

Soil Type – Foundation Condition	Calculated Bearing Capacity
Stony Colluvium or Compacted Fill	4000 psf
Compacted Loess – Level Ground	2500 psf
Compacted Loess – Top of 26.5° Slope	1500 psf

Presumptive pressures were derived based on visual classification of the soil assuming the recommendations of this report are followed. The calculations are also based on a general understanding of the proposed foundation system. Design may be improved iteratively if this office is provided a foundation plan with footing loads as the project progresses.

6.3 Lateral Loads on Foundation Walls

Lateral pressures were calculated using methods suggested by Bowles (1996) for anticipated exterior backfill: silty loess or stony, silty colluvium (see Table 6-3). Equivalent fluid pressures (γK) will vary based on the slope of the ground surface adjacent to foundation or retaining walls. Lateral pressures were calculated for active and at-rest conditions assuming a ground surface sloping up at an angle of 26.5° (2H:1V slope) from the structure and passive pressures were calculated assuming a ground surface sloping down at the same angle. Pressures are calculated for static and seismic conditions.

Table 6-3: Lateral Pressure Parameters for Compacted Exterior Backfill

Condition	Coefficient of Earth Pressure	γK (equivalent fluid pressure)*
Static Conditions Sloping Backfill**	$K_o = 0.9$ $K_a = 0.53$ $K_p = 1.13$	γK_o = 99 pcf γK_a = 59 pcf γK_p = 124 pcf
Seismic Conditions Sloping Backfill**	$K_{ae} = 0.76$ $K_{pe} = 0.93$	γK_{ae} = 84 pcf γK_{pe} = 103 pcf

* Assumes a soil unit weight of 110 pcf with a friction angle of 30 degrees

** Slope is assumed to be 2H:1V (26.5°) adjacent structures

6.3.1 Active Pressures

For lateral pressure design of retaining walls, which are allowed to deflect and develop an active soil wedge, use the calculated equivalent active fluid pressure (γK_a) for the appropriate soil type. The pressure distribution may be reduced to a resultant force of $½(\gamma K_a)H^2$ per foot of wall, where H is the wall height. This force acts at one-third the wall height (⅓H) above the base.

Seismic conditions are applied using the Mononobe-Okabe equations (Bowles, 1996; Whitman, 1990). A maximum horizontal seismic acceleration k_h in bedrock of 0.198g is predicted for this site with a uniform likelihood of exceedance of 10 percent in 50 years (USGS, 2008, Hynes and Franklin, 1984). Approximately, one-half of the maximum acceleration, or 0.10g, was used to estimate lateral loads during an earthquake.

Research has indicated that lateral pressures due to earthquakes are non-hydrostatic in distribution, and the resultant acts above the one-third point of the wall (Bakeer, et al, 1990). Accordingly, active soil pressures need to be divided into two components that act at different wall heights. The static force acts at the at one-third the wall height (⅓H) above the base, as discussed above. The seismic component of the resultant force, which is $½[\gamma\ (K_{ae}-K_a)]\ H^2$ per foot of wall, is applied at 60% of the wall height (0.6H) above the base.

6.3.2 Passive Pressures

Passive earth pressures were calculated using the Coulomb and Mononobe-Okabe equations (Bowles, 1996). Values from Table 6-3 should be applied as described for active pressures above. Passive pressure design should neglect loose fill and soil located within the frost zone.

6.3.3 At-Rest Pressures

For lateral pressure design of basement walls, which are restrained and not allowed to deflect, use the calculated at rest earth pressure (γK_o). Design control of such walls should utilize whichever generates the higher resultant force: at-rest pressures (γK_o) or active seismic pressures (γK_{ae}).

6.4 Soil Friction

It is our understanding that all concrete slabs and footings will be in contact with clean crushed stone, per the manufacturer. Terzaghi et al, (1996) suggest use of the internal strength of the soil for the friction angle along a concrete base in granular soils, with a maximum value of 30 degrees. Accordingly, a friction value of 0.58, which is the tangent of 30 degrees, is suggested. The friction value may be combined with the passive pressure to resist horizontal loads.

6.5 Excavation and Cut Slope Stability

OSHA regulations (29CFR1926) appear to classify the soil anticipated in the foundation excavations as Type A soil, unless the it is observed to be fissured. Fissured loess or colluvial soils are classified as Type B. Simple cut slopes in Type A soils should be no steeper than 0.75H:1V. Slopes for Type B soils should be no steeper than 1H:1V. According to OSHA regulations, any cut slope greater than 20 feet in height would require additional analysis. The Contractor shall ultimately be responsible for adherence to OSHA and other safety regulations.

Construction shoring should be staged to minimize loading the top of the slope while unloading the toe. An example of a good progression is as follows:

1. Perform foundation excavation for upper level units (i.e., crest of slope)
2. Perform foundation excavation for lower level units (i.e., toe of slope)
3. Construct fills, foundation system, and exterior backfills for lower level units
4. Construct fills, foundation system, and exterior backfills for upper level units

This office is available to help plan the construction to minimize risk associated with construction on and near a slope. Depending on the final construction plans, excavation shoring may be required. This office is prepared to provide design of shoring if requested.

7.0 RECOMMENDATIONS

7.1 General Foundation Recommendations

All footings should be placed below the frost line, including exterior footings for awnings and porches. The building code for Teton County requires that footings be placed at a minimum depth of 34 inches from finished grade, with a minimum foundation exposure of 6 inches above finished grade.

Minor cracks in the foundation walls, floor slabs, and sheetrock are normal and should not be a cause for concern. A structural engineer should review the plans to check that adequate lateral restraint is provided to foundation walls by the floor joists.

Local codes regarding foundation ventilation and radon mitigation should be followed. The contractor shall be ultimately responsible for following local building regulations and codes.

7.2 Site Preparation

Prior to placement of structural fill (e.g., re-compacted loess or imported stony material), the site should be cleared and stripped of topsoil and organic debris. No brush, roots, frozen material, or other deleterious or unsuitable materials shall be incorporated in the foundation subgrade or structural fill. All exposed subgrade surfaces should be free of mounds and depressions which could prevent uniform compaction. If unexpected fills or obstructions are encountered during site clearing or excavation, such features should be removed and the excavation thoroughly cleaned prior to backfill placement and/or construction. Fill, footings, or slabs should not be placed on frozen subgrade.

Excavation for the foundation footings may disturb and loosen the surface of the native subgrade. All disturbed areas should be compacted with a vibratory compactor, in vibratory mode with a minimum of three passes, prior to placement of structural fill and footing construction. The actual number of passes should be determined by observing whether the surface is yielding after each pass. If the surface appears to be yielding, the number of passes should be increased until a non-yielding condition is observed and approved by the Geotechnical Engineer.

All excavations should be inspected by a representative of this office prior to fill or concrete placement, especially if questionable materials are exposed. The presence of known sand lenses and collapsible alluvial fan deposits increase the need for construction inspection. The site has complex geological relationships that will require site-specific inspection at each structure.

7.3 Foundation Drains

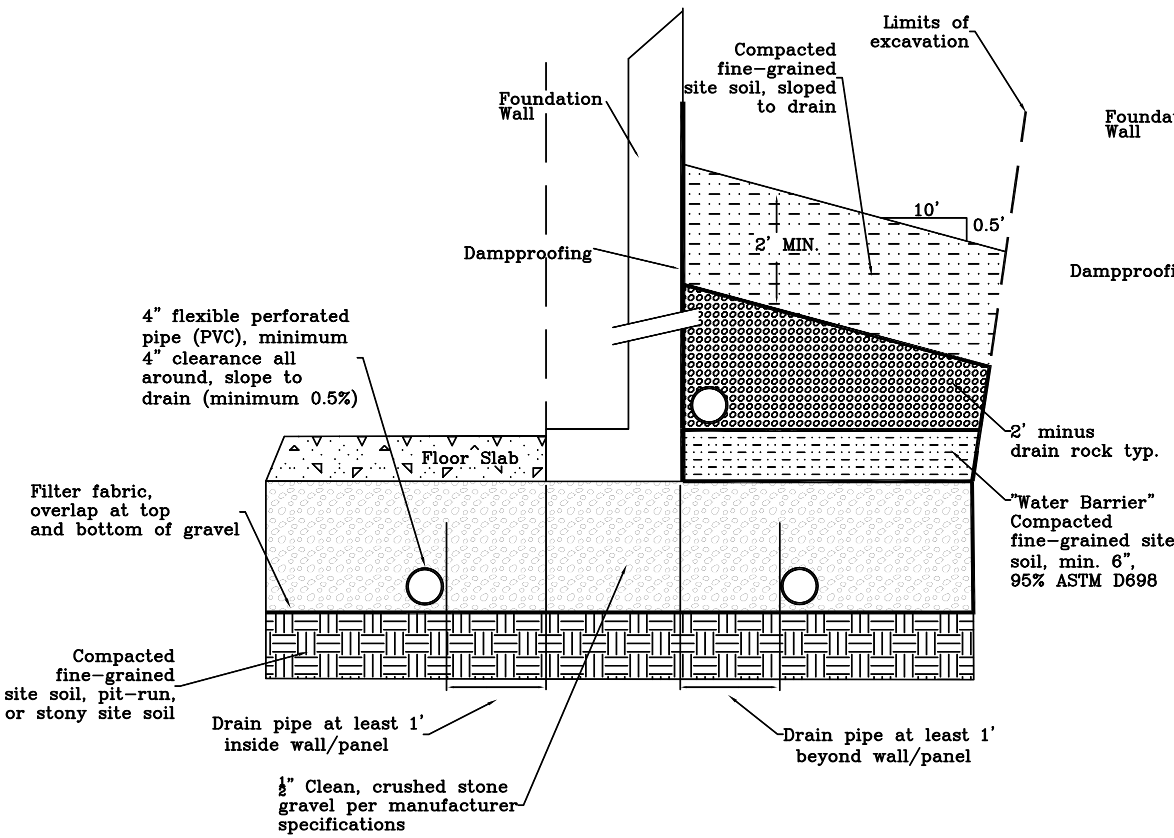
In addition to the drainage system recommended by the Superior Wall® manufacturer (shown on Figure 11), we also recommend a sub-slab drainage system (see Section 7.4) and foundation drains against frost walls or basement walls. Proper drainage is extremely important across the site because loess drains poorly and tends to collect moisture.

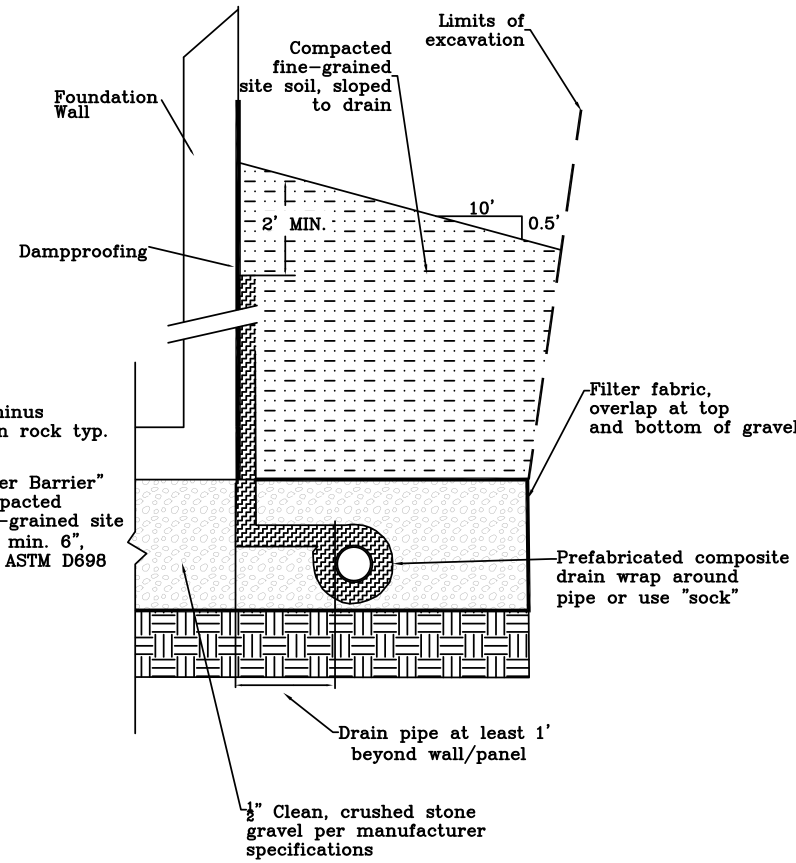
Two drainage alternatives for frost walls or basement walls are illustrated in Figure 12. Water will be kept separate from the sub-slab drainage system recommended by the Superior Walls® manufacturer with the use of a compacted fine-grained water barrier. The two options are described as follows:

1. One alternative is a prefabricated composite drain, which consists of an open wick layer laminated to filter fabric to reduce infiltration of soil. The exterior of the wall is damp-proofed and the drain is laid against the damp-proofing layer. The excavation is backfilled with compacted site material and the drain is covered by at least 2 feet of compacted site soil that is sloped to drain (minimum 5% for 10 feet). The composite drain is wrapped around a perforated drain pipe at footing level. The drain pipe may slope at a minimum of 0.5% and drain to daylight on the slope.
2. A second alternative involves placement of clean angular drain gravel or crushed stone between the foundation wall and the edge of the excavation. Drainage tiles, perforated pipe, or other approved systems should be installed at or below the area to be protected and should discharge by gravity or mechanical means into an approved drainage system. The drain pipe may slope at a minimum of 0.5% and drain to daylight or a sump. Gravel drains should extend at least 1 foot beyond the outside edge of the footing and 6 inches above the top of the footing. The gravel backfill is wrapped in an approved filter fabric. At least 2 feet of compacted fine-grained backfill (sloped to drain) is placed above the gravel envelope. The advantage of this technique is that the gravel backfill can usually be placed without compaction, reducing backfill cost and difficulty.

It is important to place the foundation drains low enough to adequately collect and discharge any water that may accumulate in utility trenches below the footings or in the gravel capillary break beneath concrete floor slabs. Drains that are placed too shallow or with insufficient gradient may fail to perform. It is also important to grade the surface of any compacted loess to a minimum of 0.5% toward the pipes of the drainage system.

It cannot be stressed enough that management of water at this site is extremely important. This office should review final plans to assure that everything drains properly.





SCHEMATIC DRAWING NOT TO SCALE

DRAFTED BY:	HC	SHEET TITLE:	PROJECT TITLE:	
REVIEWED BY:	CHL	FIGURE 12 FOUNDATION DRAINAGE DETAILS	WEST VIEW TOWNHOMES 1255 W HIGHWAY 22 JACKSON, WY	JORGENSEN Geotechnical, LLC 307-733-5150 www.jorgeng.com
PROJECT NUMBER 09040.02				

7.4 Interior Slabs-on-Grade

Interior slabs should be at least 4 inches thick, and any slabs bearing vehicles should be at least 6 inches thick, or as approved by the Structural Engineer. Minor floor cracking of slab-on-grade construction is difficult if not impossible to prevent. Such cracking is normal and should be expected to occur with time. Buildings are almost never free of cracks, and cracking is caused by many factors other than soil movement, such as concrete shrinkage, or daily and seasonal variability in temperature and humidity.

Fine-grained material (loess) should be removed below slabs-on-grade to a depth of at least 2 feet and replaced with native soil compacted to a dry density of 96% ASTM D698 covered by a minimum of 4 inches of ½ inch minus angular aggregate. A sub-slab drainage system comprising drain pipe within the aggregate layer is recommended to prevent wetting of the underlying native loess. The gravel and the compacted subgrade should be separated by a non-woven geotextile fabric.

An impermeable layer (usually plastic) is recommended beneath the slab, underlain by 4 inches of clean drain gravel that will act as a capillary break to reduce dampness. Two options are available to reduce the tendency for the concrete to crack or curls it dries. Three articles from the American Concrete Institute (ACI) that discuss these options are Appendix G. We are able to offer additional guidance if requested.

1. A blotter layer may be placed under the slab. In the past, loose sand has been used for this purpose, but is no longer recommended. A cover of 4 inches of trimmable, compactible, granular material may be placed over the sheeting to receive the concrete slab. This material usually consists of “crusher run material”, which varies in size from about 1.5-inch down to rock dust. Alternatively, 3 inches of fine graded material such as crusher fines or manufactured sand may be used.
2. The blotter layer may be eliminated if the concrete is reinforced properly. The attached article entitled “Controlling Curling and Cracking in Floors to Receive Coverings” provides a discussion of proper floor slab reinforcement. If the contractor needs additional guidance on reinforcement, a Structural Engineer should provide it.

7.5 Exterior Slabs-on-Grade

Exterior slabs (sidewalks, patios, driveways, etc.) typically sustain the greatest damage. Cracking is almost impossible to avoid, and freeze-thaw adds to the difficulty caused by soil movement. The silty loess soils may cause particularly severe frost damage. The following suggestions may reduce differential movement of exterior slabs.

Exterior slabs should be at least 4 inches thick, 6 inches if supporting vehicles, or as directed by the Structural Engineer. Exterior slabs should not be tied to foundation walls. Any movement of exterior slabs may be transmitted to the foundation walls, resulting in damage. Posts for patios or other exterior columns should not bear on exterior slabs. If the slabs settle or rise, the movement can be transmitted to the post, resulting in damage to the structure.

Fine-grained material should be removed below garage slabs and other exterior slabs to a depth of 2 feet and replaced with native soil compacted to a dry density of 96% ASTM D698 and at least 12 inches of road mix gravel. The gravel and the compacted subgrade should be separated by a non-woven geotextile fabric. Expansion joints are recommended in all concrete flatwork.

Landscaping elements placed on collapsible loess will be vulnerable to differential settlement. **"Hardscapes" that cannot tolerate movement are not recommended.** Any sensitive exterior elements should be supported treated using the same care as interior elements. Loess is likely to perform poorly if the moisture content of the subgrade increases.

If a large water feature (such as a pool, fountain, hot tub, etc.) is constructed in the loess, it should also be supported on helical piers to provide the water feature's foundation support. Plumbing attached to any water features should be attached to the supported structure (e.g., the structural pool floor) to reduce the chance for breakage, in the event that soil collapse occurs. **Landscapers and water feature designers should be provided the geotechnical report and formally briefed about the necessity to manage water and grades at the site.** Notes should be taken of meetings and instructions conveyed to all designers.

7.6 Ventilation and Treatment

Evaluation of radon was beyond the scope of this work; local codes should be followed and specialty contractors employed, if necessary. Ventilation to reduce moisture and potential accumulation of radon gas is required by code for inhabited spaces below grade. A capillary break layer may be necessary to accommodate a radon vent pipe. The building contractor is ultimately responsible for following local building codes.

7.7 Reinforcing, Utilities Testing, and Concrete Considerations

Footings, slabs, and foundation walls should be reinforced to resist differential movement. Consultation with a Structural Engineer to specify adequate reinforcement is suggested. Water and sewer lines should be pressure tested before backfilling. Exterior concrete should contain 5% to 7% entrained air.

7.8 Observation during Construction

A representative of this office should observe construction of any foundation or drainage elements recommended in this report, especially deep foundation elements. Site grading, leak-proof testing, and soil compaction should be observed by a representative of this office. Recommendations in this report are contingent upon our involvement. If any unexpected soils or conditions are revealed during construction, this office should be notified immediately to survey the conditions and make necessary modifications.

8.0 LIMITATIONS

This report has been prepared based on a limited amount of data. Actual site conditions may vary. The conclusions and recommendations presented in this letter assume that site conditions are not substantially different than expected. If subsurface conditions are different, Jorgensen Geotechnical, LLC, should be advised so that we can review those conditions and reconsider our recommendations where necessary.

This report was prepared for use by the owner and their representatives. It should be made available to prospective contractors for information on factual data only and not as a warranty of subsurface conditions. Any conclusions by a contractor or bidder relating to construction means, methods, techniques, sequences or costs based upon the information provided in this report are not the responsibility of the Owner or Jorgensen Geotechnical, LLC.

These services have been performed in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing in this area under similar conditions. Construction on potentially collapsible soils is not without risk. No warranty of performance is made or implied.

9.0 REFERENCES

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APPENDIX A
Borehole Logs

Jorgensen Geotechnical
Jackson, WY 83002
Telephone: 307-733-5150
Fax: 307-733-5187

TEST HOLE LOG

PROJECT NAME: West View Townhomes, 1255 W. Hwy 22	DATE: 6/1/2016
PROJECT LOCATION: Jackson, Wyoming	HOLE NO.: JG-1
TEST HOLE LOCATION: North edge of lower parking lot, see site map	

ELEVATION G.S. (ft.): 6183.7	TOTAL DEPTH (ft.): 31	GROUNDWATER LEVEL (ft.): NA	MEASURED FROM: Surface

DRILL TYPE: BK-81	HAMMER: 140 # Automatic	DRILL CO: HazTech Drilling, Inc.	DRILLER: Chris	LOGGED BY: chl

DESCRIPTION

COMMENTS: Asphalt surface.

DEPTH (ft.)	GRAPHICAL LOG	SAMPLE	S.P.T. (N) BLOWS/6 IN.	(N1)60 BLOWS/FT.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)	WELL COMPLETION
0–2								0.0-2.0ft Sandy GRAVEL: Dry, gray, rounded to subrounded gravel, silty sand matrix [FILL] Driller: "Rock at 24-inches"					
2–5		D1	12,8,6	20	10			2.0-10.3ft LAYER I: LOESS 2.5ft Very little recovery. Sample assumed to be cuttings/slough pounded through silty loess.					
5–7.5		D2	2,1,1	3	44			5.0ft Sandy SILT: Moist, tan brown with white calcite deposition, very soft, massive with pinhole voids [LOESS]					
7.5–10		D3	2,2,4	9	55			7.5ft Sandy CLAY/SILT: Slightly moist, dark brown, soft, massive with scattered pinhole voids, scattered broken limestone gravel [LOESS]					
10–12.5		D4	13,20,50/3.5"	50+	83			10.0ft Upper 3" - Clayey SILT: Slightly moist, dark brown, medium stiff, massive [LOESS] Lower 10" - Gravelly CLAY: Slightly moist, dark brown, very dense, intact, angular limestone clasts in matrix of clayey fines, stone in shoe [COLLUVIUM]					
12.5–14.5		D5	13,17,14	39	72			10.3-14.5ft LAYER II: COLLUVIUM 12.5ft Clayey sandy GRAVEL: Moist, brown, dense, 50-60% broken/subangular limestone gravel, silty sand matrix [COLLUVIUM]					
14.5–17.5		D6	9,50/4"	50+	90			14.5-22.0ft LAYER III: OLDER LOESS/COLLUVIUM 15.0ft Lean CLAY with gravel: Slightly moist, brown with white calcite deposition, medium stiff to stiff, limestone clast in sampler shoe [OLDER LOESS/COLLUVIUM] 16.0ft Driller: "Heavy grinding 16-17', soft at 17-ft"					
17.5–20		D7	2,4,10	18	100	1.5	CL	17.5ft Sandy lean CLAY: Moist, light tan mottled white, medium stiff, pinhole voids, massive, 65% clayey fines, 32% subangular to subrounded sand, 3% gravel [OLDER LOESS/COLLUVIUM]	24.5		33	11	

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Jackson, WY 83002
Telephone: 307-733-5150
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TEST HOLE LOG

PROJECT NAME: West View Townhomes, 1255 W. Hwy 22 | DATE: 6/1/2016

PROJECT LOCATION: Jackson, Wyoming | HOLE NO.: JG-1

COMMENTS: Asphalt surface.

DEPTH (ft.)	SAMPLE	S.P.T. (N) BLOWS/6 IN.	(N1)60 BLOWS/FT.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)
20–22	D8	11,12,13	34	77	2.25	CL	20.0ft Sandy lean CLAY with gravel: Moist, gray brown with white calcite deposits, stiff, massive, clasts angular to subangular, 61% clayey fines [OLDER LOESS/COLLUVIUM]	13.2		26	9
22–24.5	D9	9,10,6	15	55			22.0-24.5ft LAYER IV: OLDER COLLUVIUM 22.5ft GRAVEL with sand and silt: Moist, brown, matrix sandy silt, medium dense, broken clasts of black Andesite and pink Sandstone [OLDER COLLUVIUM]				
24.5–27	D10	4,4,7	14	100	2.0		24.5-31.0ft LAYER V: ALLUVIUM 25.0ft Lean CLAY with sand: Moist, brown, medium stiff, massive, 86% lean clay, ~ 15% sand with scattered small gravel [ALLUVIUM]				
27–30	D11	3,6,8	17	100	1.0	CL	27.5ft Lean CLAY with sand: Moist, brown, medium stiff, massive, lean clay with small gravel [ALLUVIUM]	21.6		37	20
30–31	U1			100			30.0ft Sample as above, thin-walled sampler inserted with 250 psi pressure.				

Note: Caved to 17' below ground surface.
Backfill with bentonite from 17' to 1' below ground surface. Cuttings to surface.
No groundwater observed at time of drilling.

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TEST HOLE LOG

PROJECT NAME: West View Townhomes, 1255 W. Hwy 22	DATE: 6/1/2016
PROJECT LOCATION: Jackson, Wyoming	HOLE NO.: JG-2
TEST HOLE LOCATION: Southwest corner of site, see site map	
ELEVATION G.S. (ft.): 6182	TOTAL DEPTH (ft.): 33.5
GROUNDWATER LEVEL (ft.): 30.48	MEASURED FROM: Surface
DRILL TYPE: BK-81	HAMMER: 140 # Automatic
DRILL CO: HazTech Drilling, Inc.	DRILLER: Chris
LOGGED BY: chl	

COMMENTS: Surface sparse grass and earth. Standard split spoon sample with plastic catcher. No liners.

DEPTH (ft.)	SAMPLE	S.P.T. (N) BLOWS/6 IN.	(N1)60 BLOWS/FT.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)
0.0							0.0-13.3ft LAYER I: LOESS				
2.5	D1	1,1,2	5	50			2.5ft Sandy SILT: Moist, dark brown, soft, massive, small roots at sample bottom [LOESS]				
5.0	D2	2,2,2	6	38			5.0ft Sandy SILT: As above, soft [LOESS]				
7.5	D3	2,2,2	6	72		ML	7.5ft SILT: Moist, tan, massive, soft, strong HCl reaction, 96% silty fines, 4% fine sand [LOESS]	13.5		NP	NP
10.0	D4	2,2,3	7	72		ML	10.0ft SILT: Moist, tan, massive, soft, 93% silty fines with 6% sand and scattered pea sized subangular gravel [LOESS]	12.7		23	3
12.5	U2			100			12.5ft SILT with gravel: Moist, tan, massive, with large gravel clasts, thin-walled tube bent at bottom [LOESS]				
13.3							13.3-23.2ft LAYER II: COLLUVIUM				
13.5	D5	5,7,13	27	100			13.5ft SILT with gravel: Slightly moist, brown, medium dense, black gravel clasts, mechanical breakage, stone in the sampler shoe [COLLUVIUM]				
15.0	D6	8,11,11	29	77			15.0ft Sandy sitly GRAVEL: Moist, brown, medium dense, mechanical breakage of clasts, 60% gravel with silty sand matrix [COLLUVIUM]				
17.5	D7	7,6,6	15	66			17.5ft Gravelly silty SAND: Moist, brown, loose, intact, 40% fine to coarse sand, 30% subangular gravel to ~1" diameter, 30% silty fines [COLLUVIUM]				

GRAPHICAL LOG | WELL COMPLETION

Jorgensen Geotechnical
Jackson, WY 83002
Telephone: 307-733-5150
Fax: 307-733-5187

TEST HOLE LOG

PROJECT NAME: West View Townhomes, 1255 W. Hwy 22 | DATE: 6/1/2016

PROJECT LOCATION: Jackson, Wyoming | HOLE NO.: JG-2

COMMENTS: Surface sparse grass and earth. Standard split spoon sample with plastic catcher. No liners.

DEPTH (ft.)	SAMPLE	S.P.T. (N) BLOWS/6 IN.	(N1)60 BLOWS/FT.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)
20.0	D8	7,11,5	13	77			20.0ft As above, with andesite and limestone clasts [COLLUVIUM]				
22.5	D9	4,2,5	9	100	.5		22.5ft Upper 8" - as above [COLLUVIUM] Lower 10" - Lean CLAY: mst, reddish brown, soft, massive [ALLUVIUM]				
23.2							23.2-33.5ft LAYER III: ALLUVIUM				
25.0	U2			100			25.0ft CLAY: Moist, reddish brown, soft, massive, strong HCl reaction [ALLUVIUM]				
27.5	D10	1,2,2	5	100	.5	CL	27.5ft CLAY: As above, wet, 93% clayey fines, 7% fine sand [ALLUVIUM]	32.9		34	13
30.0	D11	3,1,3	5	100	<.5	CL	30.0ft Sandy CLAY: wet, reddish brown, soft, 55.9% clayey fines, with 44% sand [ALLUVIUM]	27.9		26	11
32.5	U3						32.5ft Sandy CLAY: as above, wet, soft, sampler inserted with 100 psi pressure				

Note: Groundwater observed at 30.48' at time of investigation.
Hole caved to 24' below ground surface.
Backfill with bentonite .5' to 24'.
Cuttings .5' to surface.

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TEST HOLE LOG

PROJECT NAME: West View Townhomes, 1255 W. Hwy 22	DATE: 6/1/2016
PROJECT LOCATION: Jackson, Wyoming	HOLE NO.: JG-3
TEST HOLE LOCATION: Southeast ol lower lot, at top of slope	

ELEVATION G.S. (ft.): 6183.8	TOTAL DEPTH (ft.): 46.5	GROUNDWATER LEVEL (ft.): 31.8	MEASURED FROM: Surface

DRILL TYPE: BK-81	HAMMER: 140 # Automatic	DRILL CO: HazTech Drilling, Inc.	DRILLER: Chris	LOGGED BY: chl

COMMENTS: Asphalt surface.

DEPTH (ft.)	GRAPHICAL LOG	SAMPLE	S.P.T. (N) BLOWS/6 IN.	(N1)60 BLOWS/FT.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)	WELL COMPLETION
0–2								0.0-2.0ft Sandy GRAVEL: Dry, gray, rounded to subrounded gravel, silty sand matrix [FILL]					
2								2.0-14.5ft LAYER I: COLLUVIUM					
3		D1	6,8,6	20	77			2.5ft Sandy GRAVEL with silt: Moist, brown, medium dense, intact, many broken rock fragments, 60-70% gravel, 20-30% sand with silty fines [COLLUVIUM]					
6		D2	6.6,5	15	88			5.0ft Silty SAND with gravel: Moist, brown, massive, medium dense, 30-40% angular andesite limestone and sandstone gravel, 40-50% sand, 15-20% fines, strong HCl reaction in fines [COLLUVIUM]					
8		D3	5,5,5	14	44			7.5ft Silty SAND with gravel: As above, 33% coarse sand, 20% gravel, 47% fines [COLLUVIUM]					
11		D4	5,12,12	31	77			10.0ft Silty SAND with gravel: As above, dense, black andesite clasts, 30% orange subangular to angular gravel [COLLUVIUM]					
13		D5	9,8,6	16	100			12.5ft Silty SAND with gravel: As above, many broken clasts of gravel, gravel/sand [COLLUVIUM]					
14.5								14.5-26.5ft LAYER II: OLDER LOESS					
16		D6	2,3,5	10	100	.75	CL-ML	15.0ft Lean CLAY: Very moist to wet, brown, soft, massive, lean clay, mild HCl reaction, 79.6% clayey fines, ~20% fine sand [OLDER LOESS]	26.4		27	7	
17								17.0ft Driller: "Gravel at 17-ft"					
18		D7	4,11,21	40	83			17.5ft Lean CLAY with gravel: Moist, reddish brown, soft to medium stiff, with yellow sandstone, red/pink sandstone, limestone and andesite gravel, broken fragments discarded in sample [OLDER LOESS/COLLUVIUM]					

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TEST HOLE LOG

PROJECT NAME: West View Townhomes, 1255 W. Hwy 22

DATE: 6/1/2016

PROJECT LOCATION: Jackson, Wyoming

HOLE NO.: JG-3

COMMENTS: Asphalt surface.

DEPTH (ft.)	GRAPHICAL LOG	SAMPLE	S.P.T. (N) BLOWS/6 IN.	(N1)60 BLOWS/FT.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)	WELL COMPLETION
21		D8	5,6,11	23	77			20.0ft Silty SAND with gravel: Moist, brown, loose, intact, 40% fine to coarse sand, 30% subangular gravel to 1" diameter, 30% silty fines [OLDER LOESS]					
22													
23		D9	1,2,3	6	100	.75	CL-ML	22.5ft Sandy CLAY-SILT: Very moist to wet, brown , soft, massive, 84.9% fines, ~15% fine sand [OLDER LOESS]	22.6		22	6	
24													
25		U1						25.0ft Sandy CLAY-SILT: As above, very moist, medium stiff [OLDER LOESS]					
26													
27								26.5-30.5ft Sandy lean CLAY: Very moist, gray brown, soft, massive, fragment of charcoal in lower part of sample, strong HCl reaction, 87% clayey fines, ~13% fine sand [ALLUVIUM]					
28		D10	2,4,4	10	100	.5	CL		26.8		30	14	
29													
30													
31		D11	14,29,28	68	27			30.0ft Very little recovery (lost material from shoe). Sample retained: as above. Contact with gravel assumed. 30.5-46.5ft LAYER III: GLACIAL OUTWASH					
32													
33													
34													
35								35.0ft Sandy GRAVEL: Wet, tan, dense, cobbles broken by sampler (rounded quartz, black andesite, and white sandstone clasts) [GLACIAL OUTWASH]					
36		D12	19,19,11	25	20								
37								Driller: "Gravelly drilling to 40-ft"					
38													
39													
40								40.0ft Sandy GRAVEL/COBBLE: Wet, brown, very dense, assumed stratified, observed broken quartzite clasts [OUTWASH]					
41		D13	13,28,18	38	20								
42								Driller: "Gravelly drilling to 45-ft"					
43													

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TEST HOLE LOG

PROJECT NAME: West View Townhomes, 1255 W. Hwy 22	DATE: 6/1/2016
PROJECT LOCATION: Jackson, Wyoming	HOLE NO.: JG-3

COMMENTS: Asphalt surface.

DEPTH (ft.)	GRAPHICAL LOG	SAMPLE	S.P.T. (N) BLOWS/6 IN.	(N1)60 BLOWS/FT.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)	WELL COMPLETION
44													
45–46		D14	10,14,19	33	20			45.0ft Sandy GRAVEL/COBBLE: As above, dense [GLACIAL OUTWASH]					

Note:
Groundwater observed at 31.3' at time of drilling and 31.8' on 6/2/2016.
Installed vibrating wire piezometer on 6/2/16--Serial Number: 1600636 to 44' below ground surface.
Used DGSI recommended grout mix: 1 bag 94# cement, 30 gallons water, ~60# bentonite.
Finish with flush mount.

Depth scale: 44, 45, 46, 47, 48, 49, 50, 51, 52, 53, 54, 55, 56, 57, 58, 59, 60, 61, 62, 63, 64, 65, 66

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TEST HOLE LOG

PROJECT NAME: West View Townhomes, 1255 W. Hwy 22				DATE: 6/2/2016
PROJECT LOCATION: Jackson, Wyoming				HOLE NO.: JG-4
TEST HOLE LOCATION: See site map				
ELEVATION G.S. (ft.): 6184.2	TOTAL DEPTH (ft.): 36.5	GROUNDWATER LEVEL (ft.): 31.7	MEASURED FROM: Surface	
DRILL TYPE: BK-81	HAMMER: 140 # Automatic	DRILL CO: HazTech Drilling, Inc.	DRILLER: Chris	LOGGED BY: chl

COMMENTS: Asphalt parking at surface.

DEPTH (ft.)	SAMPLE	S.P.T. (N) BLOWS/6 IN.	(N1)60 BLOWS/FT.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)
0-2							0.0-2.0ft Sandy GRAVEL: Dry, gray, rounded to subrounded gravel, silty sand matrix [FILL]				
2							2.0-8.5ft LAYER I: LOESS				
3	D1	1,2,2	7	61		CL	2.5ft Lean CLAY: Moist, brown with white deposition, soft, massive, 69.8% silt-size and 24.6% clay-size particles with 5.6% sand [LOESS]	28.4		37	17
6	D2	2,2,3	8	77		CL	5.0ft Lean CLAY: As above, soft, massive with pinhole voids, 65.5% silt-size and 31.1% clay-size particles with 3% fine sand [LOESS]	29.0		35	14
8	U1			100	3.5		7.5ft As above, encountered gravel in sample at 8.5'	22.3	68.5		
8.5							8.5-11.5ft LAYER II: COLLUVIUM				
11	D3	23,23,21	55	88			10.0ft Sandy GRAVEL: Moist, brown, very dense, intact, andesite and limestone gravel, 30% silty sand matrix, stone in shoe, mechanical breakage [COLLUVIUM]				
							Driller: "Soft at about 11-ft"				
11.5							11.5-14.5ft LAYER III: OLDER LOESS				
13	D4	5,7,7	20	77			12.5ft Sandy SILT: Moist, reddish brown, medium stiff, massive, tiny pinhole voids and calcite streaking [LOESS]				
							Driller: "Gravel at 14.5-ft"				
14.5							14.5-22.0ft LAYER III: COLLUVIUM				
16	D5	50/2"	50+	0			15.0ft Sampler refusal on cobble at 15', no sample to identify.				
18	D6	12,17,27	55	50			17.5ft GRAVEL: Moist, brown, very dense, intact, 60-70% gravel cobble with silty sand matrix [COLLUVIUM]				

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TEST HOLE LOG

PROJECT NAME: West View Townhomes, 1255 W. Hwy 22 | DATE: 6/2/2016

PROJECT LOCATION: Jackson, Wyoming | HOLE NO.: JG-4

COMMENTS: Asphalt parking at surface.

DEPTH (ft.)	SAMPLE	S.P.T. (N) BLOWS/6 IN.	(N1)60 BLOWS/FT.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)
20.0–21.5	D7	22,32,17	45	94			20.0ft Sandy GRAVEL: Moist, brown, very dense, intact, 50-60% angular to subangular gravel, 30-40% sand, 10-15% silty fines [COLLUVIUM]				
22.0							22.0-27.0ft LAYER IV: OLDER LOESS				
22.5–24.0	D8	5,6,6	15	88		CL-ML	22.5ft CLAY and SILT: Moist, reddish brown, medium stiff, massive, pinhole voids, 87% clayey/silty fines with 13% coarse to fine sand [OLDER LOESS]	20.9		24	7
25.0–26.5	D9	5,7,8	19	88		CL-ML	25.0ft CLAY and SILT: Moist, gray brown, massive, medium stiff, pinhole voids, calcite stringers, scattered stones (andesite, Bacon Ridge, limestone), 81% fines and 19% fine to medium sand [OLDER LOESS]	15.7		24	7
27.0							27.0-36.5ft LAYER V: GLACIAL OUTWASH				
27.5–29.0	D10	17,36,45	96	50			27.5ft Sandy GRAVEL/COBBLE: moist, very dense, subrounded to subangular quartzite clasts [GLACIAL OUTWASH]				
30.0–31.5	D11	16,34,37	85	50			30.0ft Sand GRAVEL/COBBLE: Moist, very dense, subrounded to rounded quartzite stones [GLACIAL OUTWASH]				
35.0–36.5	D12	6,12,25	41				35.0ft Sandy GRAVEL/COBBLE: Wet, dense, subrounded to rounded quartzite stones [GLACIAL OUTWASH]				

Note:
Groundwater observed at 31.7' at time of digging.
Hole caved to 25' below ground surface.
Backfill with bentonite chips to 1' below ground surface (12 bags), cuttings to surface.

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TEST HOLE LOG

PROJECT NAME: West View Townhomes, 1255 W. Hwy 22	DATE: 6/2/2016
PROJECT LOCATION: Jackson, Wyoming	HOLE NO.: JG-5
TEST HOLE LOCATION: See site map	
ELEVATION G.S. (ft.): 6220.1	TOTAL DEPTH (ft.): 71.5
GROUNDWATER LEVEL (ft.): 67.5	MEASURED FROM: Surface
DRILL TYPE: BK-81	HAMMER: 140 # Automatic
DRILL CO: HazTech Drilling, Inc.	DRILLER: Chris
LOGGED BY: chl	

COMMENTS: Gravel surface within fenced parking area.

DEPTH (ft.)	GRAPHICAL LOG	SAMPLE	S.P.T. (N) BLOWS/6 IN.	(N1)60 BLOWS/FT.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)	WELL COMPLETION
0.0								0.0-2.0ft Sandy GRAVEL: Dry, gray, rounded to subrounded gravel, silty sand matrix [FILL]					
2.0								2.0-20.6ft LAYER I: LOESS					
5.0		D1	2,3,3	9	90		CL-ML	5.0ft Sandy SILT: Moist, brown, massive, soft, with fine sand, andesite porphyry stone in sampler shoe, 87.3% silt/clay fines with 13% sand [LOESS]	25.5		25	5	
10.0		U1			55			10.0ft Sandy SILT: As above, 4.5" sample lost out bottom [LOESS]					
15.0		D2	2,2,3	7	55			15.0ft Sandy SILT: Moist, gray brown, soft, massive, scattered andesite pebbles, with fine sand [LOESS]					

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TEST HOLE LOG

PROJECT NAME: West View Townhomes, 1255 W. Hwy 22	DATE: 6/2/2016
PROJECT LOCATION: Jackson, Wyoming	HOLE NO.: JG-5

DESCRIPTION

COMMENTS: Gravel surface within fenced parking area.

DEPTH (ft.)	GRAPHICAL LOG	SAMPLE	S.P.T. (N) BLOWS/6 IN.	(N1)60 BLOWS/FT.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)	WELL COMPLETION
								20.0ft Upper 7" - Sandy SILT, as above [LOESS]					
21		D3	3,7,6	16	72			20.6-21.0ft Middle 4" - scattered limestone/andesite/yellow sandstone gravel with silty matrix [DEBRIS FLOW/ALLUVIAL FAN]					
22								21.0-27.0ft Lower 2" - brown sandy SILT, as above [LOESS]					
23		D4	2,3,3	8	72			22.5ft Sandy SILT: Moist, brown, soft, massive [LOESS]					
24													
25, 26		D5	3,3,6	11	100			25.0ft Sandy SILT: As above, slight calcite coloring, scattered pea sized angular gravel, medium stiff [LOESS]					
27								Driller: "Gravelly drilling at 27-ft"					
28		D6	7,14,13	32	89			27.0-30.4ft LAYER II: COLLUVIUM 27.5ft Silty sandy GRAVEL: Moist, brown, dense, broken andesite and yellow sandstone, silty sand matrix [COLLUVIUM]					
29													
30, 31		D7	4,4,4	10	89		CL	30.0ft Upper 4" - Silty sandy GRAVEL: as above, reddish brown [COLLUVIUM]	25.2		42	22	
32								30.4-32.9ft LAYER III: ALLUVIUM Lower 12" - Lean CLAY: Moist, light tan, calcite deposition, soft, 75% clayey fines, 23% fine to coarse sand, with 2% scattered fine gravel less than 3/8" [ALLUVIUM]					
33		D8	11,27,29	67	89		CL	32.5ft Upper 5" - Lean CLAY: Moist, light tan, calcite deposition, scattered fine gravel, lean clay, 79.5% fines [ALLUVIUM]	30.0		43	20	
34								32.9-71.5ft LAYER III: GLACIAL OUTWASH Lower 11" - Sandy GRAVEL/COBBLE: very dense [GLACIAL OUTWASH]					
35, 36		D9	50/5"	50+				35.0ft Sandy GRAVEL/COBBLE: Moist, light brown, dense, rounded quartzite clasts in shoe [GLACIAL OUTWASH]					
37, 38, 39													
40, 41		D10	17,26,35	66	78			40.0ft Sandy GRAVEL/COBBLE: Rounded quartzite clasts [GLACIAL OUTWASH]					
42, 43													

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TEST HOLE LOG

PROJECT NAME: West View Townhomes, 1255 W. Hwy 22 | DATE: 6/2/2016

PROJECT LOCATION: Jackson, Wyoming | HOLE NO.: JG-5

COMMENTS: Gravel surface within fenced parking area.

DEPTH (ft.)	GRAPHICAL LOG	SAMPLE	S.P.T. (N) BLOWS/6 IN.	(N1)60 BLOWS/FT.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)	WELL COMPLETION
45		D11	50/1.5"	50+	10			45.0ft Very little recovery, sand in sample bag, likely sampler met refusal on cobble					
50		D12	40,40,43	79	78			50.0ft As above [GLACIAL OUTWASH]					
55		D13	11,31,41	64	89			55.0ft As above [GLACIAL OUTWASH]					
60		D14	12,18,29	40	78			60.0ft As above [GLACIAL OUTWASH]					
65		D15	16,40,44	68	78			65.0ft As above [GLACIAL OUTWASH]					

Depth scale: 44, 45, 46, 47, 48, 49, 50, 51, 52, 53, 54, 55, 56, 57, 58, 59, 60, 61, 62, 63, 64, 65, 66

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TEST HOLE LOG

PROJECT NAME: West View Townhomes, 1255 W. Hwy 22 | DATE: 6/2/2016

PROJECT LOCATION: Jackson, Wyoming | HOLE NO.: JG-5

DEPTH (ft.)	GRAPHICAL LOG	SAMPLE	S.P.T. (N) BLOWS/6 IN.	(N1)60 BLOWS/FT.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION COMMENTS: Gravel surface within fenced parking area.	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)	WELL COMPLETION
67													
68													
69													
70		D16	5,13,13	20	50			70.0ft As above, wet (below water table) [GLACIAL OUTWASH]					
71													

Note: Hole completion on 6/3/2016.
Groundwater observed at 67.5' below ground surface on 6/3/2016.
Installed vibrating wire piezometer to 69' bgs. Serial number 1600635.
Installed vibrating wire piezometer to 32' bgs. Serial number 1600515.
Grout mix: 30 gallons water, 1 bag (94#) cement, Bentonite gel (+/- 25#)
Finish upper 5' with concrete, surface mount vault.

72 73 74 75 76 77 78 79 80 81 82 83 84 85 86 87 88 89

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TEST HOLE LOG

PROJECT NAME: West View Townhomes, 1255 W. Hwy 22	DATE: 6/3/2016
PROJECT LOCATION: Jackson, Wyoming	HOLE NO.: JG-6
TEST HOLE LOCATION: North on upper bench, see site map	

ELEVATION G.S. (ft.): 6218.6	TOTAL DEPTH (ft.): 51.5	GROUNDWATER LEVEL (ft.): NA	MEASURED FROM: Surface

DRILL TYPE: BK-81	HAMMER: 140 # Automatic	DRILL CO: HazTech Drilling, Inc.	DRILLER: Chris	LOGGED BY: chl

DEPTH (ft.)	SAMPLE	S.P.T. (N) BLOWS/6 IN.	(N1)60 BLOWS/FT.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION / COMMENTS:	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)
0.0							0.0-2.0ft Sandy GRAVEL: Dry, gray, rounded to subrounded gravel, silty sand matrix [FILL]				
2.0							2.0-22.0ft LAYER I: LOESS				
5.0	D1	3,3,2	6	25			5.0ft Lean CLAY: Lost most of sample, remainder appears to be moist, tan, soft, massive silty lean clay as in JG-5 [LOESS]				
8.0	U1			100			8.0ft Lean CLAY: Moist, tan, soft, massive [LOESS]	13.7	77.8		
10.0	U2			100			10.0ft As above [LOESS]	14.4	73.1		
13.0							13.0ft Driller: "Gravel at 13-ft" - Possible small lens of gravel colluvium.				
15.0	D2	2,3,3	8	66		CL-ML	15.0ft SILT-CLAY with sand: Moist, brown, soft, massive, 58.8% silt-size and 20.9% clay-size particles with 18.3% sand and 2% scattered, fine andesite gravel [LOESS]	13.8		26	6

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TEST HOLE LOG

PROJECT NAME: West View Townhomes, 1255 W. Hwy 22 | DATE: 6/3/2016

PROJECT LOCATION: Jackson, Wyoming | HOLE NO.: JG-6

DEPTH (ft.)	GRAPHICAL LOG	SAMPLE	S.P.T. (N) BLOWS/6 IN.	(N1)60 BLOWS/FT.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION COMMENTS:	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)	WELL COMPLETION
21		D3	4,6,7	18	55			20.0ft Gravelly CLAY: Moist, brown, medium stiff, intact, silty clay with 30-40% andesite gravel, broken clast in shoe [LOESS/COLLUVIUM]					
22													
23		D4	8,10,8	21	72			22.0-24.5ft LAYER II: COLLUVIUM 22.5ft Silty GRAVEL: Moist, brown, intact, medium dense, 60-70% gravel, mechanical breakage of clasts [COLLUVIUM]					
24													
25–26		D5	5,10,9	23	72			24.5-25.6ft LAYER III: LOESS 25.0ft Upper 7" - Sandy SILT: Moist, brown, soft, massive [LOESS] Lower 6" - Silty GRAVEL: Moist, brown, medium dense, intact [COLLUVIUM]					
27								25.6-27.0ft LAYER IV: OLDER COLLUVIUM					
28		D6	7,9,14	28				27.0-37.0ft LAYER V: OLDER LOESS/COLLUVIUM 27.5ft Sandy SILT with gravel: Moist, reddish brown, stiff/medium dense, massive, about 30% gravel (andesite and yellow sandstone) [OLDER LOESS/COLLUVIUM]					
29													
30–31		D7	8,16,33	60	83			30.0ft Upper 5" - Sandy SILT with gravel: Moist, reddish brown, stiff/medium dense, massive [OLDER LOESS/COLLUVIUM] Lower 10" - Silty GRAVEL: Moist, brown, dense to very dense, 60-70% gravel derived from Bacon Ridge/yellow sandstone/black andesite [COLLUVIUM]					
32													
33		D8	8,11,11	26	94			32.5ft Silty sandy CLAY with gravel: Moist, reddish brown with white calcite deposition, stiff, massive, 30% pink sandstone and black andesite gravel [OLDER LOESS/COLLUVIUM]					
34													
35–36		D9	8,8,12	23	88		SC	35.0ft Clayey SAND with gravel: As above, 48% clayey fines, 31% sand, 21% fine gravel (< 3/4") [OLDER LOESS/COLLUVIUM]	16.0		32	13	
37													
38		D10	3,3,4	8	100		CL	37.0-39.5ft LAYER VI: ALLUVIUM 37.5ft Lean CLAY with sand: Moist, brown with white calcite stringers, soft, massive with pinhole voids, 72.8% clayey fines with about 25% sand and trace fine gravel [ALLUVIUM]	19.3		27	11	
39		U3			50			39.0ft Lean CLAY: As above, soft, sampler met refusal at 5" on rounded gravel/cobble [ALLUVIUM]					
40–41		D11	22,33,50/1"	50+	30			39.5-51.5ft LAYER VII: GLACIAL OUTWASH 40.0ft Sandy silty GRAVEL/COBBLE: Slightly moist, very dense, rounded quartzite stones [GLACIAL OUTWASH]					
42–43													

Jorgensen Geotechnical
Jackson, WY 83002
Telephone: 307-733-5150
Fax: 307-733-5187

TEST HOLE LOG

PROJECT NAME: West View Townhomes, 1255 W. Hwy 22	DATE: 6/3/2016
PROJECT LOCATION: Jackson, Wyoming	HOLE NO.: JG-6

DEPTH (ft.)	GRAPHICAL LOG	SAMPLE	S.P.T. (N) BLOWS/6 IN.	(N1)60 BLOWS/FT.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION COMMENTS:	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)	WELL COMPLETION
45		D12	12,16,25	41	77			45.0ft Sandy GRAVEL/COBBLE: Moist, dense, trace fines [GLACIAL OUTWASH]					
50		D13	16,31,50/4.5"	50+				50.0ft Sandy GRAVEL/COBBLE: As above, very dense [GLACIAL OUTWASH]					

Note: No groundwater observed at time of drilling. Backfilled hole with bentonite chips to 1-ft bgs. Finish with cuttings to surface.

WOMACK & ASSOCIATES, INC.
Geotechnical Engineering
Geology

4125 S. Hwy 89, Suite 3B
Jackson, WY 83001
Telephone: 307 733-7209
Fax:

TEST HOLE LOG

PROJECT NAME: Town of Jackson, East Pathways Project	DATE: 10/7/11
PROJECT LOCATION: Jackson, Wyoming	HOLE NO.: BH-1
TEST HOLE LOCATION: In front of Thrifty Car Rental, ~10' northeast of sidewalk	
ELEVATION G.S. (ft.):	TOTAL DEPTH (ft.): 21.5
GROUNDWATER LEVEL (ft.): Dry	MEASURED FROM: Surface
DRILL TYPE: CME 850	HAMMER:
DRILL CO: HazTech Drilling, Inc.	DRILLER: Dave/Corbin
LOGGED BY: br	

DEPTH (ft.)	SAMPLE	S.P.T. (N) BLOWS/6 IN.	(N1)60 BLOWS/FT.	RECOVERY (%)	UNCONFINED STRENGTH (TSF)	CLASSIFICATION	DESCRIPTION / COMMENTS:	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	LIQUID LIMITS (%)	PLASTICITY INDEX (%)	WELL COMPLETION
0.0-0.5							0.0-0.5ft Surface road fill					
0.5-6.0	1	9,13,6	22	78			0.5-6.0ft Sandy GRAVEL: Tan to brown, gravel to 3/4" diameter [ALLUVIAL FAN]					
6.0-10.0							6.0-10.0ft Clayey SILT: Moist, brown, no bedding [LOESS]					
10.0-15.0	2	4,6,5	18	83			10.0-15.0ft Clayey SILT: Moist, brown, very stiff, massive [LOESS] Bottom 6" of sample: CLAY with gravel to 1/4" diameter, moist, loose					
15.0-16.5	3	1,3,6	14	100			15.0-16.5ft Clayey SILT: Very moist, brown, massive, medium stiff [LOESS]					
16.5-21.5	3	7,9,20	38	89			16.5-21.5ft CLAY with gravel: Very stiff [COLLUVIUM] Note: Installed monitoring well. 0-14' 2" PVC solid pipe, stickup `2.5' 14-19' 2" PVC factory slotted pipe 0-12' cuttings 12-14' bentonite chips 14-19' 10/20 sand					

APPENDIX B
Vibrating Wire Piezometer Calibration Sheets

VW Piezometer Calibration Certificate

Serial #: 1600515
Range : 350 kPa
Cable Length: 15 m
Date of Calibration: 3/8/2016

Part #: 52611028
Cable Part # : 50613524
Calibrated by: AM
Note:

ABC Calibration Factors

	A	B	C
kPa	-1.154951E-4	-2.102657E-3	9.611544E+2
psi	-1.675115E-5	-3.049646E-4	1.394037E+2

Pressure in kPa/psi = $(A \times Hz^2) + (B \times Hz) + C$, where Hz is frequency in Hertz.

TI Calibration Factors

	C0	C1	C2	C3	C4	C5
kPa	9.600124E+2	-2.966179E-3	1.115445E-1	-1.154492E-4	4.916590E-5	-1.680620E-3
psi	1.392331E+2	-4.301927E-4	1.617759E-2	-1.674390E-5	7.130660E-6	-2.437447E-4

Pressure in kPa/psi = $C0 + (C1 \times Hz) + (C2 \times T) + (C3 \times Hz^2) + (C4 \times Hz \times T) + (C5 \times T^2)$

Where Hz is the frequency reading in Hertz and T is the Thermistor reading in degrees C.
TI factors are calculated from temperatures at 5.0, 15.0 and 25.0 degrees C.
Applied pressure and temperature are NIST traceable.

Summary of Test Results at 15℃

Thermistor reading is 14.3 °C.

Applied Pressure is referenced to 1 atm. Calculated Pressure uses ABC Calibration factors.

Applied (kPa)	Equivalent (psi)	Frequency (Hz)	Calculated (kPa)	Calculated (psi)	Error (%FS)
0.0	0.00	2875.5	0.1	0.02	-0.04
35.0	5.08	2822.7	35.0	5.08	0.00
70.0	10.15	2768.8	69.9	10.14	0.02
105.0	15.23	2713.8	104.9	15.21	0.04
140.0	20.31	2657.6	139.8	20.28	0.04
175.0	25.38	2600.1	174.9	25.36	0.03
210.0	30.46	2540.9	210.2	30.48	-0.04
245.0	35.53	2480.6	245.2	35.57	-0.07
280.0	40.61	2419.2	280.1	40.63	-0.04
315.0	45.69	2356.3	314.9	45.68	0.01
350.0	50.76	2291.5	349.9	50.74	0.04

VW Piezometer Calibration Certificate

Serial #: 1600635
Range : 350 kPa
Cable Length: 30 m
Date of Calibration: 3/17/2016

Part #: 52611024
Cable Part # : 50613524
Calibrated by: AM
Note:

ABC Calibration Factors

	A	B	C
kPa	-1.155117E-4	-1.467395E-2	9.819175E+2
psi	-1.675356E-5	-2.128277E-3	1.424151E+2

Pressure in kPa/psi $= (A \times Hz^2) + (B \times Hz) + C$, where Hz is frequency in Hertz.

TI Calibration Factors

	C0	C1	C2	C3	C4	C5
kPa	9.815833E+2	-1.616976E-2	1.117999E-1	-1.153392E-4	4.551953E-5	-1.442945E-3
psi	1.423616E+2	-2.345143E-3	1.621463E-2	-1.672795E-5	6.601817E-6	-2.092741E-4

Pressure in kPa/psi $= C0 + (C1 \times Hz) + (C2 \times T) + (C3 \times Hz^2) + (C4 \times Hz \times T) + (C5 \times T^2)$

Where Hz is the frequency reading in Hertz and T is the Thermistor reading in degrees C.
TI factors are calculated from temperatures at 5.0, 15.0 and 25.0 degrees C.
Applied pressure and temperature are NIST traceable.

Summary of Test Results at 15℃

Thermistor reading is 14.6 ^{o}C.

Applied Pressure is referenced to 1 atm. Calculated Pressure uses ABC Calibration factors.

Applied (kPa)	Equivalent (psi)	Frequency (Hz)	Calculated (kPa)	Calculated (psi)	Error (%FS)
0.0	0.00	2852.7	0.0	0.00	-0.01
35.0	5.08	2800.3	35.0	5.08	-0.01
70.0	10.15	2747.1	69.9	10.14	0.03
105.0	15.23	2692.5	105.0	15.23	0.00
140.0	20.31	2637.0	140.0	20.30	0.01
175.0	25.38	2580.3	175.0	25.38	0.00
210.0	30.46	2522.3	210.0	30.46	-0.01
245.0	35.53	2463.0	245.0	35.54	-0.01
280.0	40.61	2402.3	280.0	40.62	-0.01
315.0	45.69	2340.1	315.0	45.69	-0.01
350.0	50.76	2276.4	349.9	50.75	0.02

VW Piezometer Calibration Certificate

Serial #: 1600636
Range : 350 kPa
Cable Length: 30 m
Date of Calibration: 3/17/2016

Part #: 52611024
Cable Part # : 50613524
Calibrated by: AM
Note:

ABC Calibration Factors

	A	B	C
kPa	-8.667330E-5	-1.378747E-1	1.089942E+3
psi	-1.257090E-5	-1.999704E-2	1.580827E+2

Pressure in kPa/psi $= (A \times Hz^2) + (B \times Hz) + C$, where Hz is frequency in Hertz.

TI Calibration Factors

	C0	C1	C2	C3	C4	C5
kPa	1.091786E+3	-1.410319E-1	1.216177E-1	-8.619855E-5	4.549459E-5	-1.687207E-3
psi	1.583446E+2	-2.045423E-2	1.763854E-2	-1.250160E-5	6.598200E-6	-2.447001E-4

Pressure in kPa/psi $= C0 + (C1 \times Hz) + (C2 \times T) + (C3 \times Hz^2) + (C4 \times Hz \times T) + (C5 \times T^2)$

Where Hz is the frequency reading in Hertz and T is the Thermistor reading in degrees C.
TI factors are calculated from temperatures at 5.0, 15.0 and 25.0 degrees C.
Applied pressure and temperature are NIST traceable.

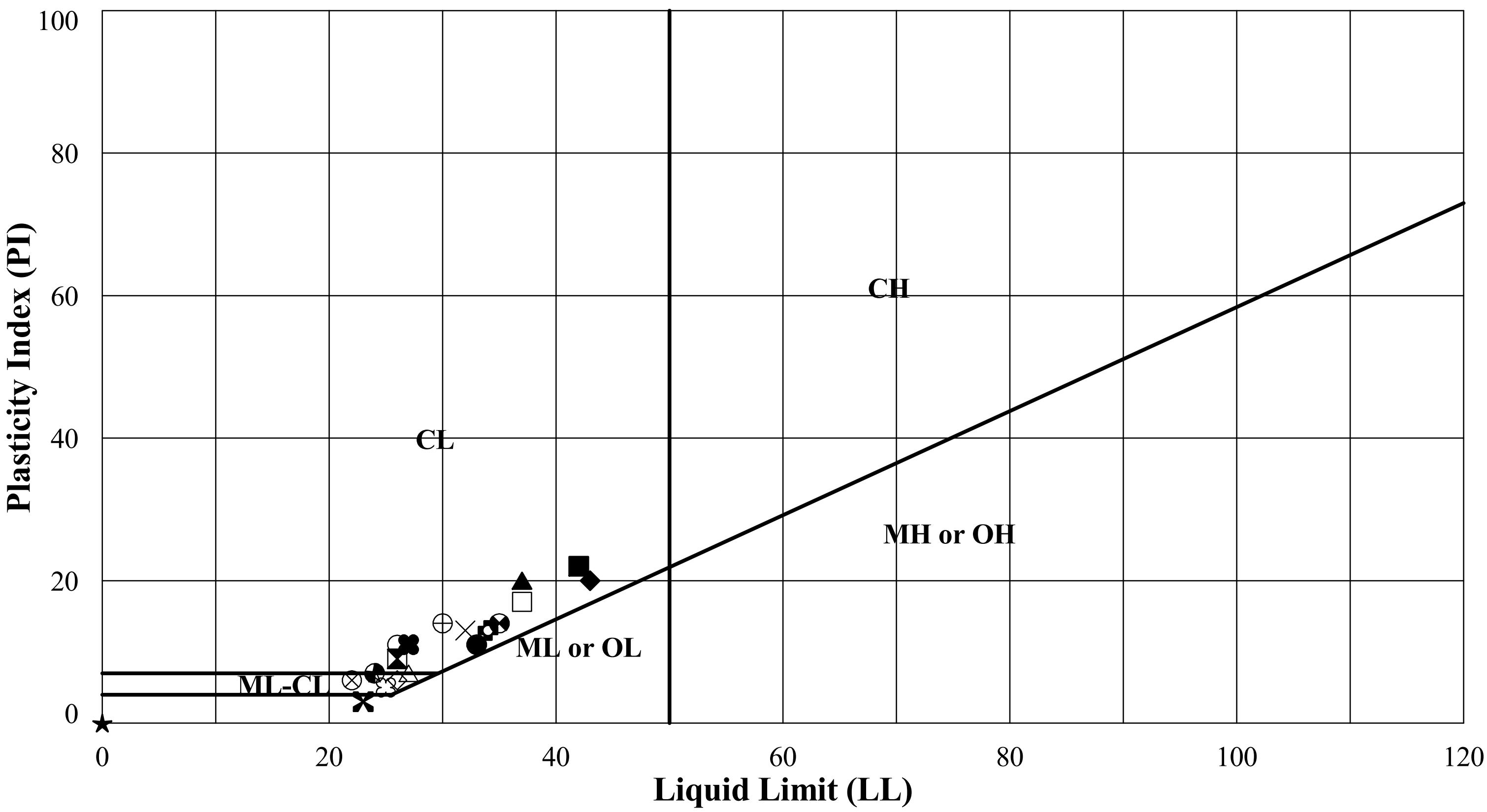
Summary of Test Results at 15℃

Thermistor reading is 14.6 °C.

Applied Pressure is referenced to 1 atm. Calculated Pressure uses ABC Calibration factors.

Applied (kPa)	Equivalent (psi)	Frequency (Hz)	Calculated (kPa)	Calculated (psi)	Error (%FS)
0.0	0.00	2838.7	0.1	0.02	-0.04
35.0	5.08	2782.9	35.0	5.08	0.00
70.0	10.15	2726.2	69.9	10.14	0.03
105.0	15.23	2668.4	104.9	15.21	0.03
140.0	20.31	2609.7	139.8	20.28	0.05
175.0	25.38	2549.7	174.9	25.37	0.02
210.0	30.46	2488.6	210.1	30.47	-0.01
245.0	35.53	2426.4	245.1	35.55	-0.03
280.0	40.61	2363.0	280.2	40.64	-0.05
315.0	45.69	2298.5	315.1	45.71	-0.04
350.0	50.76	2233.3	349.7	50.72	0.08

APPENDIX C
Laboratory Test Results



Legend	Boring	Sample No.	Depth	LL	PL	PI	P 200, %	MC	Classification
●	JG-1	D7	17.5' to 19.0'	33	22	11	65.0	24.5%	CL
⧗	JG-1	D8	20.0' to 21.5'	26	17	9	61.0	13.2%	CL
▲	JG-1	D11	27.5' to 29.0'	37	17	20	80.6	21.6%	CL
★	JG-2	D3	7.5' to 9.0'	NP	NP	NP	96.0	13.5%	ML
✖	JG-2	D4	10.0' to 11.5'	23	20	3	93.0	12.7%	ML
✿	JG-2	D10	27.5' to 29.0'	34	21	13	93.0	32.9%	CL
○	JG-2	D11	30.0' to 31.5'	26	15	11	55.9	27.9%	CL
△	JG-3	D6	15.0' to 16.5'	27	20	7	79.4	26.4%	CL-ML
⊗	JG-3	D9	22.5' to 24.0'	22	16	6	84.9	22.6%	CL-ML
⊕	JG-3	D10	27.5' to 29.0'	30	16	14	87.0	26.8%	CL
□	JG-4	D1	2.5' to 4.0'	37	20	17	94.4	28.4%	CL
◉	JG-4	D2	5.0' to 6.5'	35	21	14	96.6	29.0%	CL
◕	JG-4	D8	22.5' to 24.0'	24	17	7	87.0	20.9%	CL-ML
☆	JG-4	D9	25.0' to 27.5'	24	17	7	81.0	15.7%	CL-ML
⌘	JG-5	D1	5.0' to 7.5'	25	20	5	87.3	25.5%	CL-ML
■	JG-5	D7	30.0' to 31.5'	42	20	22	75.0	25.2%	CL
◆	JG-5	D8	32.5' to 33.0'	43	23	20	79.5	30.0%	CL
◇	JG-6	D2	15.0' to 16.5'	26	20	6	79.7	13.8%	CL-ML
✕	JG-6	D9	35.0' to 36.5'	32	19	13	48.0	16.0%	SC
✤	JG-6	D10	37.5' to 39.0'	27	16	11	72.8	19.3%	CL

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Phone: 406.652.3930
Fax: 406.652.3944

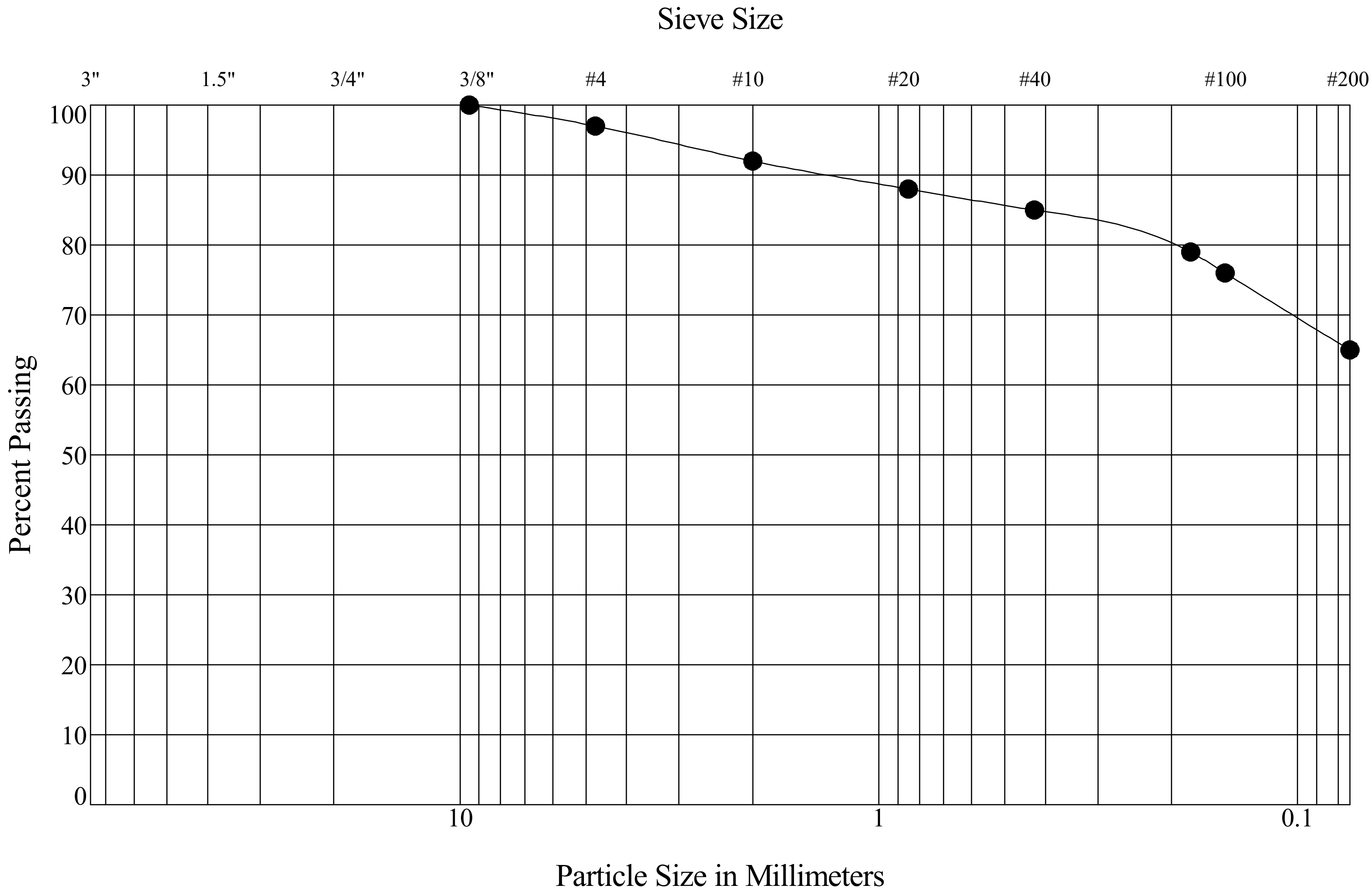
Atterberg Limits' Tests

Project Number: 15-3404L

West View Townhomes

09040.01.30

6/27/16



Gravel		Sand		
coarse	fine	coarse	medium	fine

Percent Passing U.S. Standard Sieve Size

3"	1 1/2"	3/4"	3/8"	#4	#10	#20	#40	#80	#100	#200
			100	97	92	88	85	79	76	65

Borehole: JG-1
Sample No.: D7
Depth: 17.5' to 19.0'

Date Received: 06/17/2016

Liquid Limit:	33
Plastic Limit:	22
Plasticity Index:	11
Classification:	CL
Moisture Content:	24.5%

Percent Gravel: 3.0
Percent Sand: 32.0
Percent Silt + Clay: 65.0
ASTM Group Name: SANDY LEAN CLAY

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Billings, MT 59108-0190
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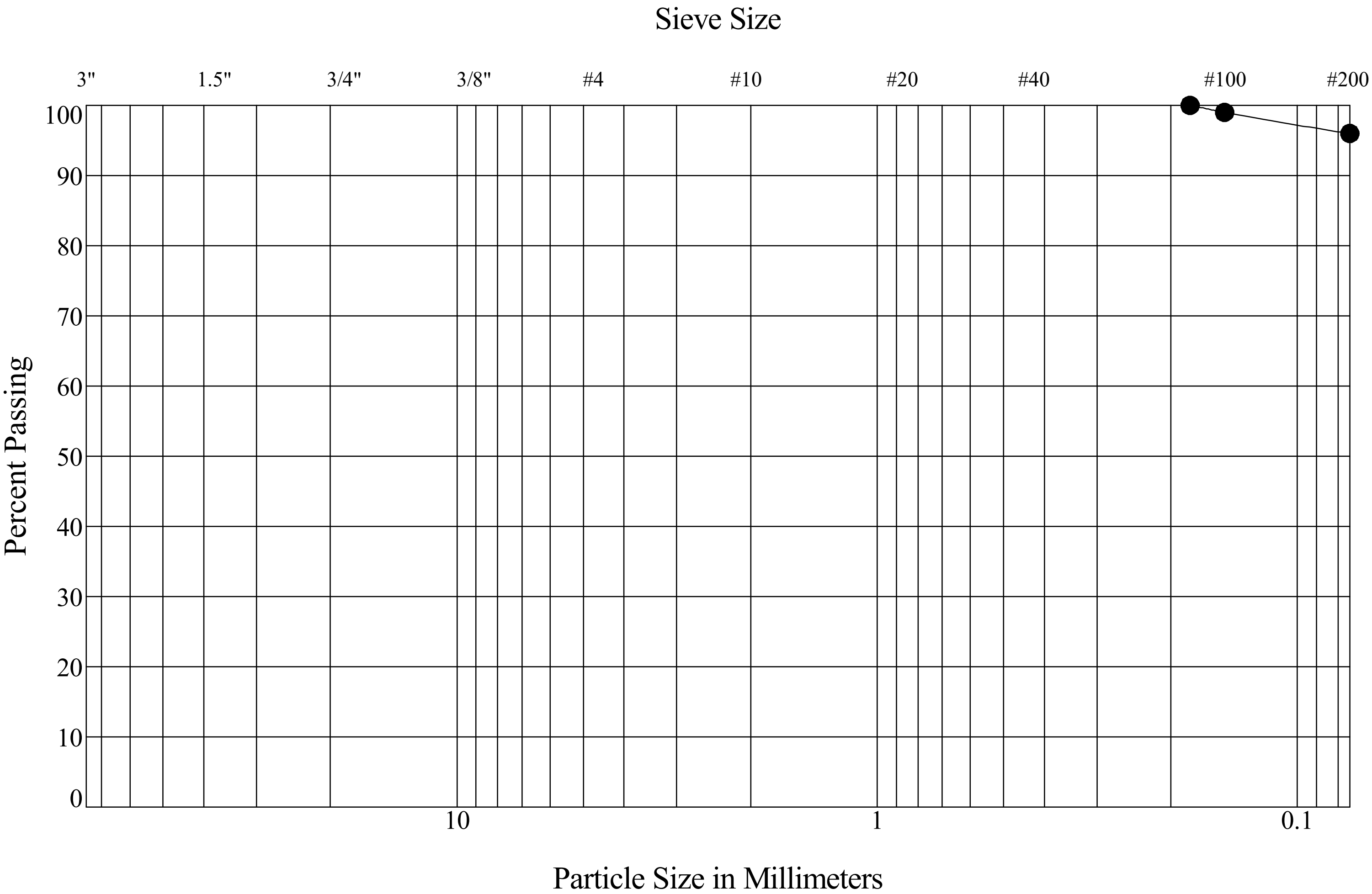
Fax: 406.652.3944

Sieve Analysis
Project Number: 15-3404L
West View Townhomes
09040.01.30

6/23/16



Gravel		Sand		
coarse	fine	coarse	medium	fine

Percent Passing U.S. Standard Sieve Size

3"	1 1/2"	3/4"	3/8"	#4	#10	#20	#40	#80	#100	#200
								100	99	96

Borehole: JG-2
Sample No.: D3
Depth: 7.5' to 9.0'

Date Received: 06/17/2016

Liquid Limit:	NP
Plastic Limit:	NP
Plasticity Index:	NP
Classification:	ML
Moisture Content:	13.5%

Percent Gravel: 0.0
Percent Sand: 4.0
Percent Silt + Clay: 96.0
ASTM Group Name: SILT

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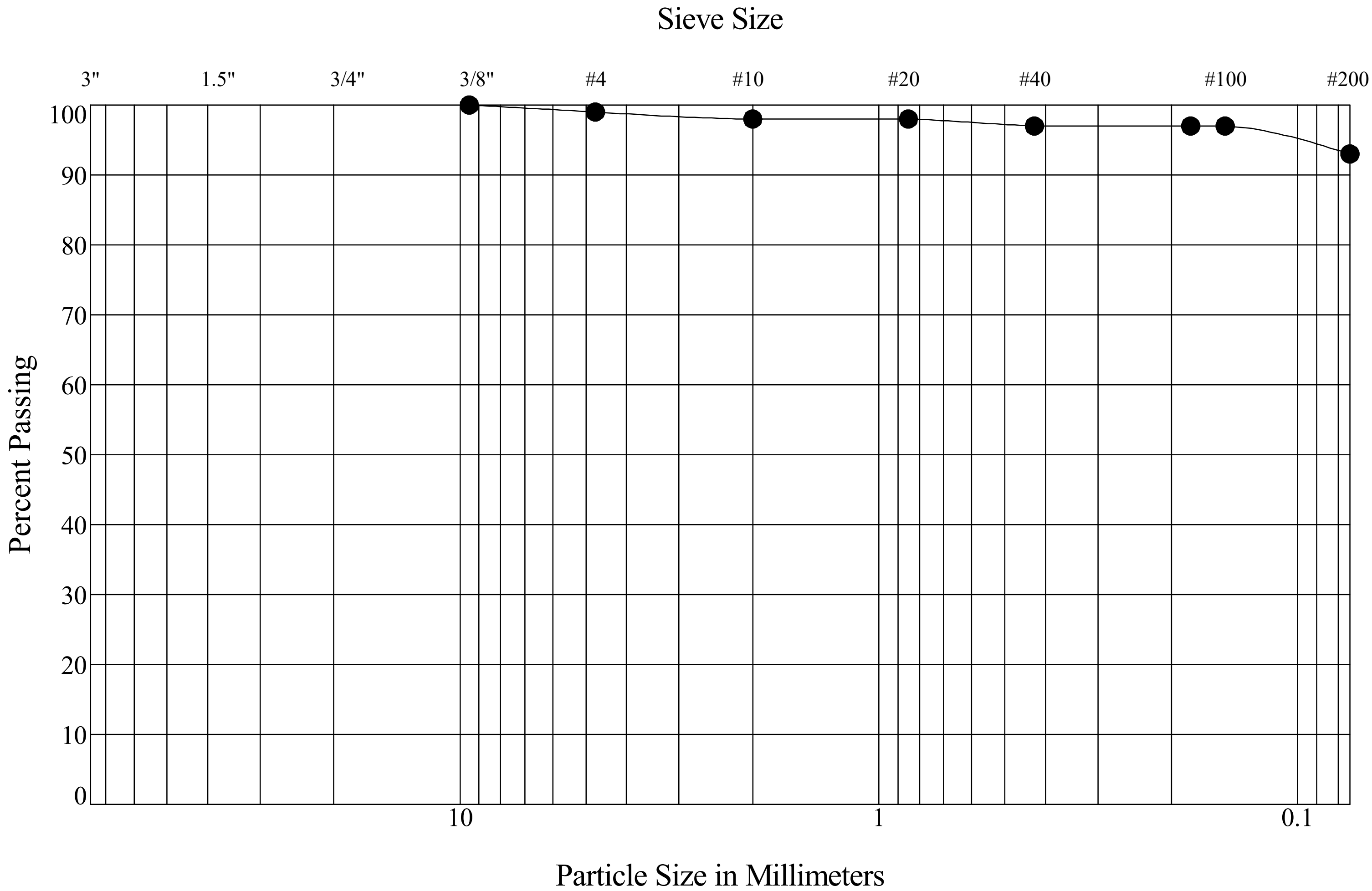
Fax: 406.652.3944

Sieve Analysis

Project Number: 15-3404L
West View Townhomes
09040.01.30

6/23/16



Gravel		Sand		
coarse	fine	coarse	medium	fine

Percent Passing U.S. Standard Sieve Size

3"	1 1/2"	3/4"	3/8"	#4	#10	#20	#40	#80	#100	#200
			100	99	98	98	97	97	97	93

Borehole: JG-2
Sample No.: D4
Depth: 10.0' to 11.5'

Date Received: 06/17/2016

Liquid Limit:	23
Plastic Limit:	20
Plasticity Index:	3
Classification:	ML
Moisture Content:	12.7%

Percent Gravel: 1.0
Percent Sand: 6.0
Percent Silt + Clay: 93.0
ASTM Group Name: SILT

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Sieve Analysis

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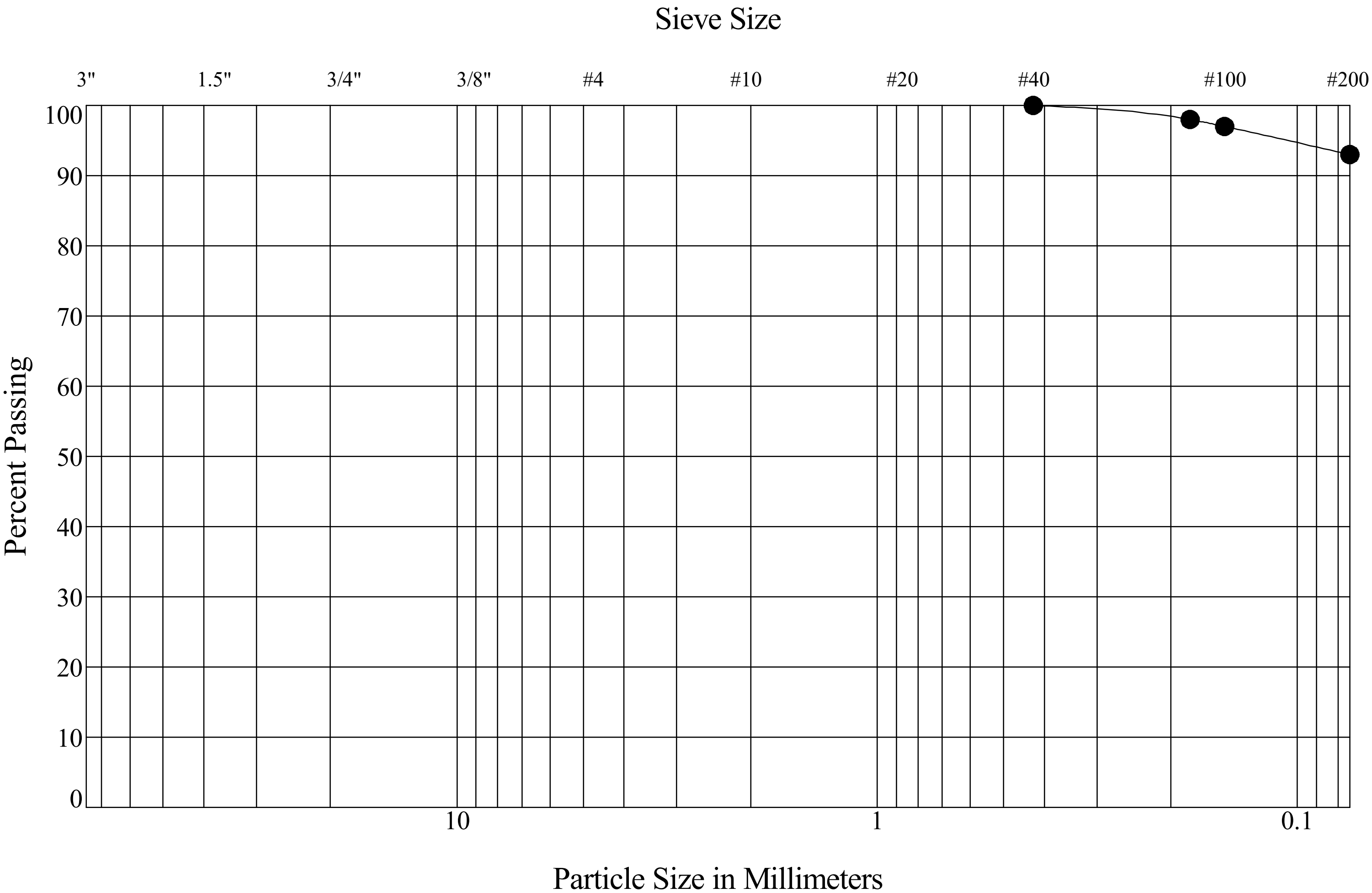

Gravel		Sand		
coarse	fine	coarse	medium	fine

Percent Passing U.S. Standard Sieve Size

3"	1 1/2"	3/4"	3/8"	#4	#10	#20	#40	#80	#100	#200
							100	98	97	93

Borehole: JG-2
Sample No.: D10
Depth: 27.5' to 29.0'

Date Received: 06/17/2016

Liquid Limit:	34
Plastic Limit:	21
Plasticity Index:	13
Classification:	CL
Moisture Content:	32.9%

Percent Gravel: 0.0
Percent Sand: 7.0
Percent Silt + Clay: 93.0
ASTM Group Name: LEAN CLAY

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Sieve Analysis
Project Number: 15-3404L
West View Townhomes
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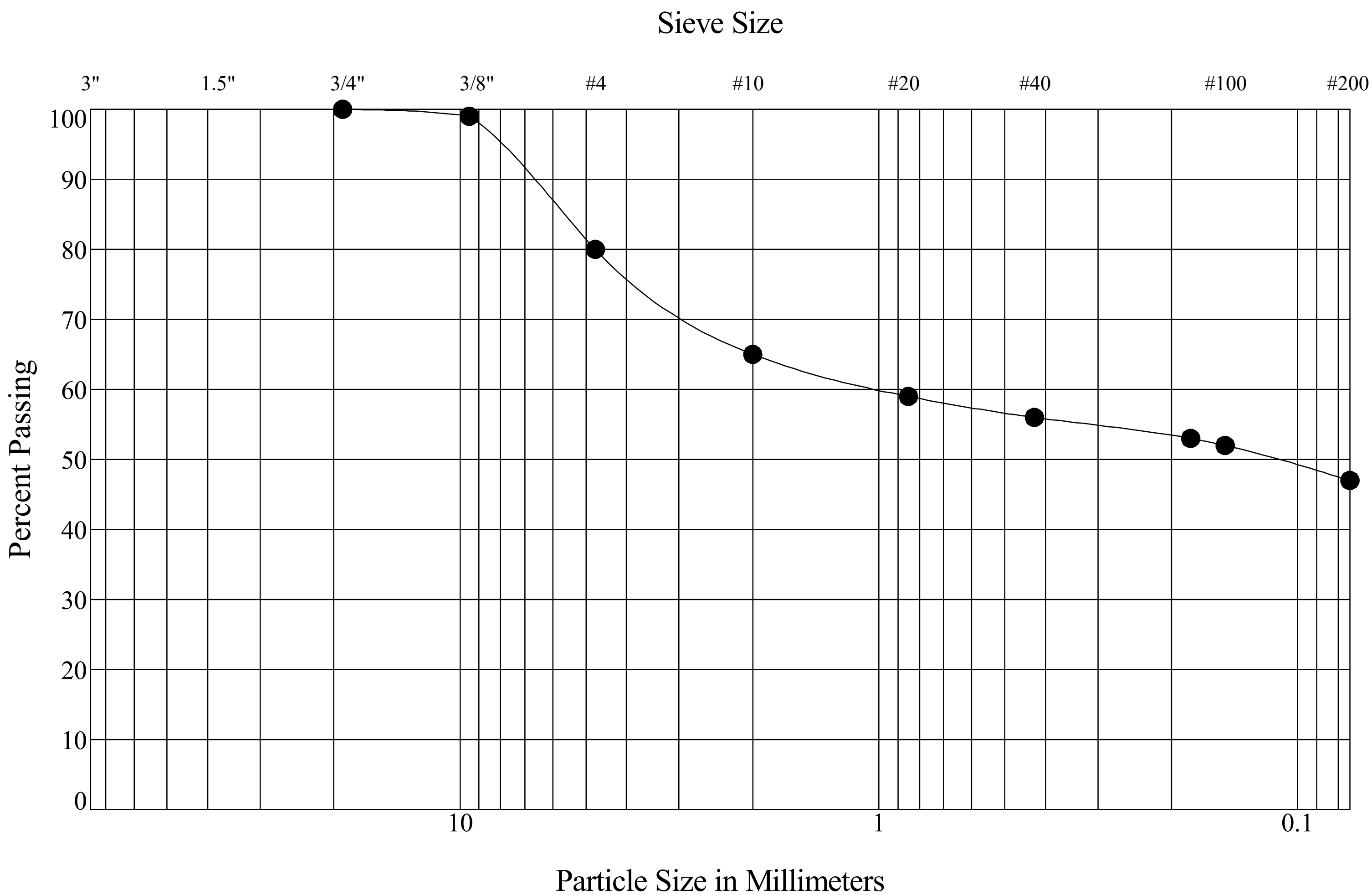
6/23/16



Gravel		Sand		
coarse	fine	coarse	medium	fine

Percent Passing U.S. Standard Sieve Size

3"	1 1/2"	3/4"	3/8"	#4	#10	#20	#40	#80	#100	#200
		100	99	80	65	59	56	53	52	47

Borehole: JG-3
Sample No.: D3
Depth: 7.5' to 9.0'

Date Received: 06/17/2016

Percent Gravel: 20.0
Percent Sand: 33.0
Percent Silt + Clay: 47.0
ASTM Group Name:

Liquid Limit:	
Plastic Limit:	
Plasticity Index:	
Classification:	
Moisture Content:	10.3%

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Sieve Analysis
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West View Townhomes
09040.01.30

6/23/16

Gravel		Sand		
coarse	fine	coarse	medium	fine

Percent Passing U.S. Standard Sieve Size

3"	1 1/2"	3/4"	3/8"	#4	#10	#20	#40	#80	#100	#200
				100	99	98	96	94	93	87

Borehole: JG-3
Sample No.: D10
Depth: 27.5' to 29.0'

Date Received: 06/17/2016

Liquid Limit:	30
Plastic Limit:	16
Plasticity Index:	14
Classification:	CL
Moisture Content:	26.8%

Percent Gravel: 0.0
Percent Sand: 13.0
Percent Silt + Clay: 87.0
ASTM Group Name: LEAN CLAY

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Sieve Analysis
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West View Townhomes
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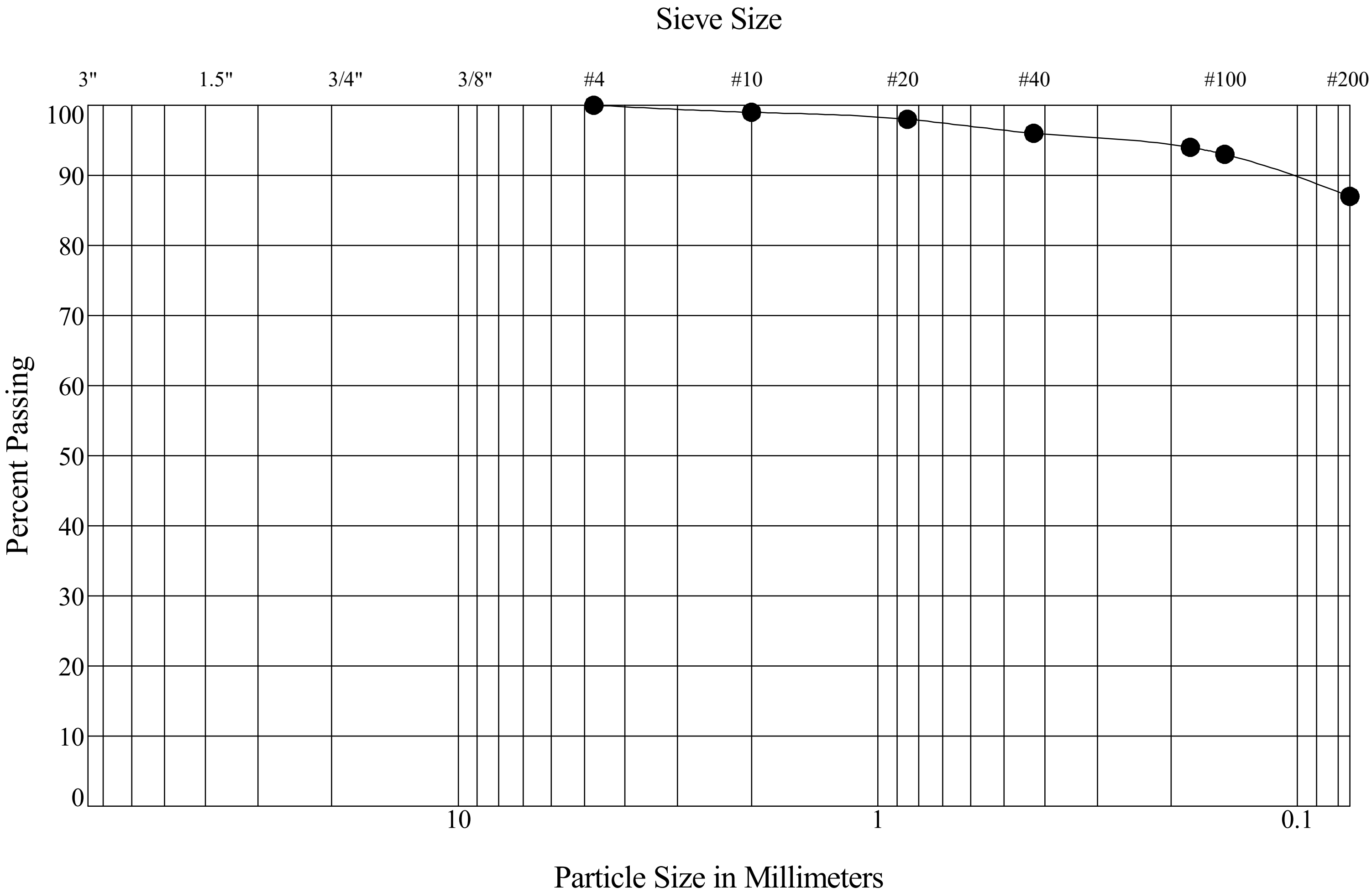
6/23/16



Gravel		Sand		
coarse	fine	coarse	medium	fine

Percent Passing U.S. Standard Sieve Size

3"	1 1/2"	3/4"	3/8"	#4	#10	#20	#40	#80	#100	#200
				100	99	98	96	94	93	87

Borehole: JG-4
Sample No.: D8
Depth: 22.5' to 24.0'

Date Received: 06/17/2016

Liquid Limit:	24
Plastic Limit:	17
Plasticity Index:	7
Classification:	CL-ML
Moisture Content:	20.9%

Percent Gravel: 0.0
Percent Sand: 13.0
Percent Silt + Clay: 87.0
ASTM Group Name: SILTY CLAY

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Sieve Analysis
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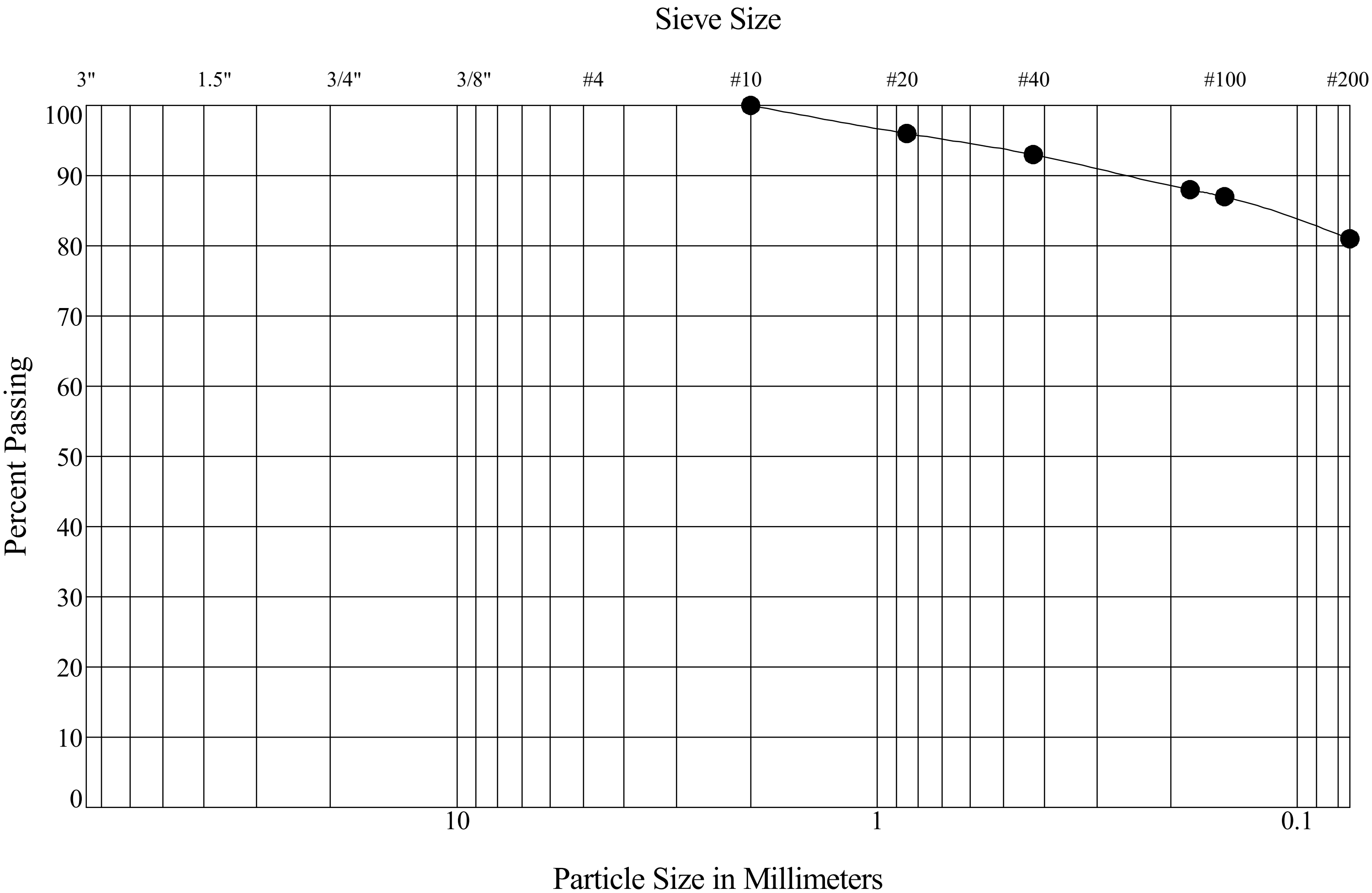
West View Townhomes
09040.01.30

6/23/16



Gravel		Sand		
coarse	fine	coarse	medium	fine

Percent Passing U.S. Standard Sieve Size

3"	1 1/2"	3/4"	3/8"	#4	#10	#20	#40	#80	#100	#200
					100	96	93	88	87	81

Borehole: JG-4
Sample No.: D9
Depth: 25.0' to 27.5'

Date Received: 06/17/2016

Liquid Limit:	24
Plastic Limit:	17
Plasticity Index:	7
Classification:	CL-ML
Moisture Content:	15.7%

Percent Gravel: 0.0
Percent Sand: 19.0
Percent Silt + Clay: 81.0
ASTM Group Name: SILTY CLAY with SAND

SK GEOTECHNICAL
2511 Holman Avenue
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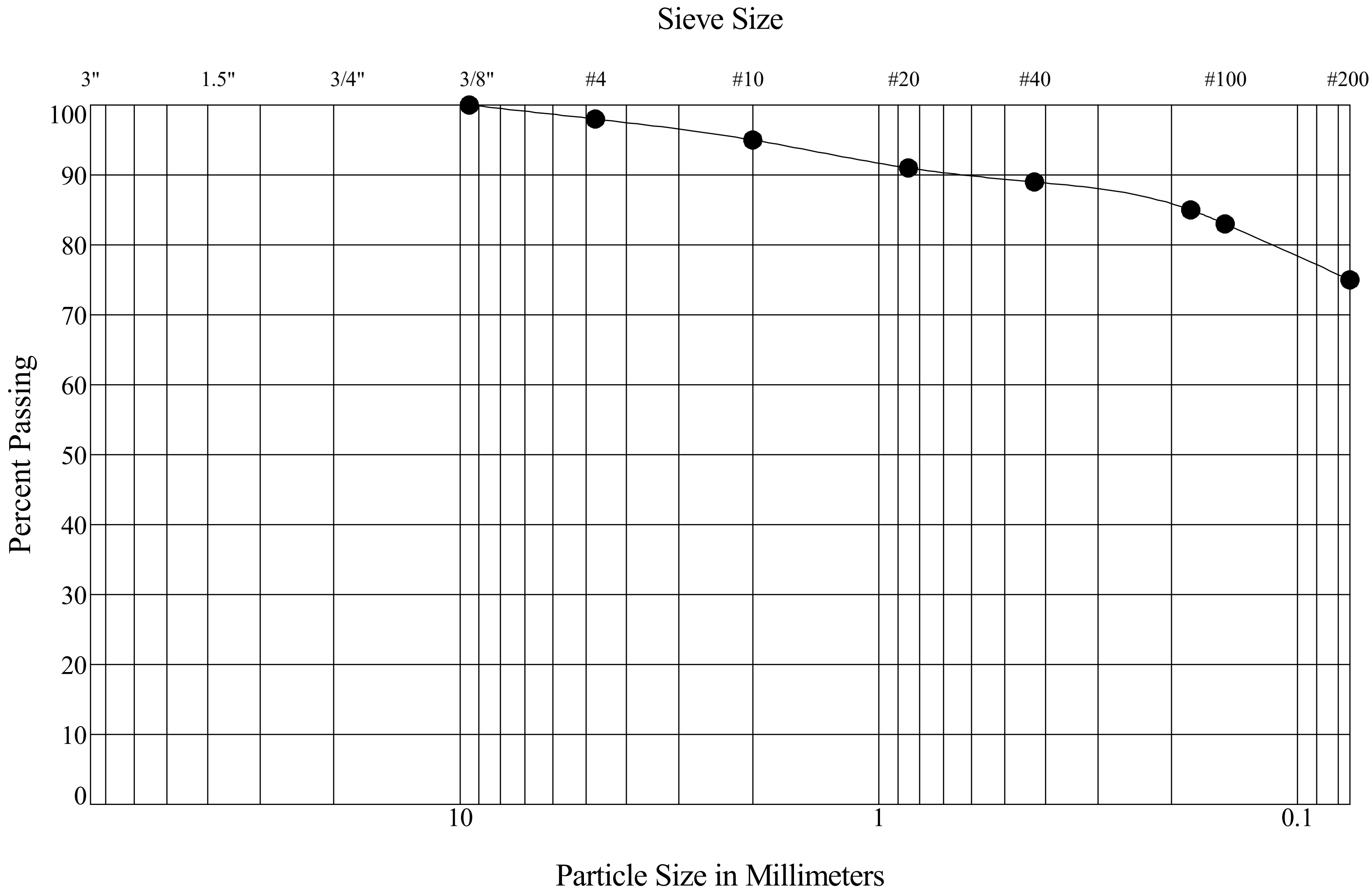
Billings, MT 59108-0190
Phone: 406.652.3930
Fax: 406.652.3944

Sieve Analysis
Project Number: 15-3404L
West View Townhomes
09040.01.30

6/23/16



Gravel		Sand		
coarse	fine	coarse	medium	fine

Percent Passing U.S. Standard Sieve Size

3"	1 1/2"	3/4"	3/8"	#4	#10	#20	#40	#80	#100	#200
			100	98	95	91	89	85	83	75

Borehole: JG-5
Sample No.: D7
Depth: 30.0' to 31.5'

Date Received: 06/17/2016

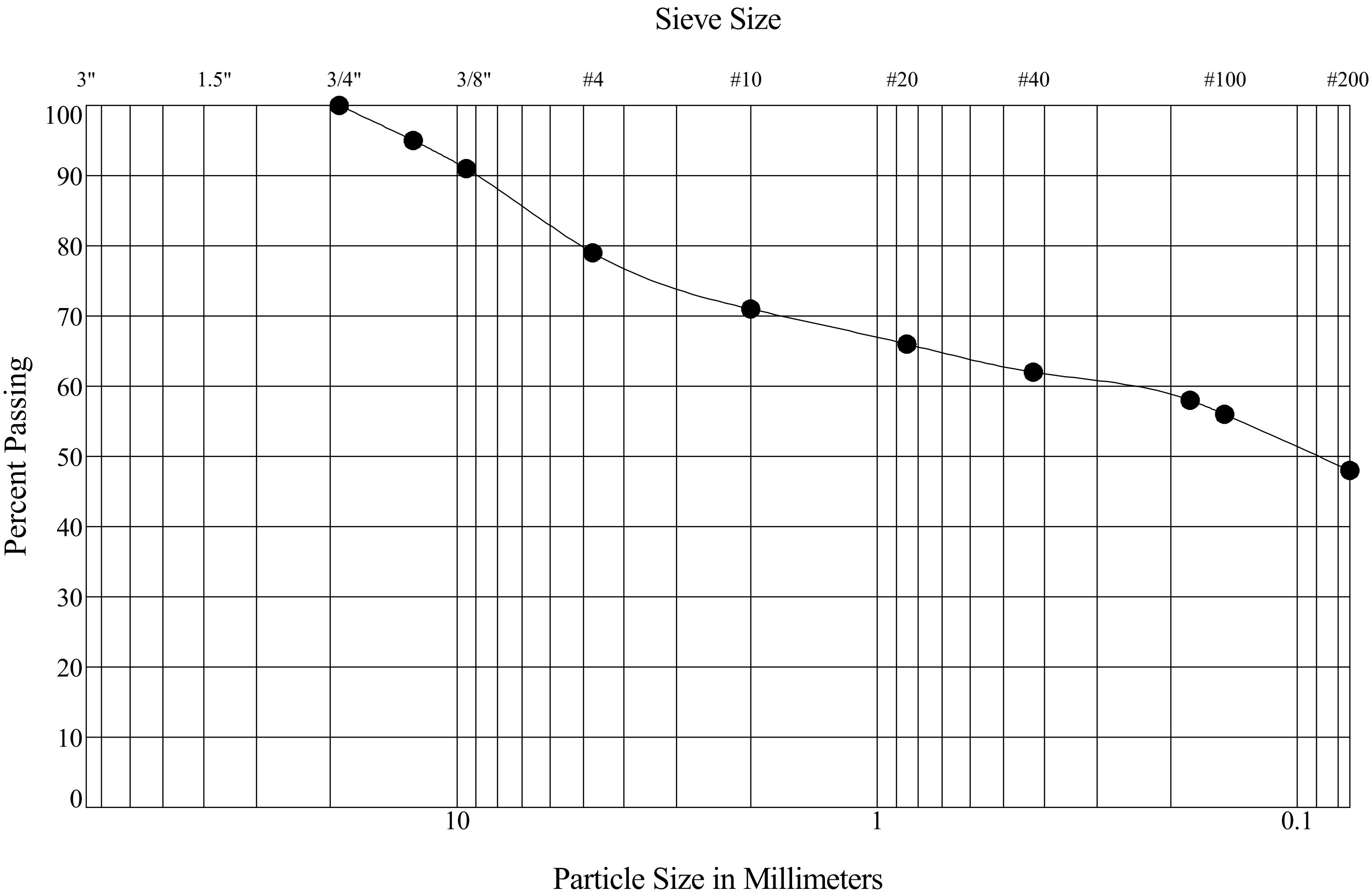
Liquid Limit:	42
Plastic Limit:	20
Plasticity Index:	22
Classification:	CL
Moisture Content:	25.2%

Percent Gravel: 2.0
Percent Sand: 23.0
Percent Silt + Clay: 75.0
ASTM Group Name: LEAN CLAY with SAND

SK GEOTECHNICAL
2511 Holman Avenue
P. O. Box 80190
Billings, MT 59108-0190
Phone: 406.652.3930
Fax: 406.652.3944

Sieve Analysis
Project Number: 15-3404L
West View Townhomes
09040.01.30

6/23/16



Gravel		Sand		
coarse	fine	coarse	medium	fine

Percent Passing U.S. Standard Sieve Size

3"	1 1/2"	3/4"	3/8"	#4	#10	#20	#40	#80	#100	#200
		100	91	79	71	66	62	58	56	48

Borehole: JG-6
Sample No.: D9
Depth: 35.0' to 36.5'

Date Received: 06/17/2016

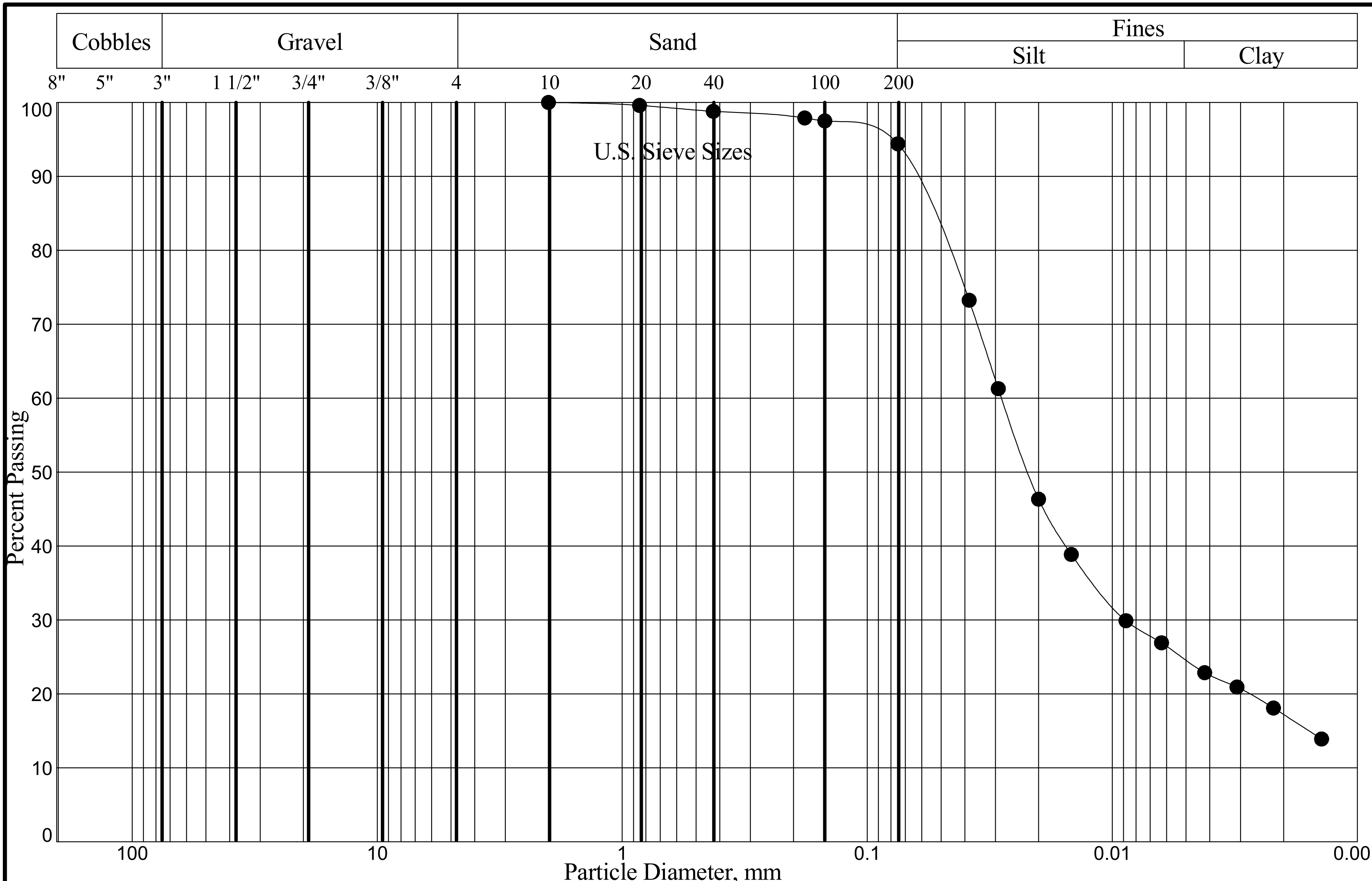
Liquid Limit:	32
Plastic Limit:	19
Plasticity Index:	13
Classification:	SC
Moisture Content:	16.0%

Percent Gravel: 21.0
Percent Sand: 31.0
Percent Silt + Clay: 48.0
ASTM Group Name: CLAYEY SAND with GRAVEL

SK GEOTECHNICAL
2511 Holman Avenue
P. O. Box 80190
Billings, MT 59108-0190
Phone: 406.652.3930
Fax: 406.652.3944

Sieve Analysis
Project Number: 15-3404L
West View Townhomes
09040.01.30

6/23/16



SK GEOTECHNICAL
2511 Holman Avenue
P. O. Box 80190
Billings, MT 59108-0190
Phone: 406.652.3930
Fax: 406.652.3944

Grain Size Analysis Curve

Project Number: 15-3404L

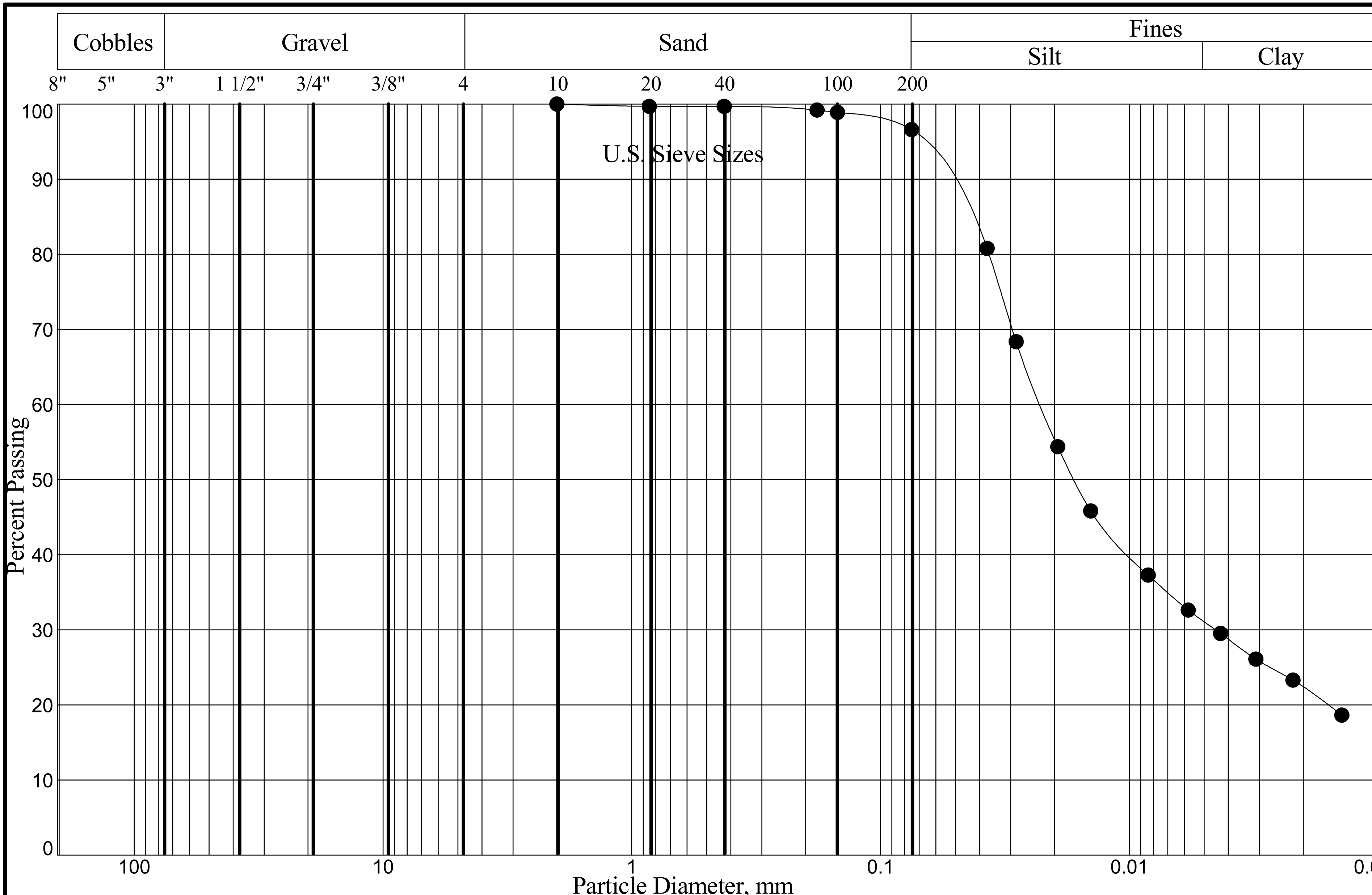
West View Townhomes

09040.01.30

Boring No.:	JG-4
Sample No:	D1
Depth:	2.5' to 4.0'
Date Received:	06/17/2016

6/27/16

% Gravel:	0.0	LL:	37
% Sand:	5.6	PL:	20
% Silt:	69.8	PI:	17
% Clay:	24.6	MC:	28.4%
Class:	CL	SG:	2.600
LEAN CLAY			



SK GEOTECHNICAL
2511 Holman Avenue
P. O. Box 80190
Billings, MT 59108-0190
Phone: 406.652.3930
Fax: 406.652.3944

Grain Size Analysis Curve

Project Number: 15-3404L

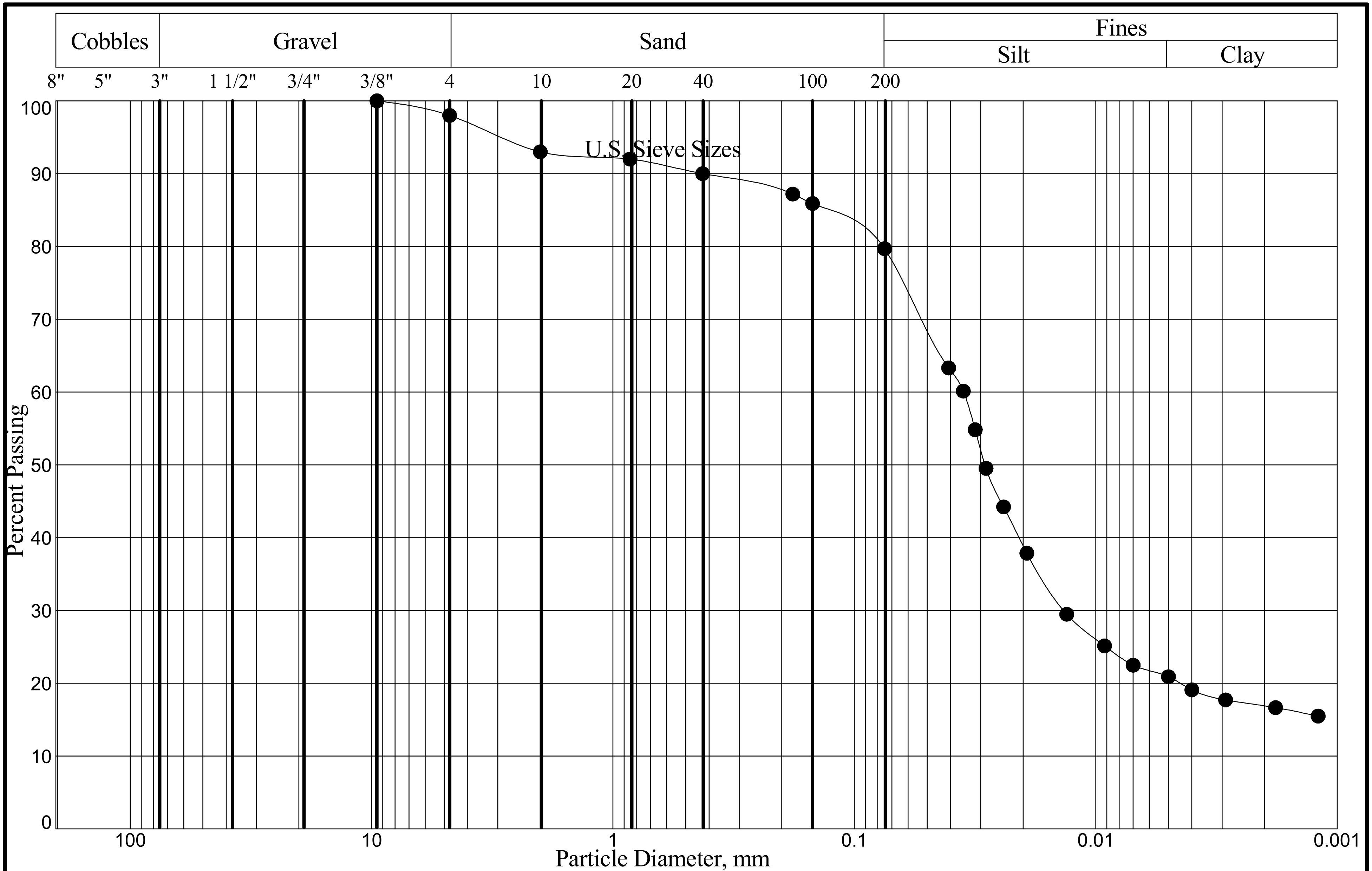
West View Townhomes

09040.01.30

Boring No.:	JG-4
Sample No:	D2
Depth:	5.0' to 6.5'
Date Received:	06/17/2016

6/27/16

% Gravel:	0.0	LL:	35
% Sand:	3.4	PL:	21
% Silt:	65.5	PI:	14
% Clay:	31.1	MC:	29.0%
Class:	CL	SG:	2.600
LEAN CLAY			



SK GEOTECHNICAL
2511 Holman Avenue
P. O. Box 80190
Billings, MT 59108-0190
Phone: 406.652.3930
Fax: 406.652.3944

Grain Size Analysis Curve

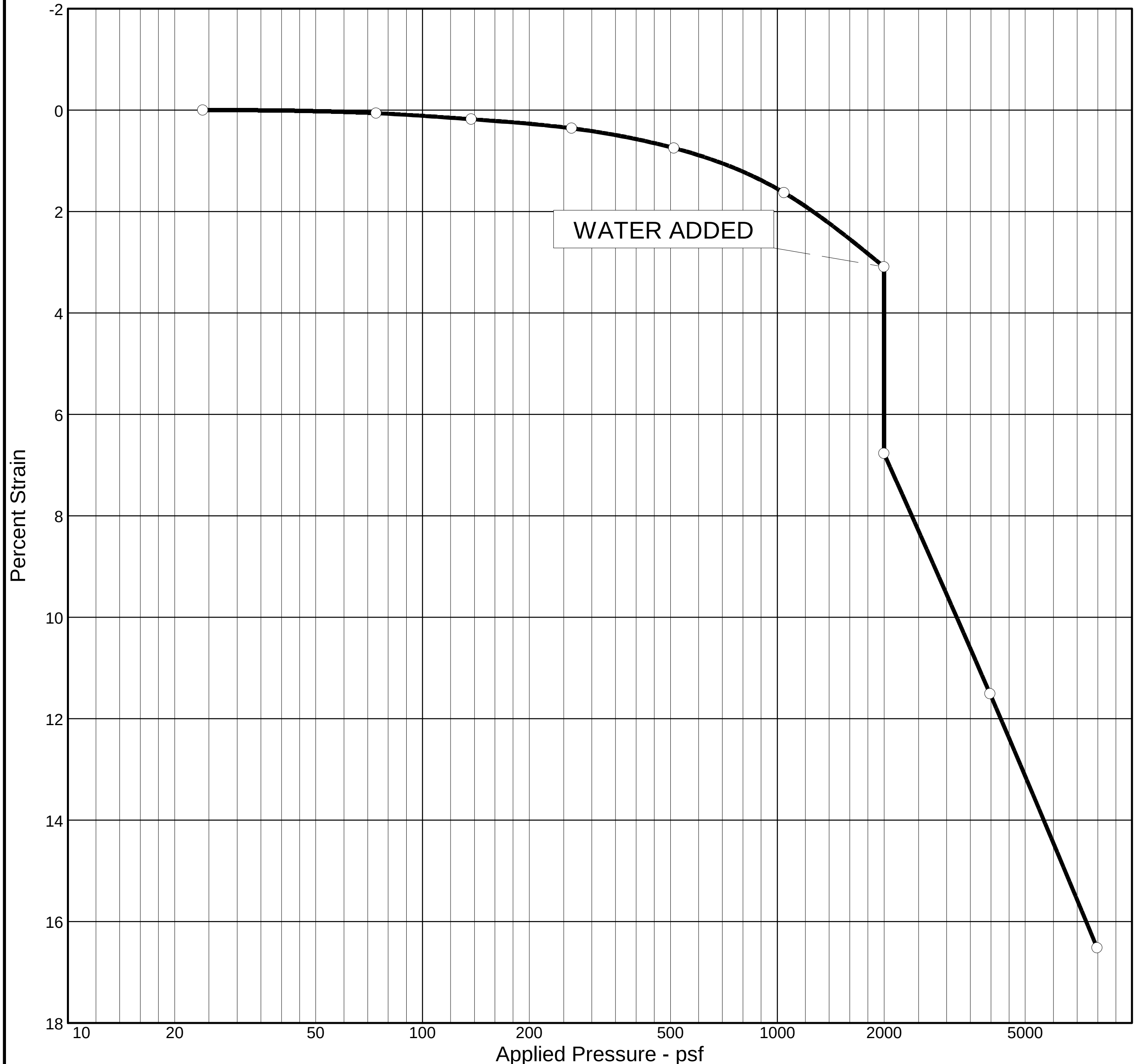
Project Number: 15-3404L

West View Townhomes

09040.01.30

Boring No.:	JG-6	% Gravel:	2.0	LL:	26
Sample No:	D2	% Sand:	18.3	PL:	20
Depth:	15.0' to 16.5'	% Silt:	58.8	PI:	6
Date Received:	06/23/2016	% Clay:	20.9	MC:	13.8%
		Class:	CL-ML	SG:	2.600
	6/27/16	SILTY CLAY with SAND			

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_r	Swell Press. (psf)	Clpse. %	e_o
Sat.	Moist.											
41.7 %	22.3 %	68.5			2.65	712	965	0.40			3.7	1.415

MATERIAL DESCRIPTION	USCS	AASHTO
Silt (ML), trace pinholes, FeO, and clay lenses, orangish brown, moist, loose		

Project No. 15-3404L **Client:** Jorgensen Associates, PC

Project: Jorgensen 09040.01.30, West View Townhomes

Location: JG-4 U1 Depth 7.5 - 8.5 ft

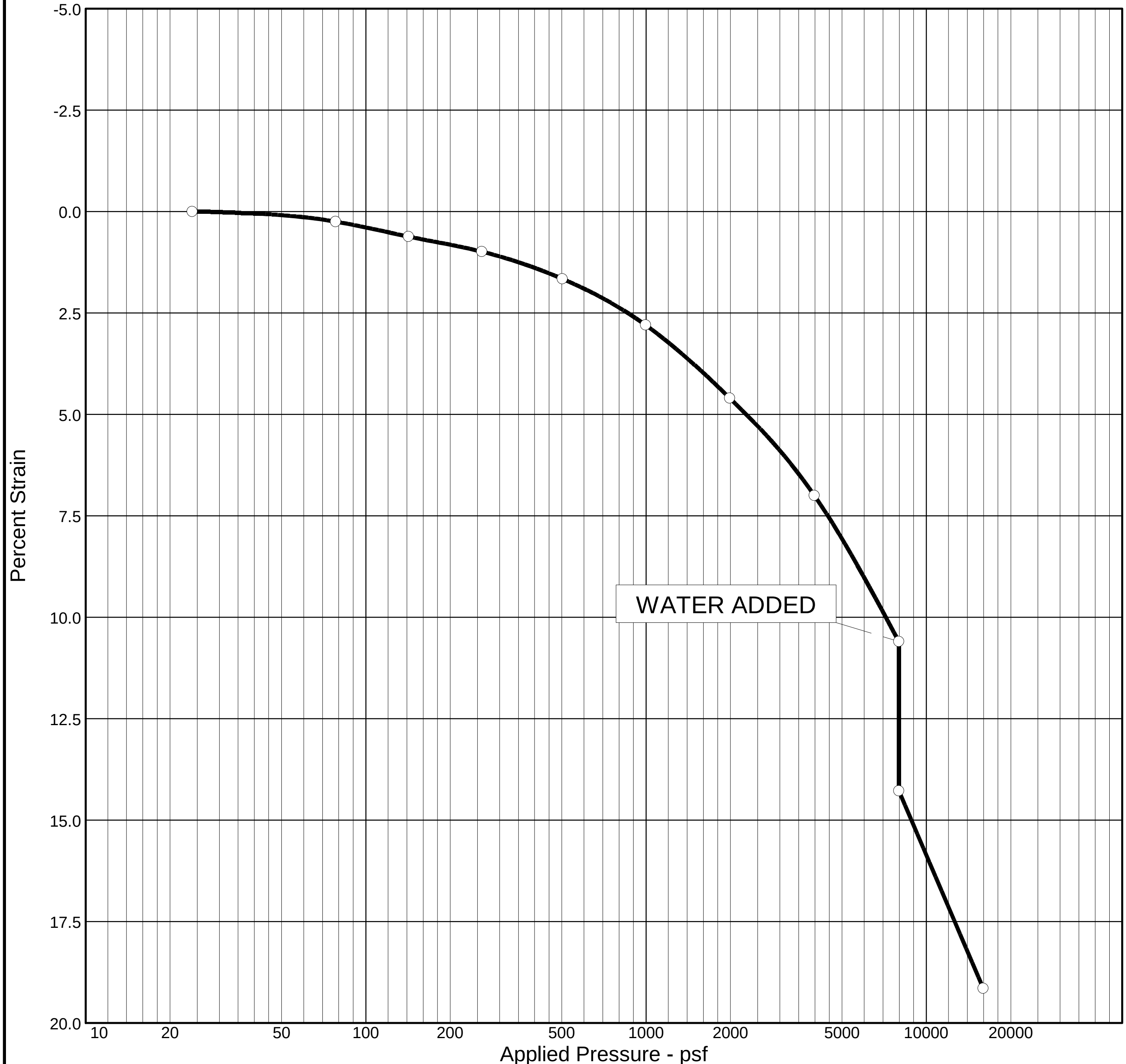
CONSOLIDATION TEST REPORT

SK GEOTECHNICAL CORP.

Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_r	Swell Press. (psf)	Clpse. %	e_o
Sat.	Moist.											
32.3 %	13.7 %	77.8			2.65	752	1566	0.34			3.7	1.126

MATERIAL DESCRIPTION	USCS	AASHTO
Silt (ML), trace pinholes and silt stone, orangish brown, moist, loose		

Project No. 15-3404L **Client:** Jorgensen Associates, PC

Project: Jorgensen 09040.01.30, West View Townhomes

Location: JG-6 U1 Depth 7.5 - 8.5 ft

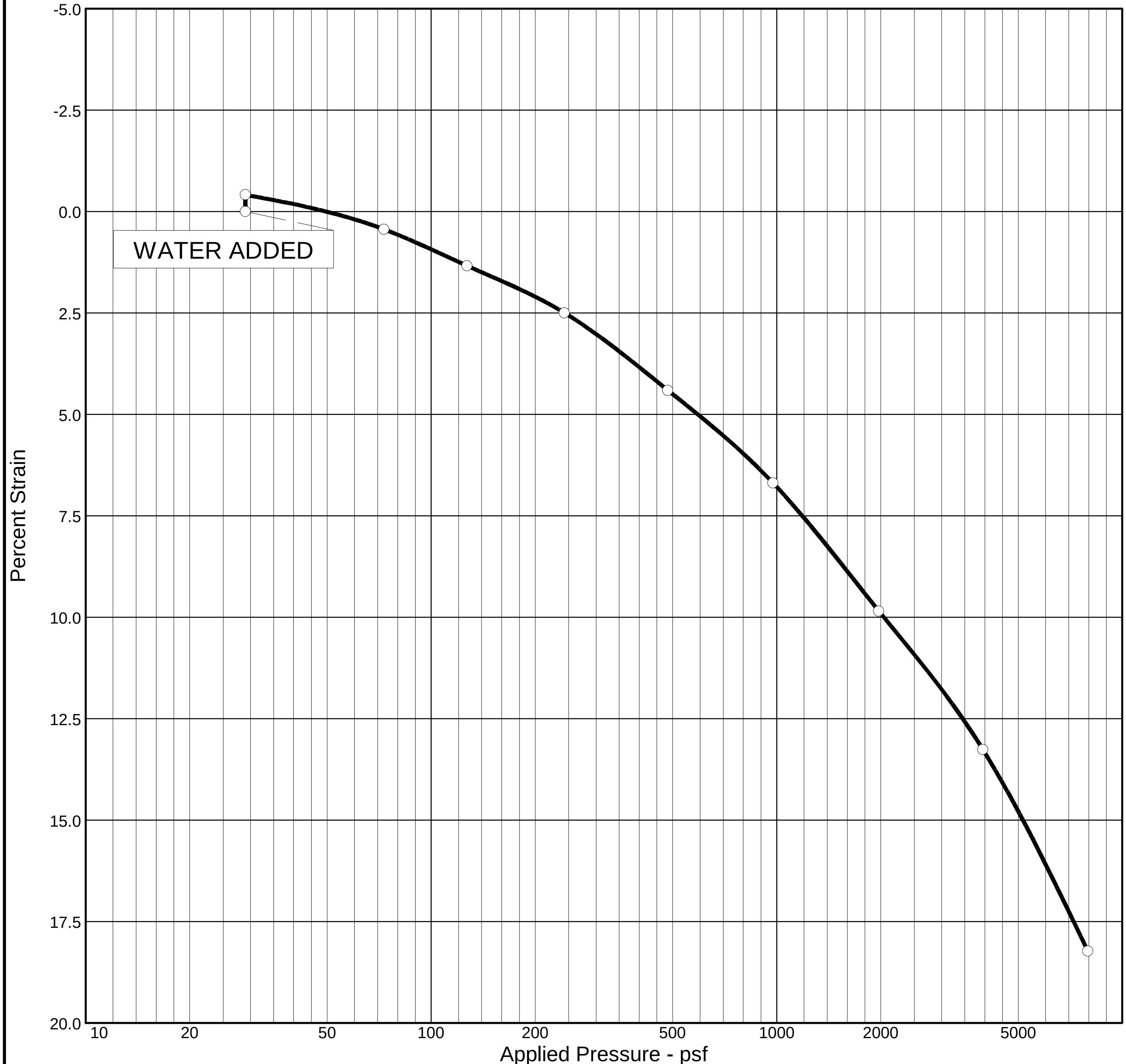
CONSOLIDATION TEST REPORT

SK GEOTECHNICAL CORP.

Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_r	Swell Press. (psf)	Swell %	e_o
Sat.	Moist.											
30.2 %	14.4 %	73.1			2.65	920	1475	0.38		50	0.4	1.262

MATERIAL DESCRIPTION	USCS	AASHTO
Silt (ML), trace pinholes, FeO, and clay lenses, orangish brown, moist, loose		

Project No. 15-3404L **Client:** Jorgensen Associates, PC

Project: Jorgensen 09040.01.30, West View Townhomes

Location: JG-6 U2 Depth 10.0 - 11.0 ft

CONSOLIDATION TEST REPORT

SK GEOTECHNICAL CORP.

Remarks:

Figure

Direct Shear of Soils Under Consolidated Drained Conditions, ASTM D3080

Date: June 22, 2016

Project: 15-3404L
Jorgensen 09040.01.30
West View Townhomes
Jackson, Wyoming

Client: Mr. Colter Lane
Jorgensen Associates, PC
PO Box 9550, 1315 HWY 89 S., Suite 201
Jackson, Wyoming 83002

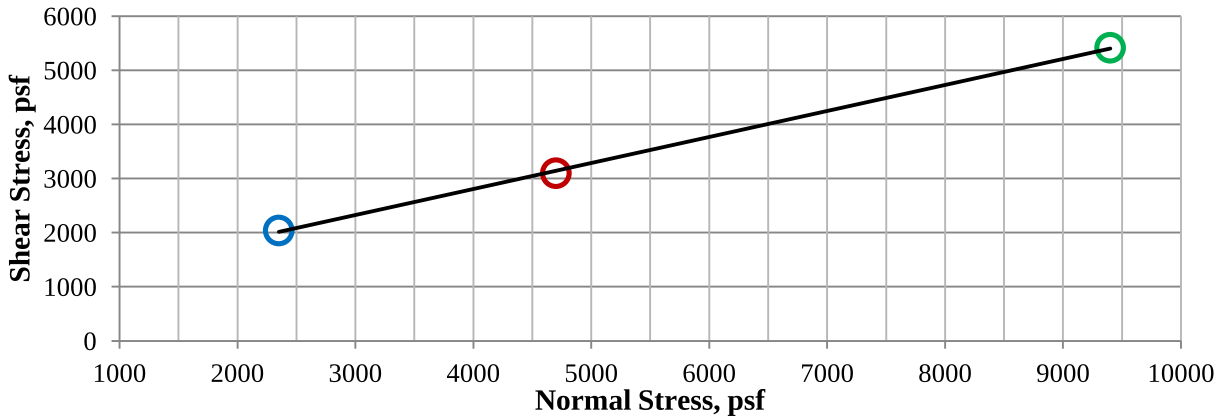
Sample Data:

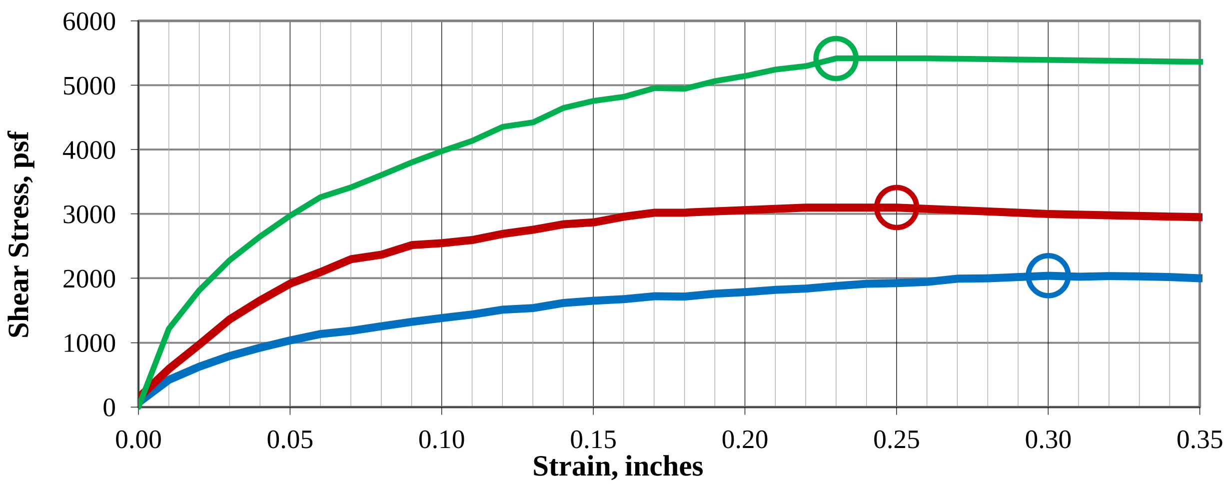
Boring: JG-6 U3 **Depth:** 39 - 40 ' **Type:** remolded

Description: Lean clay (CL) with silt, trace sand and salts, orangish brown, moist, soft

Normal Stress, psf	Initial Moisture,%	Final Moisture,%	Consol + Collapse,%	Final Wet Density, pcf	Final Dry Density, pcf	Max Shear Stress, psf	Failure Strain,%
2350	19.0	6.2	11.8	88.0	82.9	**2041**	**12.4**
4700	19.0	5.3	15.7	91.3	86.7	**3100**	**10.3**
9400	19.0	4.9	19.9	95.7	91.2	**5415**	**9.5**

Friction Angle, ϕ° **25.7** Cohesion, C, psf **883** Strain rate, %/hour **1.03**





Remarks: Friction angle and cohesion in practice are sensitive to several other material properties, and conditions, in the field and lab. No individual lab property of a material can substitute for overall best practices in geotechnical design, construction, and field testing by qualified professionals.

APPENDIX D
Loess Construction Article

Know More About Loess

By Edward D. Prost, Jr., P.E., M.ASCE and Joseph A. Waxse, P.E., M.ASCE

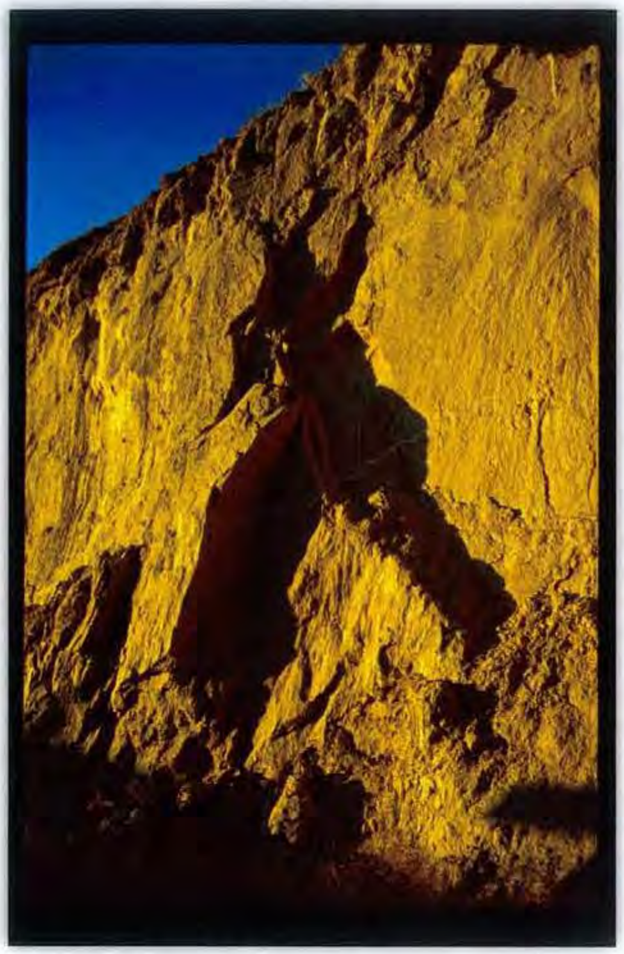


Figure 1. Near-vertical loess bluff face.

Encyclopedia Britannica defines loess as "an unstratified, geologically recent deposit of silty or loamy material that is usually buff or yellowish brown in colour and is chiefly deposited by the wind. Loess is a sedimentary deposit composed largely of silt-size grains that are loosely cemented by calcium carbonate. It is usually homogeneous and highly porous and is traversed by vertical capillaries that permit the sediment to fracture and form vertical bluffs. The word loess, with connotations of origin by wind-deposited accumulation, is of German origin and means 'loose.' It was first applied to Rhine River Valley loess about 1821." The original German pronunciation of loess is not directly translatable. The most common pronunciation in the U.S. is "luss," although some areas prefer "lo-ess" or "lerse," both of which are probably closer to the German vernacular.

Knowledgeable geotechnical engineers recognize that loess in the U.S. and Europe are Pleistocene deposits cemented by clay, rather than calcium carbonate, and refer to these wind-deposited materials as "Eolian" soils. According to the U.S. Geological Survey, loess deposits cover approximately 10 percent of the earth's surface. The major loess deposits that exist in the U.S., China, Russia, Europe, and Argentina are those most commonly cited in geotechnical literature.

Figure 2. Loess distribution in North America (courtesy of U.S. Geological Survey).

By convention, each loess stratum is named after the location where it was first officially described in a geologic type section. Each loess stratum also varies in its geotechnical properties due to differences in depositional climates, age, and prior wetting and weathering histories. The Peorian Loess, first described in Peoria, IL, is near the surface and is generally the most significant source of geotechnical problems in the Upper Midwest. The thickest, coarsest (lowest clay content and "plasticity"), and lowest density loess is typically located closest to its floodplain source. These are typically the most problematic soils.

Physical Characteristics

The original inter-particle clay cementation that holds the typical angular and elongated silt-sized particles in a loose, voided structure gives dry loess a stiff-to-hard "apparent" cohesion. However, wetting the soil weakens the clay bonds, causing a marked reduction in strength and increase in compressibility of the soil mass. The similarity of this wetting-induced collapse to the behavior of a wetted sugar cube gave rise to the local name "sugar clay" for Peorian loess soils.

Loess is relatively porous and the vertical capillaries (primarily due to vegetative root holes) markedly increase the soil's vertical permeability. Therefore, nominal surface water infiltration can occur downwards through the capillaries without necessarily causing a great enough increase in overall soil mass saturation to induce collapse. It is thought that where a capillary intersects a void or becomes somewhat larger in diameter, the associated decrease in surface tension initiates precipitation of dissolved calcium carbonate from the infiltrating pore water. This is believed to be the source of the characteristic grape- to grapefruit-sized nodules often found in loess. These oddly-shaped nodules are called Loess Kindchen (loess dolls) or other local names such as "Devil's Eggs." Some of them rattle when shaken and explode impressively when thrown against a hard surface.

Figure 3. Loess "kindchen."

Loess is found in nature at a variety of densities, moisture contents, and grain sizes, and with different degrees of cementation. Loess strata deposited from successive glacial periods are typically delineated by a weathered topsoil layer (paleosol) that developed at the ground surface during the interglacial period. The paleosol may have a lower vertical permeability due to increased organic and clay contents and collapse of the original loess structure during weathering. This characteristic can cause the layer to act as an aquitard and result in slowed infiltration and saturation of the base of the overlying loess stratum.

Figure 4. Building damage due to loess collapse.

Collapse Potential

Paleosol formation processes of wetting cycles or erosion and redeposition (alluvium or colluvium) modify the behavior of loess. Wetting generally allows the loose cementation to disintegrate and results in tremendous strength loss and soil structure collapse. These soils behave similarly to an alluvial soil with little or no over-consolidation. If the loess is exposed to cycles of wetting and drying, the soils generally densify, as is the case with most soils, lose their natural loess structure, and behave similarly to over-consolidated alluvial soils. Soils of this nature may be present at various depths within the loess formation, interspersed with zones of loess soil that have not experienced as much variation in moisture, and exist at low densities, with a structure similar to that present near the time of placement. These soils require special consideration that is unique to regions where deep or thick layers of low plasticity loess are present.

The relative collapse potential of loess is generally inversely proportional to the soil's in-situ density and clay content - the lower the density and clay content, the greater the potential for collapse. Density must be evaluated by careful exploratory methods, due to the potential for incidental sample compression. The Standard Penetration Test yields misleading data in dry loess and should not be used to try to assess collapse potential.

Collapse of loess soils due purely to increased loading is rare, as the bearing pressures of foundations supported on dry loess are generally limited to pressures much below the bearing capacity of the in-situ strength of the soil. Collapse/settlement of loess is predominantly related to wetting of the soils, which breaks down the weak bonding created by the clay or mineral paste surrounding the silt and sand particles.

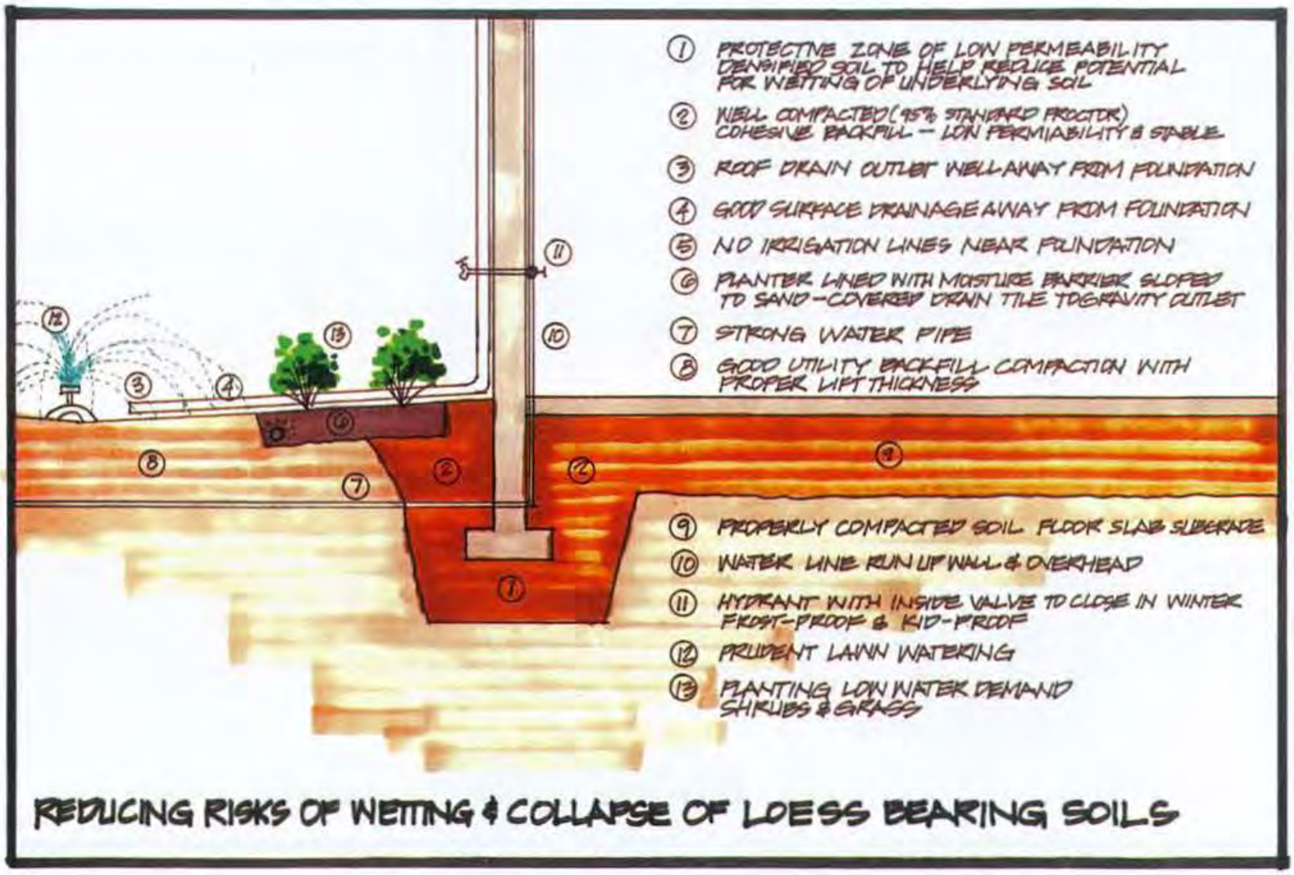


Figure 5. Common measures used to reduce wetting risks.

However, settlement and collapse are much more dramatic where foundation loads are applied.

Construction-Related Problems

Moisture changes occur due to several reasons related to construction, which may include:

- altered surface drainage patterns,
- altered subsurface drainage patterns,
- leaking utilities,
- irrigation,
- HVAC condensate and gutter downspout discharges, and
- reduced transpiration.

One would think that surface drainage should not be an issue in a constructed environment; however, this is often the primary mechanism where the soils are not properly compacted and settle adjacent to foundation walls, especially where a basement is present. The resulting ponding and infiltration into the loose backfill allows moisture to enter from natural sources as well as irrigation. Another mechanism that is not often considered is the effect of major grading of residential subdivisions or other developments where natural drainage ways are filled, thus altering the natural subsurface drainage patterns.

Leaking of utilities is an obvious potential source of moisture which must be considered. However, design for every potential possibility of utility leakage may not be practical. Prudent design of utilities to resist leakage or breakage under moderate differential movement should always be considered where the consequences of wetting can be severe. Septic system drain fields should be situated to avoid affecting the proposed construction as well as any neighboring construction or slopes. Providing a minimum 5-10 percent surface slope for at least 10 ft out from foundations is often cited as a prudent protective measure.

Irrigation of lawns and other vegetation can be a significant factor in collapse/settlement of structures supported on collapsible loess, especially where combined with poor surface drainage. Careless discharge of gutter downspouts and air-conditioning condensate near foundations are common culprits of localized settlement damage. Removal of trees and green spaces to facilitate construction removes a significant

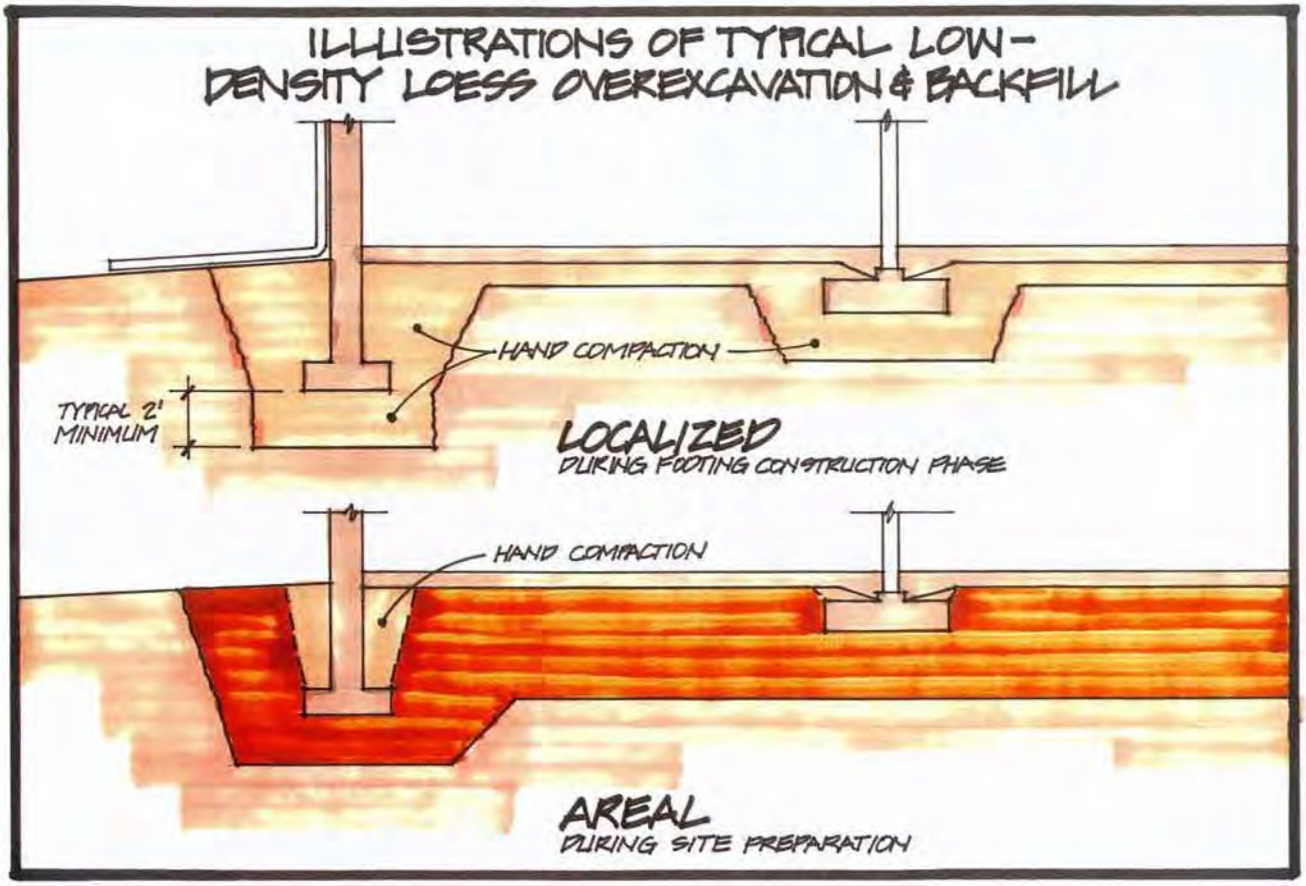


Figure 6. Cross-sections illustrating the partial excavation concept.

control on the moisture content of loess. Rising water tables as transpiration rates fall may cause wetting and subsequent collapse of otherwise stable loess.

Treatment Alternatives

A variety of measures have been attempted or proposed to remediate the effects of collapsible loess soils on foundations. These have included:

- partial or complete removal and replacement of the collapsible loess soil,
- transferring loads through the metastable soil to stable or protected underlying soils,
- barriers to minimize the potential for wetting of the soil,
- compaction grouting,
- injection of chemical stabilizers,
- prewetting (usually in combination with preloading),
- dynamic compaction, and
- deep blasting.

Partial excavation generally provides an acceptable level of risk reduction and cost effectiveness, especially for light-to-moderately loaded structures. Common practice is the removal of the loess soils to a depth of at least 2-3 ft below the foundations and floor slabs of the proposed structure.

A more reliable method of reducing the risk posed by the collapsible soils is to derive support of the structure below the depth of the collapsible soils, or below the depth of anticipated wetting potential if the collapsible soils extend to a great depth. This solution is often impractical for light structures of lesser monetary value, but can be a practical alternative for structures with substantial loading and/or monetary worth. Driven or augered pile or drilled shafts are common solutions for these types of structures. Intermediate foundations such as compacted aggregate columns may also be suitable, but the potential for creating additional seepage paths must be properly understood and addressed.

Partial excavation and recompaction of the loess soils helps retard moisture infiltration to the underlying collapsible loess, however, there are times where these measures are not considered adequate to protect the underlying soils. This is often the case for wet process buildings or where the facility itself retains water or other fluids. Secondary containment in the form of a sloped impermeable membrane with an overlying granular drainage system is often included in these circumstances. Compaction grouting or adding chemical stabilizers are corrective measures that are more often used as a remedial measure after foundation movement has occurred, because this is usually more costly than an excavation or deep foundation alternative.

Other measures, such as prewetting with a surcharge, have a distinct disadvantage in most loess soils due to substantial time delays to complete the saturation process, a need for subsequent exploration to evaluate the effects, and significant loss of soil strength due to wetting that result in relatively poor support for shallow foundations. Deep blasting and dynamic compaction in collapsible loess soils may have particular applications where the collapse susceptible soils extend to great depth and the cost is significantly less than that of supporting the structure on deep foundations.

The Importance of Knowing Loess

Experience has shown time and again that one must be a pessimist when it comes to evaluating the risk of loess bearing soils becoming exposed to some future risk of wetting. The future owners/operators of facilities seldom read geotechnical reports and should not be assumed to understand or appreciate the risks or consequences of the collapsible loess beneath them. Geotechnical engineers should assume that prudent measures may not be taken to protect against wetting sources, or that an unanticipated source may "spring" up. One need consider the full potential for foundation distress when developing recommendations and ever-important liability/loss prevention language in reports for sites underlain by collapsible loess.

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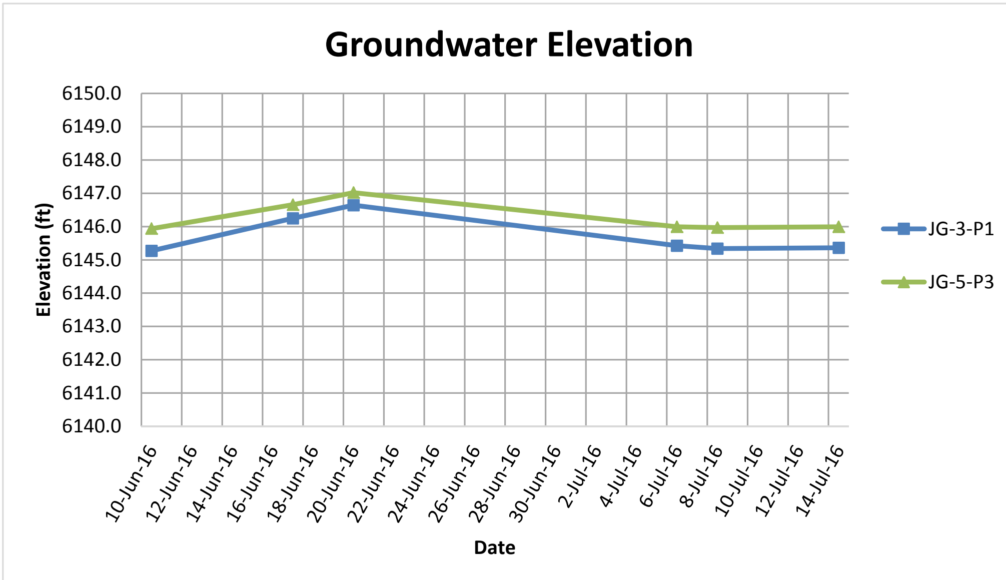
***Geo-Strata* is interested in hearing from you. Please send your comments on this article to *geo-strata@asce.org*.**

APPENDIX E
Groundwater Data and Plot

Piezometer	Borehole Elevation (ft)	Piezometer Depth, bgs (ft)	Piezometer Elevation (ft)	GW Elevation 10-Jun-16	17-Jun-16	20-Jun-16	6-Jul-16	8-Jul-16	14-Jul-16
JG-3-P1	6183.8	44	6139.8	6145.3	6146.2	6146.6	6145.4	6145.3	6145.4
JG-5-P2	6220.1	32	6188.1	DRY	DRY	DRY	DRY	DRY	DRY
JG-5-P3	6220.1	69	6151.1	6145.9	6146.7	6147.0	6146.0	6146.0	6146.0



APPENDIX F
USGS Seismic Design Maps
Summary and Detailed Reports

Design Maps Summary Report

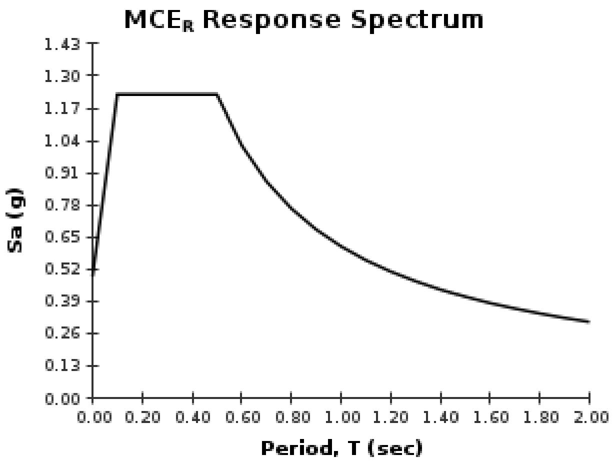
User–Specified Input

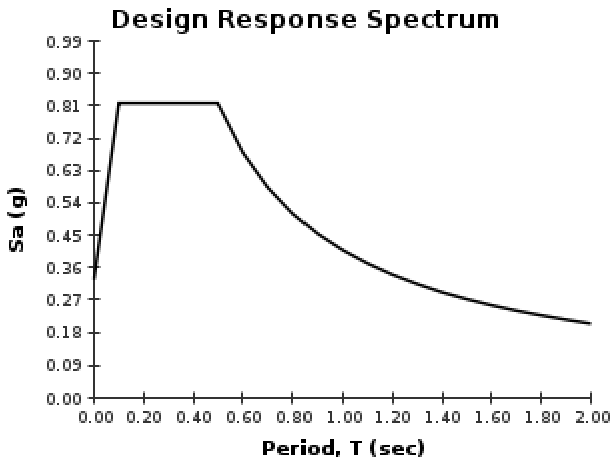
Report Title	West View Townhomes Thu July 14, 2016 02:18:29 UTC
Building Code Reference Document	2012/2015 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	43.4761°N, 110.7901°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

USGS–Provided Output

S_S =	1.203 g	S_{MS} =	1.225 g	S_{DS} =	0.817 g
S_1 =	0.368 g	S_{M1} =	0.612 g	S_{D1} =	0.408 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.





USGS Design Maps Detailed Report

2012/2015 International Building Code (43.4761°N, 110.7901°W)

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

Section 1613.3.1 — Mapped acceleration parameters

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

From Figure 1613.3.1(1) [1]	S_S = 1.203 g
From Figure 1613.3.1(2) [2]	S_1 = 0.368 g

Section 1613.3.2 — Site class definitions

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

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

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

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

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

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a

Site Class	Mapped Spectral Response Acceleration at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
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 = 1.203 g, F_a = 1.019

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 = 0.368 g, F_v = 1.665

Equation (16-37):	$S_{MS} = F_aS_S = 1.019 \times 1.203 = 1.225$ g
Equation (16-38):	$S_{M1} = F_vS_1 = 1.665 \times 0.368 = 0.612$ g

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39):	$S_{DS} = ⅔\ S_{MS} = ⅔ \times 1.225 = 0.817$ g
Equation (16-40):	$S_{D1} = ⅔\ S_{M1} = ⅔ \times 0.612 = 0.408$ 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} = 0.817 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} = 0.408 g, Seismic Design Category = D

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

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

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

References

1. *Figure 1613.3.1(1)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(1).pdf
2. *Figure 1613.3.1(2)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(2).pdf

APPENDIX G
Concrete Construction Publications

Controlling curling and cracking in floors to receive coverings

Do you worry about excessive cracking or curling in concrete floor slabs placed directly on a vapor retarder? Here are some hints on using reinforcing steel to minimize these defects and avoid floor-covering failures.

By Jerry A. Holland and Wayne Walker

Because of an increasing number of moisture-related floor-covering failures in the past several years, some designers now recommend eliminating the granular blotter layer that's often used between the concrete and the vapor retarder or vapor barrier. Though a blotter layer offers several advantages, it can hold water from many possible sources and cause problems if the floor will receive moisture-sensitive coverings such as sheet vinyl, rubber, wood or similar materials (see reference).

Many designers, however, are reluctant to place concrete directly on a vapor retarder because they fear the floor slab will curl or crack excessively. These defects also can cause floor-covering failures that, in some cases, require remedial work after the building is in service. However, with the correct positioning and amount of reinforcing steel, both curling and cracking can be controlled.

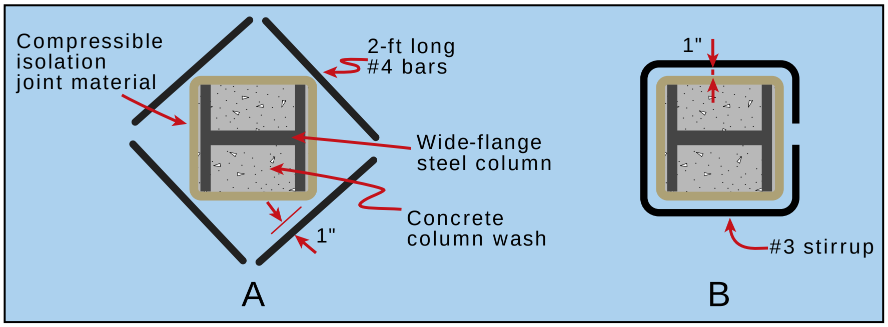
Rebar in concrete slabs placed directly on a vapor retarder help to control slab curling and cracking. Use supported deformed bars no smaller than #4, and space the bars far enough apart so workers can step between them.

Positioning is key

Cracks in a slab-on-grade floor surface are wider at the top than at the bottom. For the best crack control, then, you want the reinforcing steel to be as close to the surface as possible. And you must be able to control the location of the steel so it doesn't change during floor construction. Because of this, I prefer to use supported deformed bars no smaller than #4 instead of light-gauge mesh. Smaller-diameter bars are too limber, requiring too many bar supports, and light-gauge mesh is difficult to keep in the correct location.

For a 5-inch-thick floor slab, I prefer to use #4 bars near the top with 1 inch of clear cover, or #5 bars with 1½ inches of clear cover. For #5 bars, greater cover depth is needed to control plastic settlement cracking over the bar.

Typically, I specify #4 bars spaced 18 inches on center both ways. This amount of steel holds crack faces together tightly enough for nonrigid floor coverings by maintaining aggregate interlock and significantly reducing slab curling. In some instances, closer spacing or larger-diameter bars may be needed. Constructability becomes an issue when bar spacing is so close that workers



Eliminate the normal isolation-joint box outs at wide-flange steel columns by wrapping the column with compressible material and using 2-foot lengths of #4 bars (A) to control cracking at the reentrant corners. To speed up steel placement at the columns, have the rebar supplier fabricate continuous #3 stirrups that workers can easily bend open to fit around the column (B). In either case, the steel should be positioned with a top-and-side clear cover of 1 inch.

can't step into openings between bars. Then larger-diameter bars may be the better choice.

Eliminate joints

Because the reinforcing steel limits crack width, I prefer to eliminate contraction joints and the traditional diamond-shaped isolation joints at columns when floors will receive a covering. I suggest wrapping wide-flange steel columns for the full floor depth with ⅛- to ¼-inch-thick compressible isolation-joint material. For floors receiving coverings that won't tolerate wide cracks, such as ceramic tile, I also suggest placing four 2-foot-long #4 bars near the floor surface, with a top-and-side clear cover of 1 inch to control reentrant-corner cracking (Fig. A). As an alternative, the rebar supplier can fabricate #3 bars as a continuous stirrup that can easily be bent open so the ironworker can fit it around the column (Fig. B). This speeds placement of the steel when there are many columns to be treated. The stirrups also should have a 1-inch top-and-side clear cover.

Carpeting or other floor coverings can tolerate larger crack widths in the concrete subfloor without noticeable distress. When these coverings are used, crack-control measures at columns may not be needed. Simply wrap the columns to isolate them from the slab.

Construction considerations

Some designers use an upper and lower layer of reinforcing steel in the slab to control cracking at both the top and bottom. However, bottom-crack width doesn't affect floor-covering performance. And some of the advantages of these double layers of rebar are offset by placement difficulties; workers spreading the concrete have trouble stepping around the rebar and may displace it during concrete placement.

If the concrete is tailgated or struck off by a self-propelled laser-guided screed, ironworkers can lay out a single layer of steel on the vapor retarder and chair it up as concrete placement and strike-off proceeds. To prevent damage to the vapor retarder, workers can lay down thin sheets of plywood or several folds of plastic sheeting beneath the tires of the concrete truck or the screed. These materials are then moved back as the pour proceeds. The same procedure will help prevent damage to the vapor retarder if motorized buggies are used to place the concrete.

If the concrete is placed by pump or conveyor, all the steel can be chaired up before the pour begins, provided there's enough space between the rebar for workers' feet. If control of crack width requires rebar spacings of a foot or less both ways, I sometimes require placement of a heavy-gauge welded-wire fabric (4x4-inch spacing of 4-gauge wire) on top of the bars. Workers can easily walk on this mesh without sinking into the concrete or twisting their ankles. The closely spaced mesh wires improve crack control, and the material cost is about the same because you can reduce the rebar diameter and maintain about the same steel cross-sectional area.

Weighing the costs

Although controlling curling and cracking by using rebar in the way I've described increases project costs by requiring more than the normal amount of steel, part of this cost increase is offset by savings in other areas. You eliminate the costs associated with overexcavation to accommodate the blotter-layer thickness and for purchasing, placing and compacting the granular material used for the layer. You also save money because workers don't have to cut contraction joints and fill them with a sealant. Nor do they have to form and strip column box outs and place the in-fill concrete later.

Use of a blotter layer is still a viable alternative for controlling curling and cracking. But if the floor will receive a moisture-sensitive floor covering and the blotter layer picks up excessive moisture before, during or after floor construction, a flooring failure is likely. The cost of correcting the failure almost always will be much higher than the cost of using more reinforcing steel.

Jerry A. Holland is structural engineering consultant and Wayne Walker is senior structural engineer for Lockwood Greene Engineers Inc., Atlanta. Holland has more than 30 years of experience and Walker has 20 years of experience designing and troubleshooting concrete slabs on grade.

Reference

Bruce A. Suprenant and Ward R. Malisch, "Where to Locate the Vapor Retarder," *Concrete Construction,* May 1998, pp. 427-433.

Where to place the vapor retarder

For slabs on grade, should the vapor retarder be located under a granular layer or directly under the concrete? Here are the pros and cons of each location.

BY BRUCE A. SUPRENANT AND WARD R. MALISCH

In the real estate industry, location is everything. The importance of location also applies to a hotly debated topic in the concrete industry—where to place the vapor retarder (or vapor barrier) for slabs on grade. Some specifiers require concrete to be placed directly on the vapor retarder, and others require placement of a granular blotter layer between the concrete and the vapor retarder. Advocates of each option argue that their preference results in a better concrete slab.

Like all engineering decisions, the location of a vapor retarder often is a compromise between minimizing water-vapor movement through the slab and providing the desired short- and long-term concrete properties. However, specifiers must consider the benefits and liabilities of the choice they make.

The case for a granular layer

Finishers prefer concrete placed on a granular base because the base absorbs mix water, shortens the bleeding period and allows floating to start earlier. Australian researchers noted that 4½-inch-slump concrete placed on a granular base lost its bleedwater sheen about two hours faster than the same concrete placed directly on a vapor barrier (Ref. 1).

Base conditions also affect concrete stiffening. In tests performed by The Aberdeen Group, 2½-inch-slump concrete was used for two 4x4-foot, 4-inch-thick slabs. One slab was placed directly on a vapor retarder and the other on a crushed-stone base. Technicians periodically set a steel-shot-filled rubber boot weighing 75 pounds on the surface and measured the footprint indentation (Fig. 1). Concrete on the stone base had stiffened enough after 90 minutes to allow a ¼-inch footprint

Figure 1. Concrete is generally considered to be ready for floating when finishers leave a ¼-inch-deep footprint in the surface. Using a boot filled with steel shot (inset) to produce footprints, we found that 2½-inch-slump concrete placed on a stone base was ready for floating about 45 minutes earlier than the same concrete placed directly on a vapor retarder.

indentation, an indication that floating could begin. Concrete placed directly on the vapor retarder required 45 more minutes of stiffening time before it was ready for floating.

Specifiers who require a granular blotter layer cite additional benefits, saying there is less chance of :

- Puncturing the vapor retarder
- Surface blistering or delaminations caused by an extended bleeding period
- Settlement cracking over reinforcing steel
- Slab curling during drying
- Cracking caused by plastic or drying shrinkage

Many specifiers recommend a 3- or 4-inch-thick layer of trimmable, compactible, self-draining granular fill for the blotter layer. Although concrete sand is sometimes recommended, it doesn't provide a stable working platform. Concrete placement and workers walking on the sand can disturb the surface enough to cause irregular floor thickness and create sand lenses in the concrete.

The case for placing concrete on a vapor retarder

Floor-covering contractors prefer to install their products on concrete slabs that are placed directly on a vapor retarder. If the vapor retarder effectively reduces moisture inflow from external sources, only water in the concrete pores must exit the slab. They believe the often-required vapor-emission rate of 3 pounds/1,000 square feet/24 hours is achieved faster under these conditions. They also believe the uncovered vapor retarder acts as a slip sheet, reducing slab restraint and thus reducing random cracking.

Placing concrete directly on a vapor retarder also eliminates a potential water reservoir that's created when using a blotter layer. Because more subgrade soil must be removed to accommodate the additional 3- to 4-inch-thick blotter layer, the layer is more likely to be placed below finished-grade level, thus increasing the chance of its holding water.

Specifiers who require concrete to be placed directly on the vapor retarder cite these additional advantages:

- Reduced costs because of less excavation and no need for additional granular material
- Better curing of the slab bottom, since the vapor retarder minimizes moisture loss
- Less chance of floor moisture problems caused by water being trapped in the granular layer
- Less radon-gas infiltration

These specifiers recommend using a low water-cement-ratio concrete and water-reducing admixtures to reduce bleeding, shrinkage and curling of concrete placed directly on the vapor retarder. They believe the higher-quality concrete and better curing reduces cracking and produces a better floor.

Table 1. Amount of water in granular layer per 1,000 square feet of floor*

Layer thickness	Water absorbed	Water in voids	Total water
2 in.	220 lbs	2,080 lbs	2,300 lbs
3 in.	330 lbs	3,120 lbs	3,450 lbs
4 in.	440 lbs	4,160 lbs	4,600 lbs

*Well-graded, compactible granular-base material with assumed density of 130 pounds per cubic foot, 1% absorption capacity and 20% voids. A 7% to 8% moisture content would normally be needed to achieve the compaction density typically required.

Granular layer as a water reservoir

When a low-permeability floor covering will be installed on a concrete floor, special care is needed during construction to control moisture content of the subgrade, subbase or granular layer (if used over the vapor retarder). It's best to place the floor after the building is enclosed and the roof is watertight. On many projects, however, this isn't possible, and the granular layer can become a water reservoir.

Water sources and access points. To provide unrestricted floor access for construction activities such as tilt-up panel forming and casting, columns sometimes aren't erected and column blockouts aren't filled until months after floor placement. But rainwater can enter column blockouts that are left open. It can also penetrate joints and cracks, utility penetrations or open closure strips, and increase the moisture content of the subgrade, capillary break or granular layer.

Excessive sprinkling of a granular layer before concrete placement can create a moisture reservoir that will delay drying of the concrete floor. ACI 302.1R-96 (Ref. 2) recommends that the base be dry at the time of concreting unless severe drying conditions exist.

Wet-curing methods such as ponding or continuous sprinkling allow water to enter joints, cracks and other openings, again contributing to a higher than necessary moisture content beneath the floor slab.

Water from construction operations on a newly placed slab also can increase the granular-layer moisture content by entering joints, cracks or slab openings. Such operations include joint sawing, abrasive wet blasting or wet grinding, which may be needed to achieve a flatter floor profile. Sometimes power washing is used to clean debris or other contaminants from the floor.

Most slabs are constructed using a strip-placement sequence that leaves the granular layer exposed to rainwater in uncompleted portions of the slab. Rollings (Ref. 3) determined that a tile-floor failure was caused by rainwater accumulating in a 3-inch-thick sand layer placed between a 5-inch-thick concrete slab and a polyethylene vapor retarder. One portion of the slab had been left uncompleted for an extended period, exposing the sand layer to prolonged rain and turning it into a reservoir of trapped water.

Water capacity of the granular layer. Table 1 shows the maximum amount of water that can be held in a layer of well-graded, compactible granular-base-course material of various thicknesses. If the floor concrete contained 250 pounds of mix water per cubic yard, 1,000 square feet of 6-inch-thick floor would contain 4,630 pounds of mix water. As shown in Table 1, a 4-inch-thick granular layer under the floor can contain about the same amount of water. And if sand or other high-void-content granular materials are used, the water capacity is much higher.

If the 250 pounds of mix water are used in concrete with a water-cement ratio of 0.50, about 100 pounds of the water will be free water that must evaporate as the floor dries (Ref. 4). Thus a 6-inch-thick, 1,000-square-foot floor slab would hold 1,850 pounds of free (evaporable) water.

Based on Brewer's work (Ref. 5), it would take about 82 days, or roughly three months, for enough free water to evaporate and produce a water-vapor emission rate of 3 lbs/1,000 sf/24 hours. A saturated 2-inch-thick granular layer would need to lose as much water as the concrete. And the water in the layer must move through the concrete. Thus it's likely that a 2-inch-thick saturated, well-graded granular layer could double the time required for the slab vapor-emission rate to reach 3 lbs/1,000 sf/ 24 hrs. It could even prevent the slab from ever reaching that emission rate.

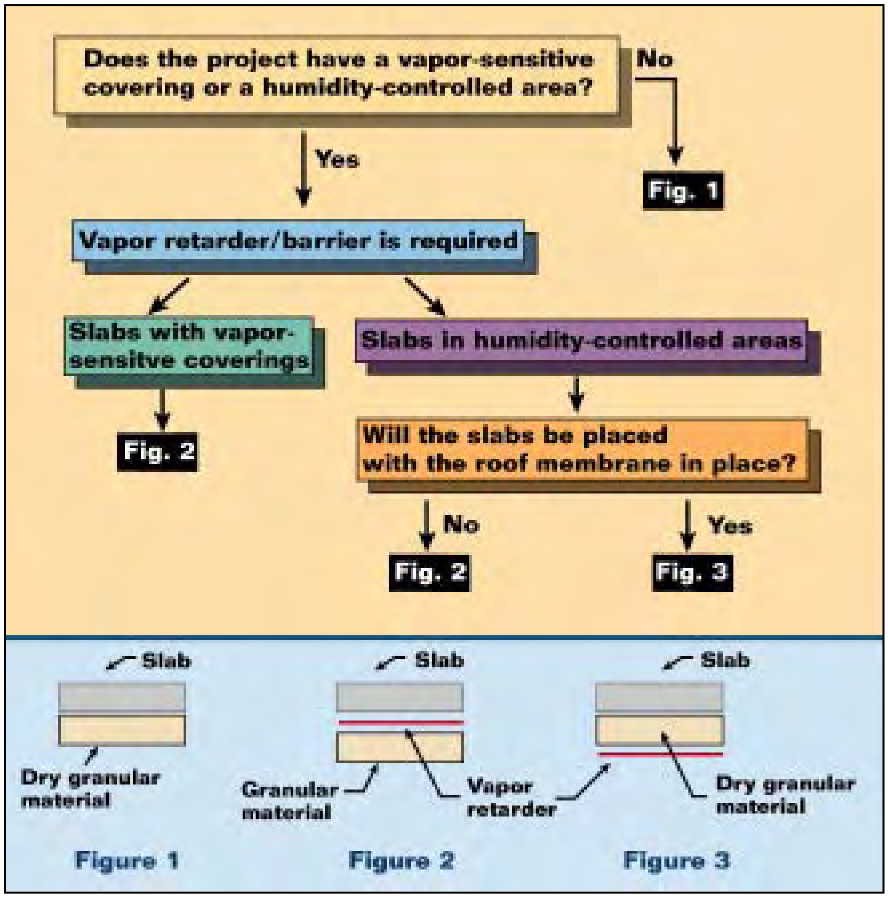


Figure 2. This flow chart helps designers decide if a vapor retarder or barrier is needed and where it should be placed.

Weighing the alternatives

Consulting engineers Jerry Holland and Wayne Walker, Lockwood-Greene Engineers, Atlanta, have developed a flow chart to help designers decide if a vapor retarder is required and, if so, where to place it (Fig. 2).

The chart gives designers the following three options based on the floor's in-service environment and the presence or absence of a vapor-sensitive floor covering:

- Use no vapor retarder
- Use a vapor retarder directly below the slab
- Sandwich a granular layer between the vapor retarder and the slab

ACI Committee 360 is considering inclusion of the flow chart in ACI 360R, *Design of Slabs on Grade*. Because curling is a major concern when concrete is placed directly on the vapor retarder or barrier, notes in the flow chart will provide suggested design options for minimizing curling effects.

Establishing responsibility for moisture-related floor problems

Consider the following scenario based on a concrete subcontractor's actual experience. The subcontractor places and finishes a concrete floor. Flatness and levelness measurements show specification compliance, and test reports indicate the 28-day compressive strength is acceptable. He leaves the job and submits his bill.

Two months later, he's called back by the general contractor. Rainwater has penetrated the slab, which has curled. The floor-covering contractor is concerned about high water-vapor emission rates, and the general contractor worries that the required slab drying time will delay project completion.

The concrete subcontractor is being held responsible for:

- Curling, even though floor flatness met specifications when measured within 72 hours after concrete placement as required by ACI 117-90, *Standard Specification for Tolerances for Concrete Construction and Materials*
- Protecting the slab from external moisture, even though he has completed all the concrete work and is no longer at the site
- Water-vapor emissions from the slab, even though the general contractor followed specification requirements by placing a granular layer over a vapor retarder
- Delays in completion of the project due to these problems

Sound familiar? On this project, the floor contractor returned at his own expense to grind the slabs and minimize curl. Luckily, he was able to convince the design team that the other issues were not his responsibility.

All of these issues should be resolved with the general contractor, design team and owner *before* the slab is placed. Concrete subcontractors should be held responsible for flatness and levelness within the time frame designated by ACI tolerance standards, but not longer. General contractors should be responsible for protecting the slab from external moisture. Only they can coordinate and direct the services of the roofer, excavator and other subcontractors who can help to minimize moisture infiltration. And, unlike the concrete subcontractor, the general is on the project from start to finish.

Concrete subcontractors need to resolve these issues at prepour planning meetings. If they don't, they had better be prepared for the phone call telling them they're responsible for fixing problems caused by rainwater infiltration. To avoid that call, add the items discussed here to your prepour conference checklist.

Editor's note

Discussions, pro and con, for differing vapor-retarder installation options are also given in ASTM E 1643, *Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill under Concrete Slabs.*

References

1. T. Anderson and H. Roper, "Influence of an Impervious Membrane Beneath Concrete Slabs on Grade," Concrete Symposium, Brisbane, Australia, 1977, p. 51.

2. ACI 302.1R-96, *Guide for Concrete Floor and Slab Construction*, American Concrete Institute, Farmington Hills, Mich., February 1997.

3. Raymond S. Rollings, "Retail-Grocery — Floor Failure," *Journal of the Performance of Constructed Facilities,* American Society of Civil Engineers, Reston, Va., May 1995.

4. Herman G. Protze, III, "Construction of Concrete Slabs-On-Grade: Moisture Emission Problems," *Solving Moisture-Related Problems with Slabs-On-Grade,* Seminar 24-63, World of Concrete 1997.

5. Bruce A. Suprenant, "Moisture Movement through Concrete Slabs," *Concrete Construction*, November 1997, pp. 879-885.

PUBLICATION #C980427

Don’t use loose sand under concrete slabs

A thin, loose sand layer reduces subgrade support, which can lead to increased slab cracking and poor joint performance

BY BRUCE A. SUPRENANT AND WARD R. MALISCH

Over the past five years, we’ve received phone calls from contractors who had built floors under which the specifier required a thin sand layer, with no compaction requirement for the sand. The contractors had been called back to repair cracks and joints 6 to 24 months after the slab was placed. The cracks didn’t appear to be caused by drying shrinkage, and the joints were showing more than normal deterioration. The problems occurred primarily in slabs subjected to forklift traffic.

The contractors were being held responsible for the repair costs, and they asked, “Is it possible that the sand layer reduces subgrade or subbase support, causing cracking and poor joint performance, especially under repeated loading such as forklift traffic?”

Figure 1. A technician applies load to a compacted soil specimen in a CBR mold. Specimens were loaded with and without sand layers to determine the effect of differing sand-layer thicknesses.

Table 1 Soil sample properties

Soil sample	Dry density (pcf)/moisture content (%) No sand	1-in. sand	2-in. sand	Compaction test (standard Proctor) Density/moisture	Soil classification
1A	100.1/19.2	99.8/19.6	100.6/19.0	104.9 pcf/19.5%	SC: A-6(5)
1B	100.1/19.7	99.7/19.8	99.8/19.6		
2A	109.5/14.5	109.5/14.5	109.8/14.4	115.0 pcf/14.7%	SC: A-6(3)
2B	109.3/14.6	109.5/14.6	109.4/14.7		
3A	125.4/8.9	125.1/9.1	125.7/9.1	131.9 pcf/9.1%	SC: A-2-4(0)
3B	125.2/9.0	125.1/9.2	125.3/9.0		

The soil is a sand with silty clay and a trace of gravel. The SC is a sand-plastic fines soil classification based on the Unified Soil Classification System. The A-soil classification system is based on the AASHTO soil classification system.

We developed a testing program to gather data that could help answer this question.

Testing subgrade support

To assess the effect of a thin, loose sand layer on subgrade support, we performed duplicate California Bearing Ratio tests (see "What's a CBR Test") using three soil samples with varying dry densities. Each test specimen was tested with no sand, a 1-inch sand layer and a 2-inch sand layer. In addition, we placed 1- and 2-inch sand layers over a steel base and tested that combination to show how the sand would affect subgrade support over a very stiff base.

To start the test, a technician placed the soil into a 6-inch-diameter cylinder mold and compacted it. After compaction, he removed the top extension collar and trimmed the soil to a 4½-inch height. He then inverted the mold and added a 10-pound surcharge weight to the top surface. Consisting of steel discs with holes in the center to accom-

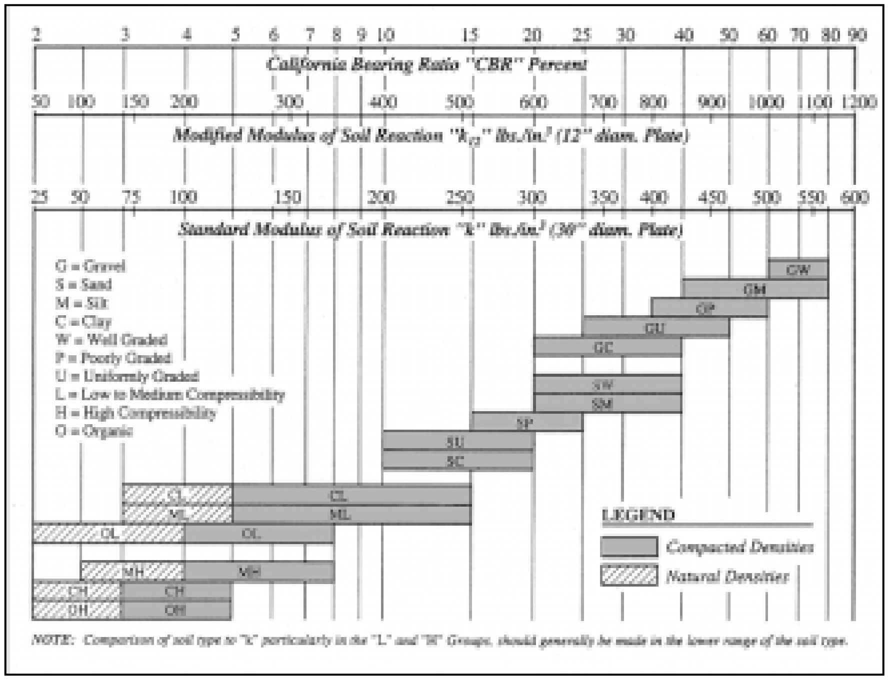


Figure 2. Interrelationships of CBR, k-values and soil classification (from Ref. 2).

Table 2 Effect of a sand layer on measured CBR

Soil sample	No sand	CBR value, % 1-in. sand	2-in. sand
1A	4.0	2.6	1.0
1B	4.0	3.1	2.1
Average	4.0	2.9	1.6
% of no-sand value	100	73	40
2A	8.1	6.3	4.9
2B	8.0	5.6	3.9
Average	8.1	6.0	4.4
% of no-sand value	100	74	54
3A	11.4	4.6	2.5
3B	11.5	4.8	2.6
Average	11.5	4.7	2.6
% of no-sand value	100	41	23
Steel base - A	100*	5.2	2.5
Steel base - B	100	4.9	2.6
Average	100	5.1	2.6
% of no-sand value	100	5.1	2.6

** Not tested; maximum CBR is 100.*

What's a CBR test?

The California Bearing Ratio test, described in ASTM D 1883 (Ref. 1), is a penetration test commonly used to evaluate the potential strength of subgrade, subbase and base course material. To perform the test, a technician uses a cylindrical piston with a 3-square-inch cross section to penetrate the soil at a rate of 0.05 inch per minute. At each 0.1 inch penetration up to 0.5 inch, the technician records the stress needed to push the piston into the soil. The CBR value is the ratio of this stress at different penetration levels to the bearing value of a standard crushed rock. In most cases, CBR decreases as the penetration increases, so the ratio at 0.1-inch penetration is used as the recorded CBR value. Sometimes designers use this value to choose an appropriate slab thickness for anticipated loadings.

modate the piston, the surcharge weight is nearly equivalent to that of a 4½-inch-thick concrete slab. At this point in the test, it's possible to include a four-day wet soaking period. However, we omitted this step since we weren't interested in the CBR of a wet subgrade.

The soil specimen contained in the mold and loaded by the surcharge weights was placed in a testing machine (Fig. 1) that applied load to the piston. A technician measured load and piston penetration distances and used the resulting stress-vs.-penetration curve to compute the CBR values.

To measure the sand-layer effect, the technician placed loose concrete sand in the mold to completely and uniformly cover the compacted subgrade to a depth of 1 or 2 inches. For the steel base used to simulate a stiff base, the technician placed loose sand over the base and added the surcharge weights before applying load to the piston.

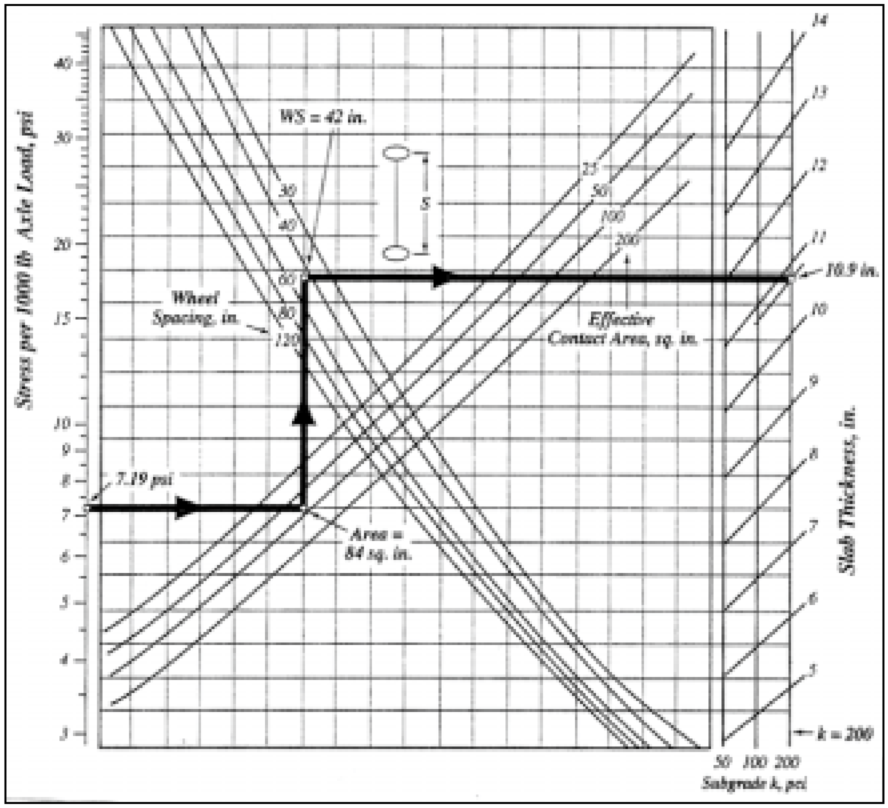


Figure 3. The example in this chart shows that decreasing the k-value from 200 to 50 increases the required slab thickness about an inch. For lighter loadings that yield a thinner slab, the same k-value reduction would still increase thickness about an inch.

Table 3 Effect of sand layer on k-values*

Soil sample	No sand	1-in. sand	2-in. sand
1A	100	50	10**
1B	100	75	25
Average	100	63	18**
% of no-sand value	100	63	18
2A	175	145	125
2B	175	135	100
Average	175	140	113
% of no-sand value	100	80	64
3A	210	125	50
3B	210	125	50
Average	210	125	50
% of no-sand value	100	60	24
Steel base - A	650**	125	50
Steel base - B	650	125	50
Average	650	125	50
% of no-sand value	100	19	8

**The k-value is a modulus of soil reaction in lbs/in.^{3} for a 30-inch-diameter plate and was estimated using the CBR values shown in Table 2.*

*** Off the chart. In Figure 2, minimum k-value is 25 and maximum is 600. Since a CBR of 100 is possible, a k-value of 650 was estimated.*

The density and moisture content of the compacted specimens also were determined. A comparison of standard Proctor dry-density values shown in Table 1 with the dry densities of the soil samples, also given in the table, shows that all the CBR specimens reached about 95% compaction. Great care was exercised in making sure that the compacted density for a set of specimens was essentially the same. Thus, any measured changes in CBR value would be the result of the presence of a sand layer and not a change in specimen density.

For all the soil samples tested, CBR values decreased dramatically when a thin layer of loose sand was placed over the compacted sample

(Table 2). The decrease was especially large for the sand layer placed over the steel base. For soil sample No. 1 (lowest density), the 1-inch and 2-inch sand layers decreased CBR values to 73% and 40% of the original values, respectively. For sample No. 3 (highest density), the CBR decreases were to 41% and 23% of the original values.

The CBR values for sand layers placed over a steel base provided an interesting comparison. Percentage loss in CBR was very high, but the raw CBR values appear to show that the highest-density soil provided almost as stiff a base as the steel when a sand layer was added. The CBR values for the lowest-density soil with a sand layer are lower, which is understandable given the weaker subgrade support. The CBR values for soil sample No. 2 don't follow this pattern, and we don't know whether this was the result of soil or sand variability or the variability of the test itself. The steel-base values do seem to indicate that if a designer uses a sand layer, the maximum CBR values he could reasonably expect to attain are about 5 and 2.5 for a 1- or 2-inch-thick layer, respectively.

Slab design: Using loose sand requires more concrete

CBR values are sometimes used by floor designers to estimate the modulus of soil reaction (lbs/in.^{3}), or k-value. Using Figure 2, we converted the CBR values from our study to k-values, as shown in Table 3. The k-values are used in slab-thickness design charts to represent the support of the underlying subgrade-subbase combination.

Figure 3 is a design chart from the Portland Cement Association's commonly used slab-on-grade design method. As Table 3 shows, the estimated k-value for soil sample No. 3 decreased from 210 to 50 when a 2-inch sand layer was used. The example problem shown on the chart illustrates the effect of this decrease. For a k of 200, the design slab thickness is about 11 inches, but for a k of 50 it increases to 12 inches (see Reference 3 for the complete example). For lighter loadings that yield thinner slabs, required thickness would still increase by about an inch for a k-value decrease from 200 to 50. For soil sample No. 1, the average k-value with a 2-inch sand layer is 18, which is lower than the lowest value (50) on the design chart.

What's the significance of an extra inch of concrete floor thickness? A value-engineering audit for a floor design sometimes results in slab-thickness decreases as small as ½ inch. Increasing the thickness of a 100,000-square-foot warehouse floor slab by 1 inch would cost about $20,000. The cost of the extra concrete (more than 300 cubic yards) would be about equal to what the concrete floor contractor would be paid for placing and finishing.

What happens if the concrete slab is designed without considering the effect of the sand layer? Based on the design charts and other information (Refs. 2 and 3) for the example shown in Figure 3, the use of a loose sand layer that decreases the k-value from 200 to 50 would result in:

- A flexural stress increase of 25%
- A safety factor decrease from 2.0

Reasons to avoid using sand

There are many reasons for not placing a sand layer under a concrete slab (Ref. 1). These include difficulty in:

- Maintaining a flat, level sand surface during concrete placement
- Maintaining the specified reinforcing steel or dowel basket elevation due to sinking chair supports
- Producing a uniform slab thickness due to shifting sand displaced by concrete

In addition, one engineer (Ref. 2) has linked a sand layer to poor joint performance. He found that under forklift traffic, shifting sand beneath the joint resulted in reduced load-transfer efficiency across the joint. This was especially true at joints where aggregate interlock was the only means of load transfer.

ACI 302.1R-96 (Ref. 3) also discourages the use of a sand layer: "Base material should be a compactible, easy-to-trim, granular fill that will remain stable and support construction traffic. The use of so-called cushion sand or clean sand with uniform particle size, such as concrete sand meeting ASTM C 33, will not be adequate. This type of sand will be difficult, if not impossible, to compact and maintain until concrete placement is complete."

In revising its "Concrete In Practice" series, the National Ready Mixed Concrete Association is eliminating references to a sand layer and using ACI 302 terminology for base material. But specifiers still call for sand cushions, and some articles and publications still suggest using a sand layer under a concrete slab (Refs. 4 and 5).

References

1. Carl Bimel, "No Sand, Please," *The Construction Specifier,* June 1995, p. 26.

2. William J. Brickey, "Try Crushed Stone Under Slabs-on-Grade," The *Construction Specifier*, July 1990, p. 9.

3. ACI 302.1R-96, "Guide for Concrete Floor and Slab Construction," American Concrete Institute, Farmington Hills, Mich., 1996, p. 16.

4. "Concrete Slab Surface Defects: Causes, Prevention, Repair," Portland Cement Association, Skokie, Ill., 1987, pp. 4, 7, 10.

5. Gregory Dobson, "Concrete Floor Slabs: Recognizing Problems Before They Happen," *Concrete International*, June 1995, p. 46.

to 1.6
- An actual flexural stress that exceeds the fatigue limit, meaning that floor failure would now be determined by load repetitions rather than maximum load
- Failure at 14,000 load repetitions, though the floor was designed for an unlimited number of load repetitions

When specifiers require contractors to place concrete over a sand layer, the contractors don't know if the designer has increased the slab thickness to account for the weaker sand-layer support shown by our data. If the slab thickness wasn't increased, more later-age cracking and poorer joint performance may result, especially for slabs subjected to heavy construction loads, such as cranes or concrete trucks.

There are many good reasons for not using a sand layer under a concrete slab (see sidebar). If specifications call for a sand layer, contractors should discuss the implications with the architect and engineer before the project begins, and request that the sand layer be replaced with a compactible stone base. Based on our data, repair costs for slabs placed on thicker sand layers shouldn't necessarily be borne by the contractor.

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1. ASTM D 1883-92 "Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils," ASTM, West Conshohocken, Pa., 1992.

2. Boyd C. Ringo and Robert B. Andersen, *Designing Floor Slabs on Grade,* 2nd ed., The Aberdeen Group, 1996.

3. ACI 360R-92, "Design of Slabs on Grade," American Concrete Institute, Farmington Hills, Mich., 1997.

Acknowledgment

All of the laboratory work for this investigation was carried out by Blair G. Peterson, senior engineering technician, CTC-Geotek, Denver. We appreciate Blair's conscientious attention to detail while conducting the tests.

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September 29, 2015

Mr. Eric Grove
F.S.D. Investments, Inc.
P.O. Box 9879
Jackson, WY 83002

RE: PRELIMINARY SLOPE STABILITY ANALYSIS, WESTVIEW TOWNHOMES, 1255 WEST HWY 22, JACKSON, WYOMING

Dear Mr. Grove:

At your request, we have performed a preliminary slope stability analysis for the proposed Westview Townhomes development at 1255 West Highway 22 in Jackson, Wyoming. This letter briefly summarizes our procedure and presents our recommendations for the project. In summary, the preliminary modeling indicates the slope is likely to be stable and there is relatively low risk of destabilizing the slope with the proposed development.

This analysis does not constitute an appropriate final design and a site specific geotechnical investigation is required to better understand the underlying subsurface conditions. Three areas of greatest uncertainty are strength of underlying soils, the depth to an anticipated failure surface (i.e., weak soil), and seasonal groundwater fluctuations. We are happy to provide a scope of work for such an investigation and analysis at your request.

Site Description

The project site located in Jackson, Wyoming, along Highway 22 just north of the intersection of Highways 22 and 89, at the southwestern toe of East Gros Ventre Butte. A slope steeper than 30% separates two portions of the lot referred to in this report as the upper and lower benches (Figure 1). Preliminary plans indicate several townhouses are proposed for the lot, including 2 to 3 on the upper bench and 4 to 6 along the toe of the slope on the lower bench.

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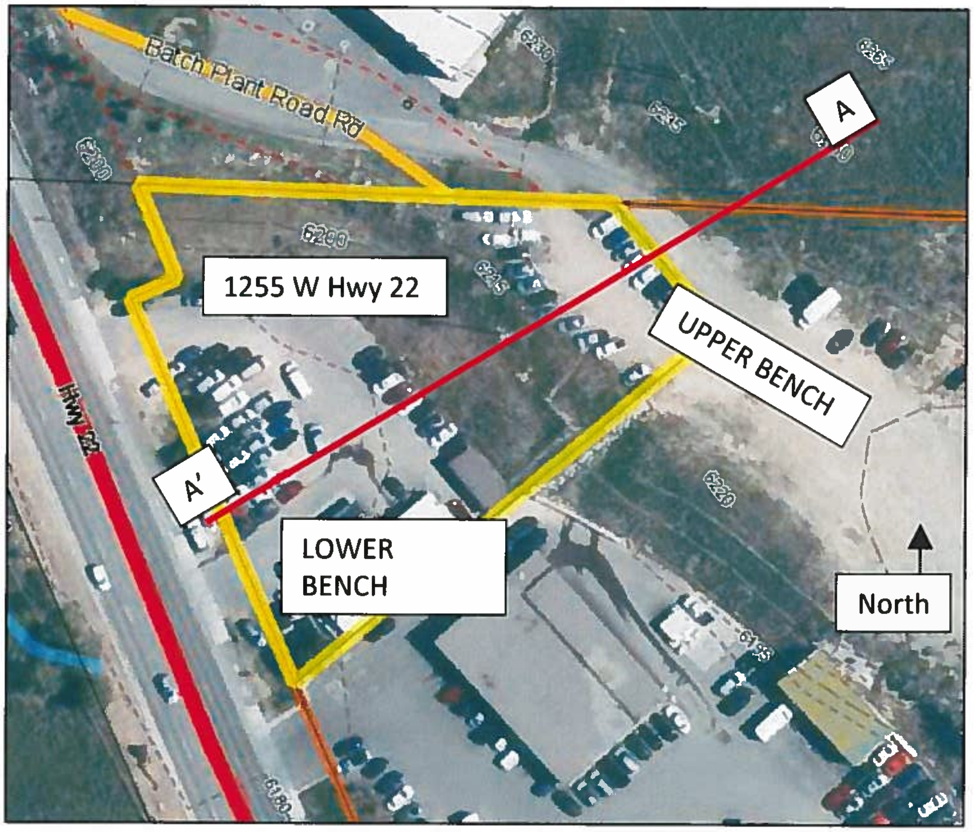

Figure 1: Site Plan and Cross Section Location Map

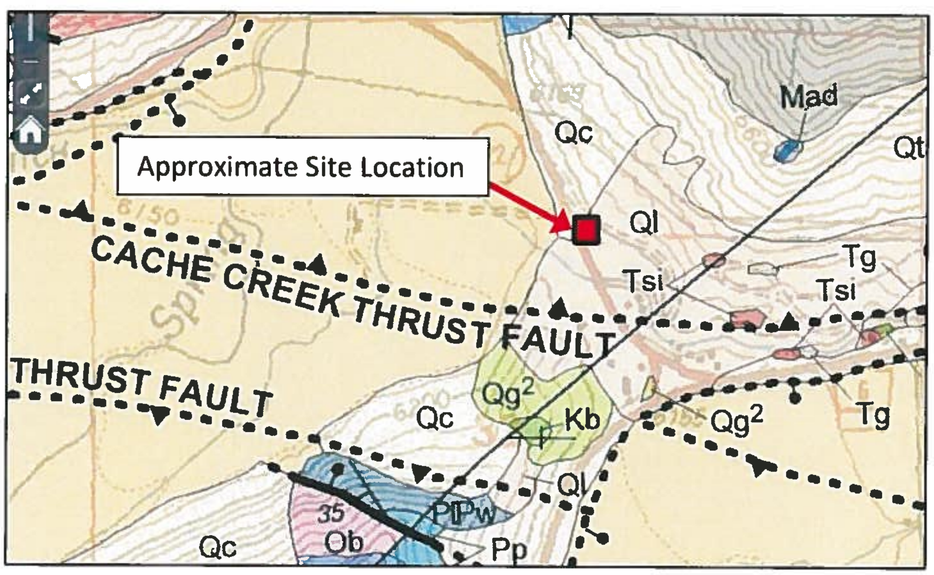

Figure 2: Geologic Map (Love, 2003)

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Stability Analysis Methodology

The modeled cross section chosen to be representative of the slope geometry on the property is shown on Figure 1. The following methodology was performed in order to develop the stability model:

1. **Geology**: The property is found on the Geologic Map of the Jackson Quadrangle (Love and Albee, 1972; Love, 2003), shown in Figure 2. The map shows the location of surface deposits, bedrock units, and geologic structures (i.e., faults and folds). The project site is shown on the map along the boundary between Quaternary-age windblown deposits called loess (Ql) and gravity deposits (i.e., colluvium, Qc). Geologic contacts are rarely as abrupt as indicated by the map and mixing or layering of the loess and colluvium in the subsurface is possible.

   Drilling on properties nearby to the southeast observed clay and silt lake beds believed to be part of the Tertiary Shootin' Iron Formation (Tsi) at elevations similar to the elevation of the lower parking lot of the project site.

2. **Geometry:** Figures 3 through 6 show the modeled cross section and predicted external and internal geometry. External geometry (i.e., ground surface) of the cross-section was developed using topographic data and historical aerial photography from the Teton County GIS website. Historical photography shows the upper bench and lower parking lot were constructed sometime in the 1960s with minor improvements being made from then to the present. It would appear the upper bench was constructed using cut and fill techniques meaning the fill would be recycled native soil (i.e. loess or colluvium).

   Internal geometry (i.e., subsurface ground conditions) is limited by our understanding of the subsurface conditions at the site. For this preliminary analysis, we projected conditions from boreholes on an adjacent property. **Additional investigation (i.e., site specific drilling, lab testing, etc.) is required for final analysis and design.** Lakebeds may be assumed to be horizontal and little geological movement is believed to have occurred since Tertiary time. However, nearby borings did not reach far enough into the fill or native material of the upper bench to determine the nature of the geological contacts.

   Slip surfaces were developed using a "Block Specified" approach. In this model, the left and right "blocks" were collapsed to create points along a line to examine translational failure of the slope along the lakebed deposits, assumed to be the critical mechanism of failure. The program creates hundreds of slip surfaces by connecting points of the blocks and selects the critical slip surface as the one with the lowest Factor of Safety (FS). FS is the ratio of forces resisting slope failure divided by forces tending to cause failure. A FS

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of 1.0 indicates imminent slope failure. FS<1.0 implies failure and FS>1.0 implies stability.

3. **Materials:** Effective stress shear strength parameters pertaining to a Mohr-Coulomb strength model were estimated for the site soils. Strength of the lakebeds was estimated using correlations between the soil's plasticity index (PI) and peak (Ladd et al, 1977) or residual (Voight, 1973) strength. Soils often display strain-softening behavior, meaning they become weaker with shearing as in the case of slope movement, going from peak strength to residual strength. The peak and residual strengths provide an upper and lower bound for behavior of the soil.

   Lakebeds along the base of East Gros Ventre Butte are likely comprised of layers of silt and clay. Failure, should it occur, would be assumed to follow a layer of clay and we have estimated strength values assuming PI values in the range of 25 to 30. Table 1 shows estimated soil parameters used in the stability analysis.

   The shear strength consists of two parameters: cohesion (c'), which expresses the shear strength at zero overburden pressure, and friction angle (φ'), which expresses the relationship between overburden pressure and shear strength, i.e., that shear strength increases with loading, from a minimum of c'.

   As indicated in Table 1, the residual strength is the lowest strength, usually occurring in soils that have been previously sheared. Most undisturbed soils exist at peak strength. Unless the slope is known to have previously moved, it is appropriate to use the peak strength.

Table 1: Modeled Soil Properties

Layer Name	Strength Model	Unit Weight (pcf)	Cohesion (c',psf)	Friction Angle (φ', degrees)
SILTY LOESS	Mohr-Coulomb	110	50	28
STONY COLLUVIUM	Mohr-Coulomb	125	0	35
LAKEBEDS – RESIDUAL STRENGTH, LOWER BOUND	Mohr-Coulomb	100	0	20
LAKEBEDS – PEAK STRENGTH, UPPER BOUND	Mohr-Coulomb	100	0	30

4. **Phreatic Surface:** Groundwater at this site appears to be deep and has not been included in the model. A site investigation will involve installing piezometers at depth to measure seasonal fluctuations of groundwater at the site, which if present will be used in a more detailed model.

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5. **Seismicity**: The site (Latitude: N 43.476°, Longitude: W 110.790°) is in an area of moderate seismic activity. The current peak horizontal acceleration (%) with 10% probability of exceedence in 50-years is 0.198g, according to the USGS National Seismic Hazard Maps (2008). Seismicity is assessed in the slope stability models using a pseudo-static method with half the horizontal seismic load, or k_h = 0.1g.
6. **Building Loads**: Due to the number of construction variables at this point in design and uncertainties involved in the preliminary model, we decided not to include building loads, which is typically small compared to soil pressures. Loading from building and site grading is estimated to be minimal. Foundations constructed on the upper bench may result in a net reduction of driving force, increasing the FS with respect to slope stability.
7. **Analyses:** The slope stability analyses were performed using the SLOPE/W stability module of GeoStudio 2012 version 8.15.1.11236, produced by GEO-SLOPE International, Ltd. The Morgenstern-Price limit equilibrium method, which takes into moment and force equilibrium, was used to analyze slope stability. Schematic cross-sections are shown on Figures 3, 4, 5, and 6.

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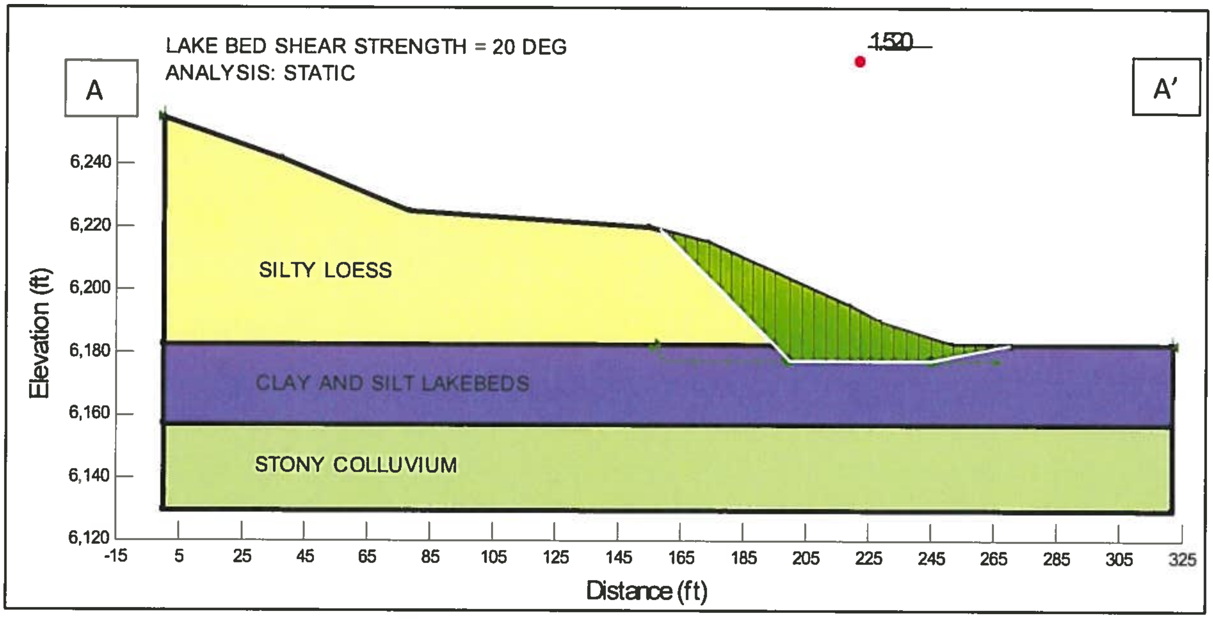

Figure 3: Lower Bound (Residual) Strength of Lakebed Soil - 20°, Static Analysis

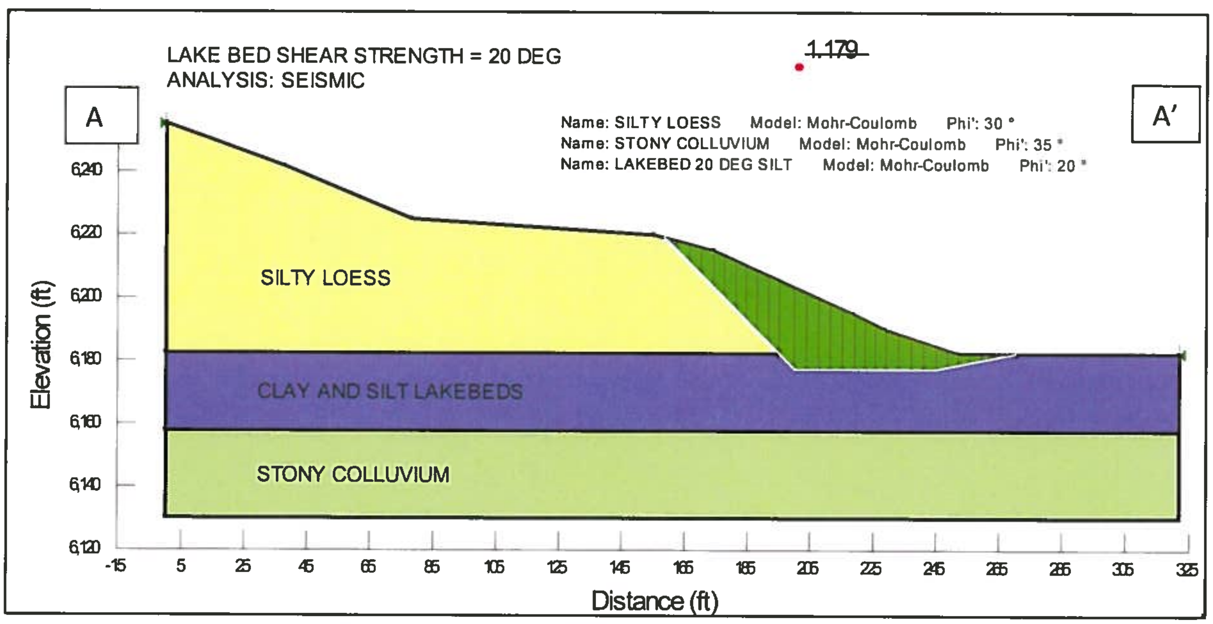

Figure 4: Lower Bound (Residual) Strength of Lakebed Soil - 20°, Seismic Analysis

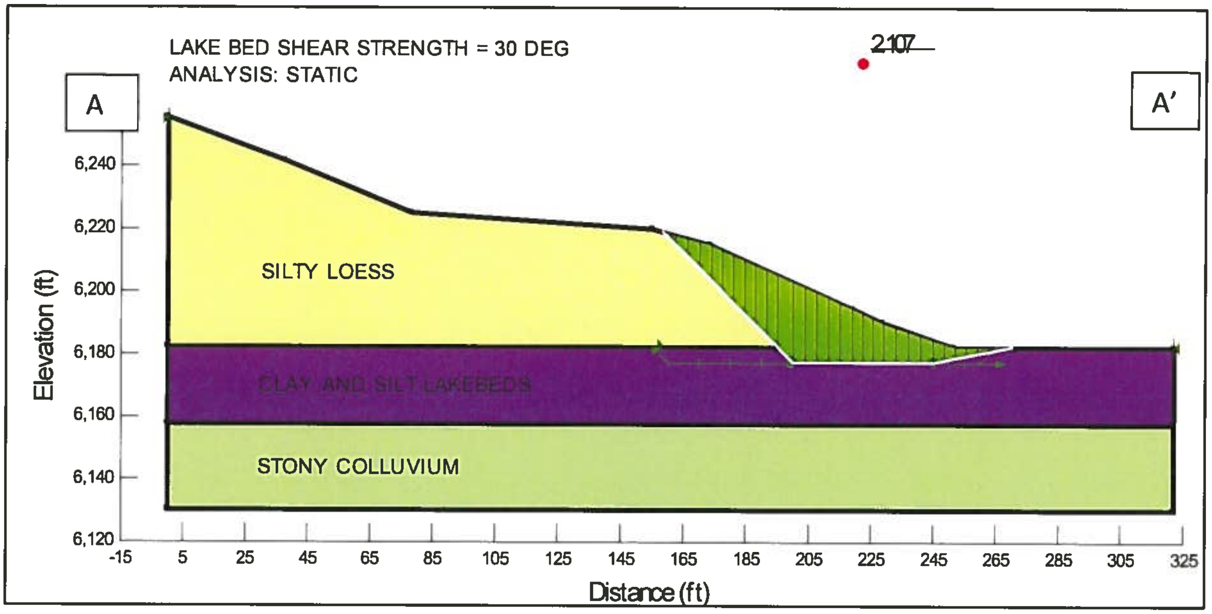

Figure 5: Upper Bound (Peak) Strength of Lakebed Soil - 30°, Static Analysis

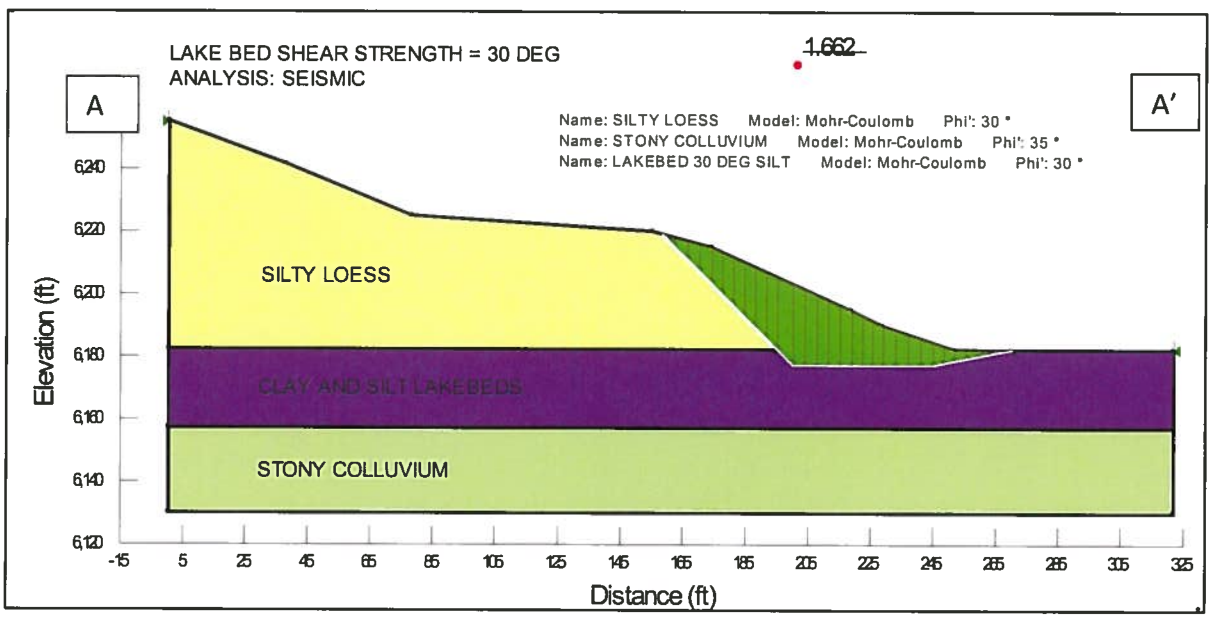

Figure 6: Upper Bound (Peak) Strength of Lakebed Soil - 30°, Seismic Analysis

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Stability Analysis Results

Results of the preliminary stability analyses are shown in Table 2 and cross sections of each analysis with critical slip surfaces and associated factors of safety (FS) are presented in Figures 3, 4, 5, and 6. FS with respect to slope stability indicate the slope is likely stable..

Table 2: Stability Analysis Results

Figure No.	Lakebed Strength	Analysis	Factor of Safety
3	Lower Bound – 20° (Residual)	Static	1.52
4	Lower Bound – 20° (Residual)	Seismic	1.18
5	Upper Bound – 30° (Peak)	Static	2.11
6	Upper Bound – 30° (Peak)	Seismic	1.66

FS values are above values generally accepted by engineering practice for slope stability (FS > 1.5 static and FS > 1.1 for seismic). Soil strength of the lakebed soils are likely greater than the estimated lower bound and likely to result in FS values well above required limits. Laboratory testing of the lakebed soils during the recommended site investigation will verify these estimates.

Limitations

This report has been prepared based on a very limited amount of data. At this point, geotechnical uncertainties are high and actual site conditions may vary considerably from the assumptions made in these analyses. Site specific investigation, laboratory testing, and modeling is required before final development and design. Stability analyses are dependent upon a number of conditions including, but not limited to: slope geometry, construction methods, building loads, runoff and other water features, etc. Changes in design and construction of the proposed development could dramatically change the inputs to the model. As such, recommendations in this letter and future stability analysis are contingent upon our involvement for the duration of the project.

These services have been performed in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing in this area under similar conditions. No other warranty is made or implied.

If you have any questions about this report, or if we may provide other services to you, please contact us. As the project progresses, we will be available to answer questions.

Respectfully submitted,
JORGENSEN GEOTECHNICAL, LLC

Reviewed by:

Colter H. Lane

Ray Womack

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April 27, 2016 2498

Mr. Tyler Sinclair
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Geotechnical 3rd Party Review
Proposed Westview Townhomes Project
1255 West Highway 22, Jackson, Wyoming

Dear Mr. Sinclair:

In accordance with your request, we have performed a 3rd Party geotechnical review for this proposed residential development. Tasks included:

1. Evaluate project information (letters from Jorgensen dated September 29, 2015 and February 9, 2016).
2. Review geologic maps and geotechnical reports for nearby sites.
3. Perform stability analyses to check Jorgensen's model and to evaluate the effect of groundwater and strength parameters.

Background Information

The site is located near the intersection of West Highway 22 and West Broadway Avenue, at the toe of the East Gros Ventre Butte slope. The site has been regraded in the past to create two benches with a steep fill slope between, which was presumably accomplished with a combination of excavation and filling. The preliminary project plan is to construct townhomes on both benches.

Geologic conditions are described on the Geologic Map of the Jackson Quadrangle, LMS-9, published by the State of Wyoming Geologic Survey (Love & Albee, 2004). In addition, subsurface conditions and geotechnical data are provided in the Womack report for the adjacent Clark property to the southeast (dated March 14, 2008) and the Landslide Technology reports for the nearby landslide at Budge Drive / West Broadway Avenue (June 2014).

Pleistocene glaciation shaped many of the valleys and sediment deposits in the region. The primary geologic units affecting slope stability include loess, talus and colluvium on the butte slope and alluvial and lakebed clay/silt deposits near the valley floor. Based on the 1963 USGS topographic map for the Jackson Quadrangle, the lower portion of the natural butte slope is inclined approximately 14 degrees from horizontal (approximately 25% slope). Gravelly silt fill comprises the slope between the two benches (based on subsurface explorations by Womack, 2008), where the slope is inclined approximately 23 degrees from horizontal (approximately 43% slope), based on the topographic map shown on Figure 2 of the Womack report on the Clark property (2008).

The colluvium developed due to erosion and raveling from the upper butte slope. Glacial advances and retreat contributed to butte slope erosion, raveling and sliding. The colluvium displaced to the lower portion of the butte slope and slid onto, or interfingered with, the lakebed clay/silt and alluvial soils. The lakebed unit varies in consistency from soft to hard, and includes sheared zones where past landsliding and interfingering occurred. Undisturbed lakebed clay will have relatively high peak shear strength; however, softened and/or sheared clay will have comparatively lower, residual shear strengths.

The Budge Drive / West Broadway Avenue landslide is located approximately 2,000 feet from the project site. Ring shear tests and stability back-analyses were performed in 2014 to determine the strength of sheared clay (residual shear strength). The tested residual shear strength values for two specimens were 12.4 and 15.0 degrees (angle of internal friction). Stability analyses were performed to back-calculate the residual shear strength friction angle, which ranged from 10 to 12 degrees (angle of internal friction).

Groundwater exists within the butte and spring/seepage areas near the toe of the butte slope have been observed and reported by others. Groundwater was encountered and measured in instrumented borings made in the landslide at Budge Drive / West Broadway Avenue. While no groundwater was detected during the subsurface investigations for the Clark property, those borings were not instrumented to measure groundwater levels. Seasonal groundwater fluctuations cannot not be captured without instrumentation. In our opinion, groundwater should be anticipated to occur perched on the clay layers underlying the lower butte slope in response to infiltration during wet periods and snowmelt, consistent with observations in the vicinity.

Slope Stability

The stability of the fill slope that exists between the two benches on the project site was analyzed parametrically by Jorgensen Geotechnical for conceptual planning purposes (September 29, 2015 report). To perform these analyses, assumptions were made for material properties and groundwater conditions. The shear strength for the lakebed clay unit was modeled for parametric analysis using lower bound and upper bound assumptions (20 and 30 degrees angle of internal friction, respectively). Groundwater was assumed to be deep, below the trial slip surface in the clay layer used in the stability analysis. The results of the stability analysis assuming the lower bound shear strength of the clay indicated a Factor of Safety (FS) of approximately 1.52 (static). The report states "FS values are above values generally accepted by engineering practice for slope stability (FS > 1.5 static and FS > 1.1 for seismic). Soil strengths of the lakebed soils are likely greater than the estimated lower bound and likely to result in FS values well above the required limits. Laboratory testing of the lakebed soils during the recommended site investigation will verify these estimates."

The subject regraded slope has reportedly not experienced slope instability in recent years, which indicates the Factor of Safety is greater than 1.0. Whether or not the slope exceeds a FS of 1.0 is not discernable by precedence alone.

We performed a check of the stability analyses using the same cross section, material properties and assumed no groundwater impact on stability and calculated a similar Factor of Safety (1.5) for the lower bound strength assumption of 20 degrees angle of internal friction.

There is a possibility that the lakebed clay may be locally sheared, similar to that found at Budge Drive. As stated previously, ring shear tests on lakebed clay from Budge Drive explorations resulted in a 12.4 degree residual shear strength (angle of internal friction). We performed a stability analysis using an alternative lower bound residual shear strength of 12 degrees angle of internal friction. The resulting FS assuming no groundwater impact is approximately 1.1 (static), which indicates marginal stability.

We also analyzed the effect of perched groundwater on top of the clay, which we consider reasonable to assume based on seepage evidence and measurements of groundwater levels in the vicinity. Seasonal groundwater levels are not known at this site. We would estimate the groundwater to possibly be 5 to 15 feet high above the clay for preliminary parametric analyses. Assuming a groundwater head of 10 feet above the assumed slip surface, the FS could be reduced approximately 15%. For the lower bound shear strength case used in Jorgensen's model (20 degrees angle of internal friction), the estimated Factor of Safety would reduce to approximately 1.25, with the addition of 10 feet of groundwater head acting on the sliding plane.

Review Comments

It is possible that the clay shear strength could be locally less than 20 degrees angle of internal friction and that there could be groundwater pressures that affect slope stability during wet periods. A combination of these factors would result in a local Factor of Safety less than 1.5, and possibly in the range of FS = 1.0 to 1.2 (static), indicating marginal stability. It is possible that the slope stability Factor of Safety could be less than generally accepted by engineering practice.

The effect of the proposed project on slope stability has not been evaluated since conceptual regrading plans have not been provided.

Very truly yours,

LANDSLIDE TECHNOLOGY

George Machan

George Machan, P.E.
Senior Associate Geotechnical Engineer