JORGENSEN
Geotechnical Engineering and Geology

May 15, 2015

Mrs. Shawn Ankeny
P.O. Box 11062
Jackson, WY 83002

RE: GEOTECHNICAL INVESTIGATION REPORT FOR NEW RESIDENTIAL CONSTRUCTION AT LOT 10 RIVER MEADOWS SUBDIVISION, TETON COUNTY, WYOMING

Dear Shawn:

We are pleased to present this geotechnical investigation report for the proposed new residence at Lot 10 of the River Meadows subdivision in Teton County, Wyoming. The report describes site conditions and presents conclusions and recommendations to support design and construction of foundation elements.

In summary, the site is covered by a deep deposit of loess (i.e. windblown silt) greater than 23-ft thick in some areas. Loess is subject to collapse when wetted and building directly on these soils is not recommended without taking measures to reduce the risk of damage associated with collapsible soils.

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,

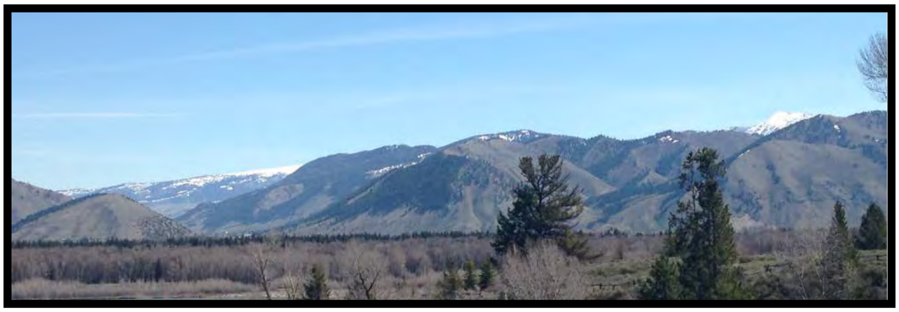
JORGENSEN GEOTECHNICAL, LLC

Ray Womack

WESLEY R. WOMACK
4958
Date 5/15/2015

Ray Womack, P.E., P.G.
Enclosure: Report

GEOTECHNICAL INVESTIGATION REPORT
LOT 10 RIVER MEADOWS SUBDIVISION
TETON COUNTY, WYOMING



Prepared for:

Mrs. Shawn Ankeny
P.O. Box 11062
Jackson, WY 83002

Prepared by:

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May 15, 2015

TABLE OF CONTENTS

1.0 EXECUTIVE SUMMARY 1
2.0 INTRODUCTION 1
3.0 PROPOSED CONSTRUCTION 2
4.0 INVESTIGATION PROCEDURE 2
  4.1 Field Investigation 2
  4.2 Laboratory Analyses 2
  4.3 Report Preparation 2
5.0 SITE CONDITIONS 5
  5.1 Description 5
  5.2 Geology 5
  5.3 Soils 5
  5.4 Groundwater 6
  5.5 Earthquakes and Seismicity 6
  5.6 Geologic Hazards and Liquefaction 6
6.0 ENGINEERING ANALYSES AND RECOMMENDATIONS 8
  6.1 Settlement 8
    6.1.1 Helical Piers 8
    6.1.2 Over-Excavation and Re-Compaction of Collapsible Soil 9
  6.2 Bearing Capacity 11
  6.3 Lateral Loads on Foundation Walls 11
  6.4 Soil Friction 12
  6.5 Excavation and Cut Slope Stability 12
  6.6 Final Grading 12
  6.7 Interior Slabs-on-Grade 12
  6.8 Exterior Slabs-on-Grade 13
  6.9 Foundation or Basement Drains 14
  6.10 Ventilation and Treatment 16
  6.11 Reinforcing, Utilities Testing, and Concrete Considerations 16
  6.12 Observation during Construction 16
7.0 LIMITATIONS 16
8.0 REFERENCES 17

LIST OF FIGURES

Figure 1 - Site Location and Geologic Map.......... 3
Figure 2 - Test Pit Location Map.......... 4
Figure 3 - Design Seismic Spectral Response.......... 7
Figure 4 - Over-Excavation and Re-Compaction of Fine-Grained Soil.......... 10
Figure 5 - Foundation Drain Alternatives.......... 15

LIST OF TABLES

Table 6-1: Lateral Pressure Parameters for Compacted Exterior Loess Backfill.......... 11

LIST OF APPENDICES

Appendix A: Test Pit Logs
Appendix B: Loess Article
Appendix C: Concrete Publications

1.0 EXECUTIVE SUMMARY

Lot 10 of the River Meadows subdivision is about 3.5 miles south of the town of Wilson, Wyoming, in Teton County. The subsurface of the site is characterized by Quaternary age glacial terrace deposits (Qtg) consisting of loess (i.e. windblown silt) overlying dense glacial gravel and cobbles. The loess deposit at the site is greater than 18-19 feet thick.

Loess is very susceptible to frost and is potentially subject to collapse when wetted. Distress to buildings and landscaping founded on loess tends to be very episodic and structures lacking special treatment of the loess may last for years without any problems. Damage typically occurs in the context of abrupt moisture increases caused by broken water lines, landscaping defects, or poor site grading. In our opinion, the risk of damage is too high and consequences too costly to ignore the potential collapse settlement of the native subgrade. Loess is notorious worldwide for causing severe damage to buildings.

As such, this report recommends two options reducing risk to settlement damage: installing deep foundation elements, such as helical piers, or over-excavation and re-compaction of the native loess. Landscaping elements and hardscapes should also be designed to tolerate movement or be supported with the same care as interior foundation elements. Landscapers and other designers should be provided this geotechnical report and formally briefed about the necessity to manage drainage and grades at the site. Building on loess is not without risk. No warranty of performance is made or implied.

Compaction of native fine-grained soils, either for engineered fill or exterior backfill, can be difficult and should be tested during construction. This office is available to provide density testing and supervision during fill placement. Should re-compacted native loess be used as structural fill, bearing capacities of is 3,000 psf and 5,500 psf for 16-inch wide footings buried at 4 feet bgs and 8 feet bgs, respectively. This office should review the foundation design once completed. Heavy foundation loads and large footings are not typically compatible with over-excavation. This office should be provided foundation plans for review.

This summary does not replace the entire report. It is the responsibility of any contractors and designers to read and understand the report in its entirety.

2.0 INTRODUCTION

Jorgensen Geotechnical, LLC, conducted a geotechnical site investigation at Lot 10 of the River Meadows Subdivision in Teton County, Wyoming (Figure 1). The purposes were to observe soil and groundwater conditions, evaluate soil-engineering properties, and to provide recommendations to support design and construction of foundation and drainage elements. The scope of services included excavating and logging two exploratory test pits, engineering analyses, and generating this geotechnical investigation report.

3.0 PROPOSED CONSTRUCTION

Design of the new residence is still in the early stages and preliminary plans were not available at the time of this report. It is our understanding that the proposed residence will have two floors. The lot comprises 0.38 acres and we understand the house will occupy a substantial portion of it. Woodchuck Road is the existing access up the slope from South Fall Creek Road. Most of the lot is timbered and at the time of the investigation no improvements had been made on the lot.

4.0 INVESTIGATION PROCEDURE

4.1 Field Investigation

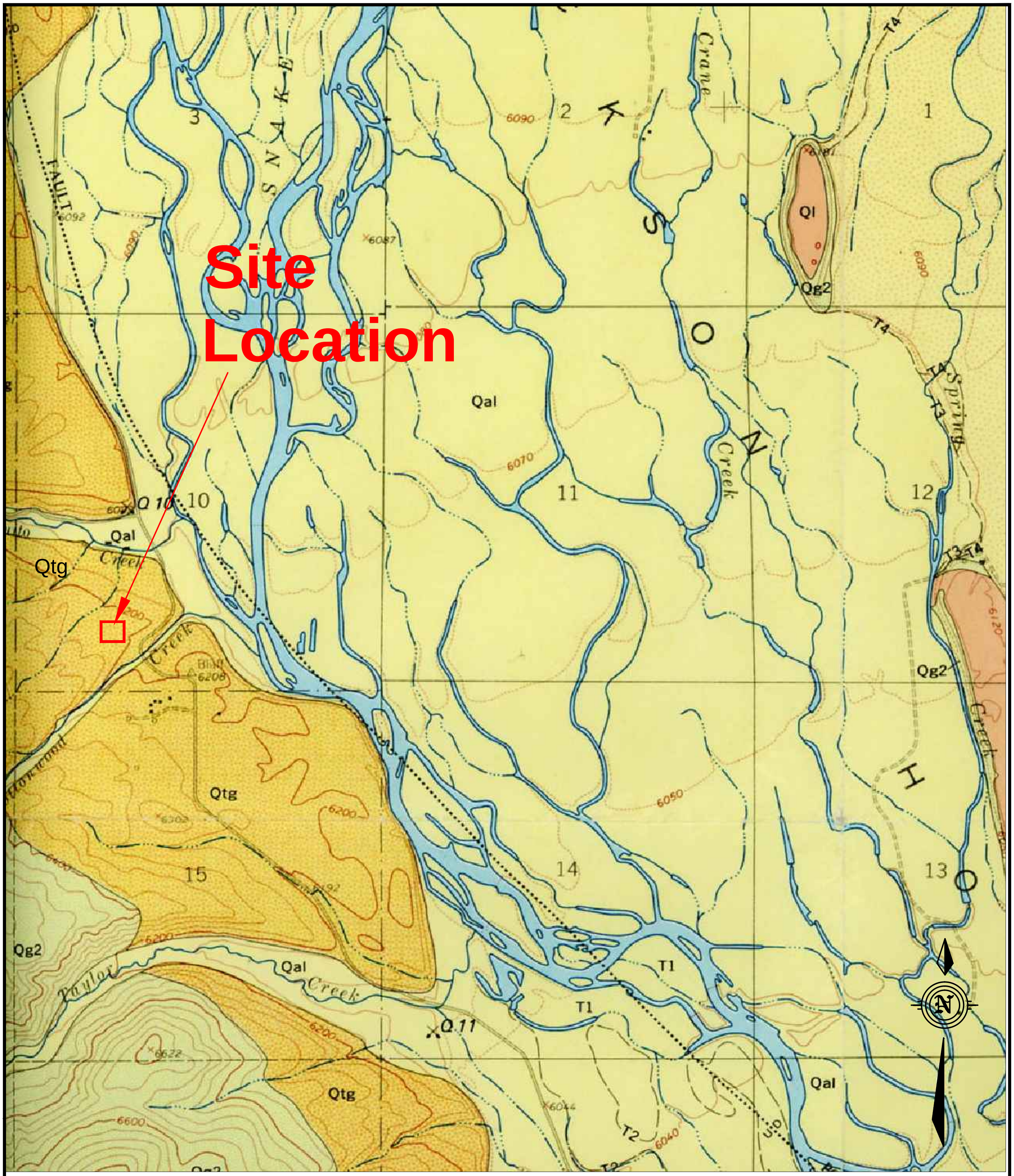
The field investigation was conducted on April 28, 2015. Two test pits were excavated to depths of about 18 and 19 feet below the ground surface (bgs). Excavation of the test pits was terminated at the limits of the excavation equipment. Soil type, thickness, consistency, and relative moisture content were observed and documented by a staff Geotechnical Engineer. Site conditions may vary and actual soil conditions may differ from those represented in the exploration logs. Test pit locations are shown on Figure 2 and test pit logs are presented graphically in Appendix A.

4.2 Laboratory Analyses

Laboratory tests were not performed for the site due to our familiarity with the loess; our office has tested similar soils from sites nearby.

4.3 Report Preparation

The report describes the geological site conditions and includes a site location and geologic map, and test pit logs. The report provides engineering analyses and preliminary recommendations for construction of foundation elements.



Map Symbols: Qal - Alluvium
Ql - Loess
Qtg - Terrace Gravel
Qfp - Flood Plain Deposit

From Love et al, 2000, Geologic Map of the TetonVillage Quadrangle, Teton County, Wyoming, MapLMS-2

SCALE: 1" = 2000'

Geotechnical Investigation Report
5370 Woodchuck Road
River Meadows Subdivision
Teton County

Site Location and Geologic Map

FIGURE
1

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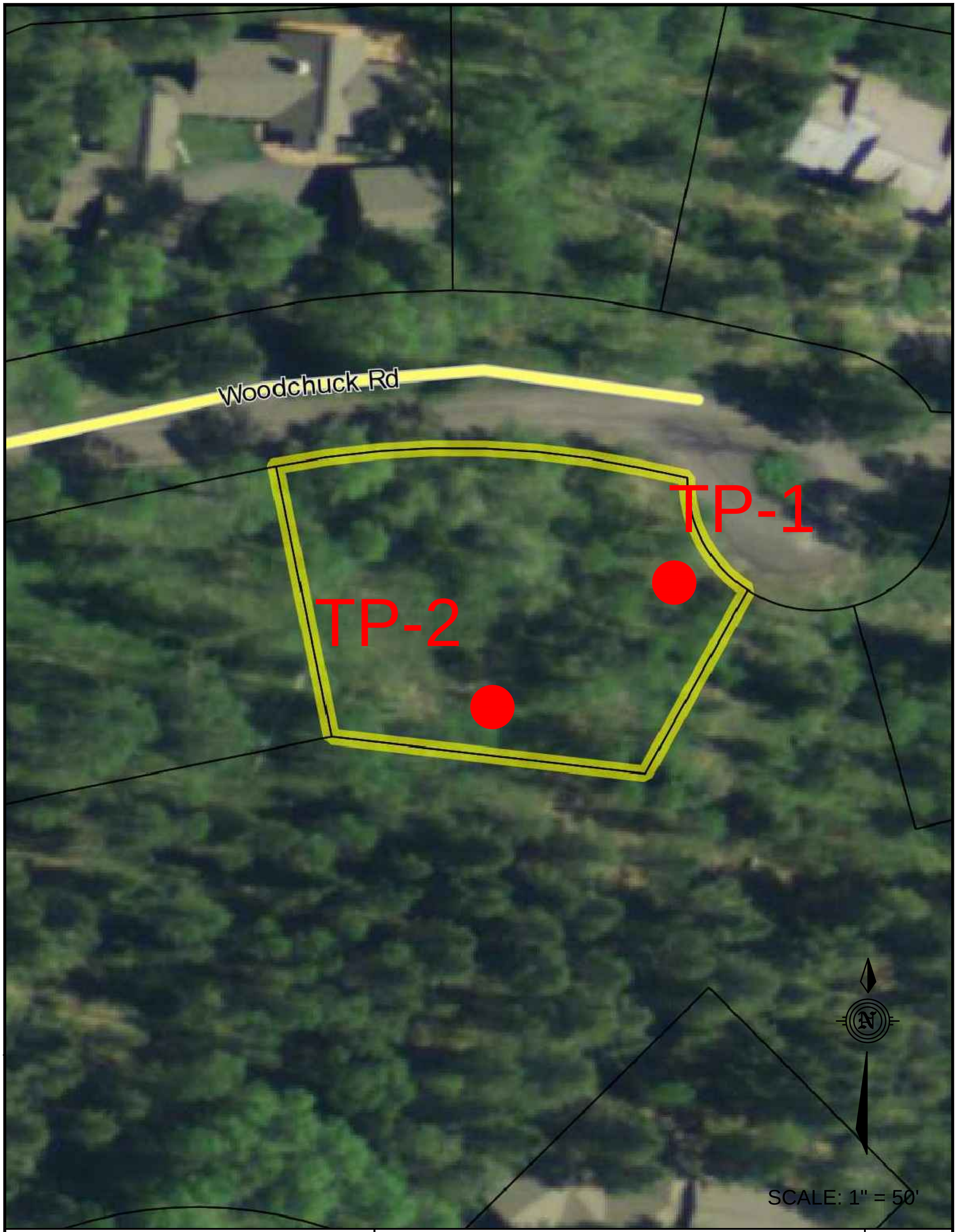

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River Meadows Subdivision
Teton County, Wyoming

Test Pit
Location Map

FIGURE
2

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5.0 SITE CONDITIONS

5.1 Description

Lot 10 of the River Meadows subdivision is located approximately 3.5 miles south of Wilson, Wyoming on the eastern flank of the Snake River Range in Teton County (Figure 1). The project site is located on a gently-sloping bench above the west side of South Fall Creek Road. The bench is about 70 feet higher than the valley floor and features steep slopes to the south. The lot sits at an approximate elevation of 6,215 feet above mean sea level

5.2 Geology

Figure 1 is a generalized geologic map of the Jackson Quadrangle (Love and Albee, 1972), which shows the location and type of surface deposits, bedrock units, and geologic structures (i.e., faults and rock orientations). This portion of Jackson Hole is mapped as undifferentiated Quaternary-aged terrace deposits (Qtg), consisting of loess overlying glacial gravel and cobbles. Soils encountered at the site are consistent with the mapped units.

In 1972, Love and Albee mapped a “postulated” portion of the Teton Fault just east of the property, as evidenced by the steep drop from the bench down to the Snake River floodplain (Figure 1). The dotted portion indicates that the fault contact is concealed. Current thinking assumes the fault ends at Teton Pass. However, the site is subject to earthquake shaking and additional seismic discussion is covered in Section 5.5 of this report.

5.3 Soils

Soils encountered at the site consist of loess, windblown silt that is susceptible to collapse when wetted under load (such as building foundations). Loess was encountered in both of the test pits and the bottom of the loess was not observed within the reach of the Hitachi Zaxis 160 trackhoe, or 19.3 feet bgs. The loess is described as clayey silt, dry to moist, dark brown to buff/tan, soft to medium stiff, and massive. Color and stiffness of the deposit varies widely with moisture content generally becoming lighter in color and stiffer with depth (ie. less moisture influence from surface infiltration). Consolidation tests performed on loess sampled nearby indicate the soil can collapse up to 6-8% of its volume when saturated under load of 2,000 psf. For your convenience, we have provided an article regarding construction in loess soils in Appendix B.

Glacial till is expected to underlie the loess, but the depth to the stony soils is not known. The glacial till is likely from the older Bull Lake glacial period and is very dense and lime-cemented, providing a suitable bearing material for deep foundations, such as helical piers. The age of the till and our observations in the area indicate the glacial terrace was exposed to an extended period of erosion prior to deposition of the windblown loess; therefore the underlying gravel/cobble surface is likely to be uneven.

5.4 Groundwater

Groundwater was not encountered in the test pits and is expected to be over 50 feet in depth.

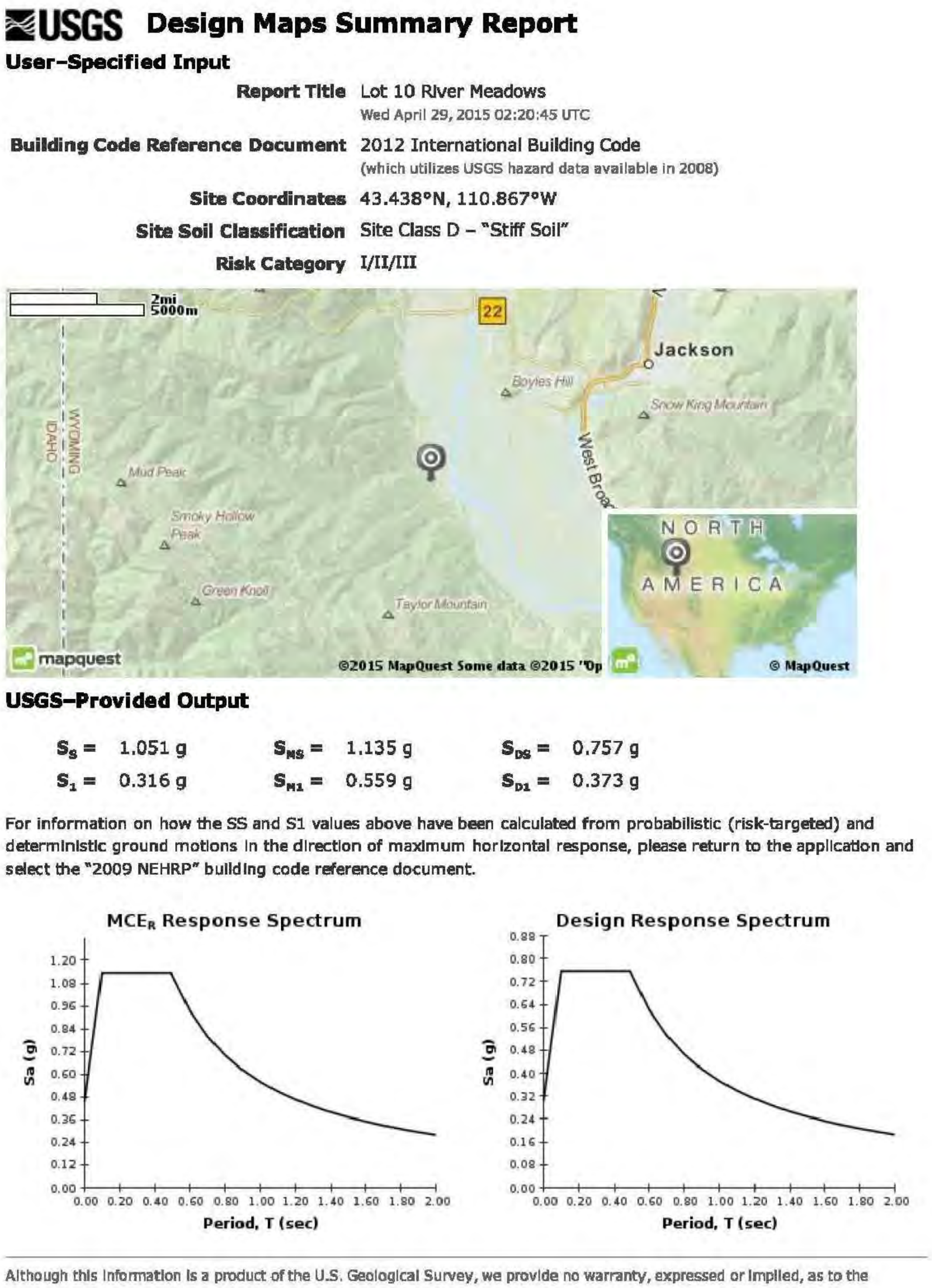
5.5 Earthquakes and Seismicity

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. 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).

Ground motion accelerations and spectral response were derived for the project site in accordance with the general procedure defined in the 2012 International Building Code (IBC). 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 Site Ground Motion and Spectral response is presented in Figure 3. 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.

5.6 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.



USGS Design Maps Summary Report

User–Specified Input

Report Title Lot 10 River Meadows
Wed April 29, 2015 02:20:45 UTC

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

Site Coordinates 43.438°N, 110.867°W

Site Soil Classification Site Class D – "Stiff Soil"

Risk Category I/II/III

USGS–Provided Output

S_S = 1.051 g S_{MS} = 1.135 g S_{DS} = 0.757 g

S_1 = 0.316 g S_{M1} = 0.559 g S_{D1} = 0.373 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.

Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the

Geotechnical Investigation Report 5370 Woodchuck Road River Meadows Subdivision Teton County, Wyoming	Spectral Response	FIGURE 3

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6.0 ENGINEERING ANALYSES AND RECOMMENDATIONS

6.1 Settlement

Loess may collapse when wetted, indicating that differential settlement may cause damage to the residence if soils below footing depth become wet. 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 the loess encountered at the site is collapsible and should be addressed accordingly. We recommend two foundation options to reduce the risk of excessive differential settlement: deep foundation elements or over-excavation and re-compaction of the native loess.

6.1.1 ***Helical Piers***

We recommend foundations consisting of grade beams supported by deep foundation elements, such as helical piers. Helical piers and grade beam foundations will allow very little vertical movement of structural elements. Installation of helical piers is likely to be more viable and economical than over-excavation and replacement of the collapsible loess. Helical piers should bear directly on the stony glacial deposits. Designing the building with a basement will decrease the length of the individual piers. However, please note that the depth to the glacial outwash gravel may be greater than 10 feet even from the bottom of a typical basement.

Helical piers may accommodate allowable loads of 25 to 50 kips, depending on the product chosen. The Structural Engineer should provide the spacing and placement of piers. 50 kip design compression loads have been used successfully in many places in Jackson Hole with similar ground conditions. Test piers would aid in determining the depth required and expected available bearing capacity (i.e., better estimate of cost per pier) and are strongly recommended. Since the depth to the glacial till is unknown across the project site, test piers should be installed as soon as possible as the design process may require alteration based on test pier results. Installation of all piers, including test piers, should be observed by a representative of this office. It will be the responsibility of the helical pier installer to bring appropriate test equipment to the site. If the contractor is retained for installation of production piers, they will often credit the cost of test piers.

Experience indicates that piers are likely to reach “refusal” within short distances of encountering the gravel/cobble, sometimes before the pier reaches design torque. Testing of production piers should verify tensile capacity.

6.1.2 *Over-Excavation and Re-Compaction of Collapsible Soil*

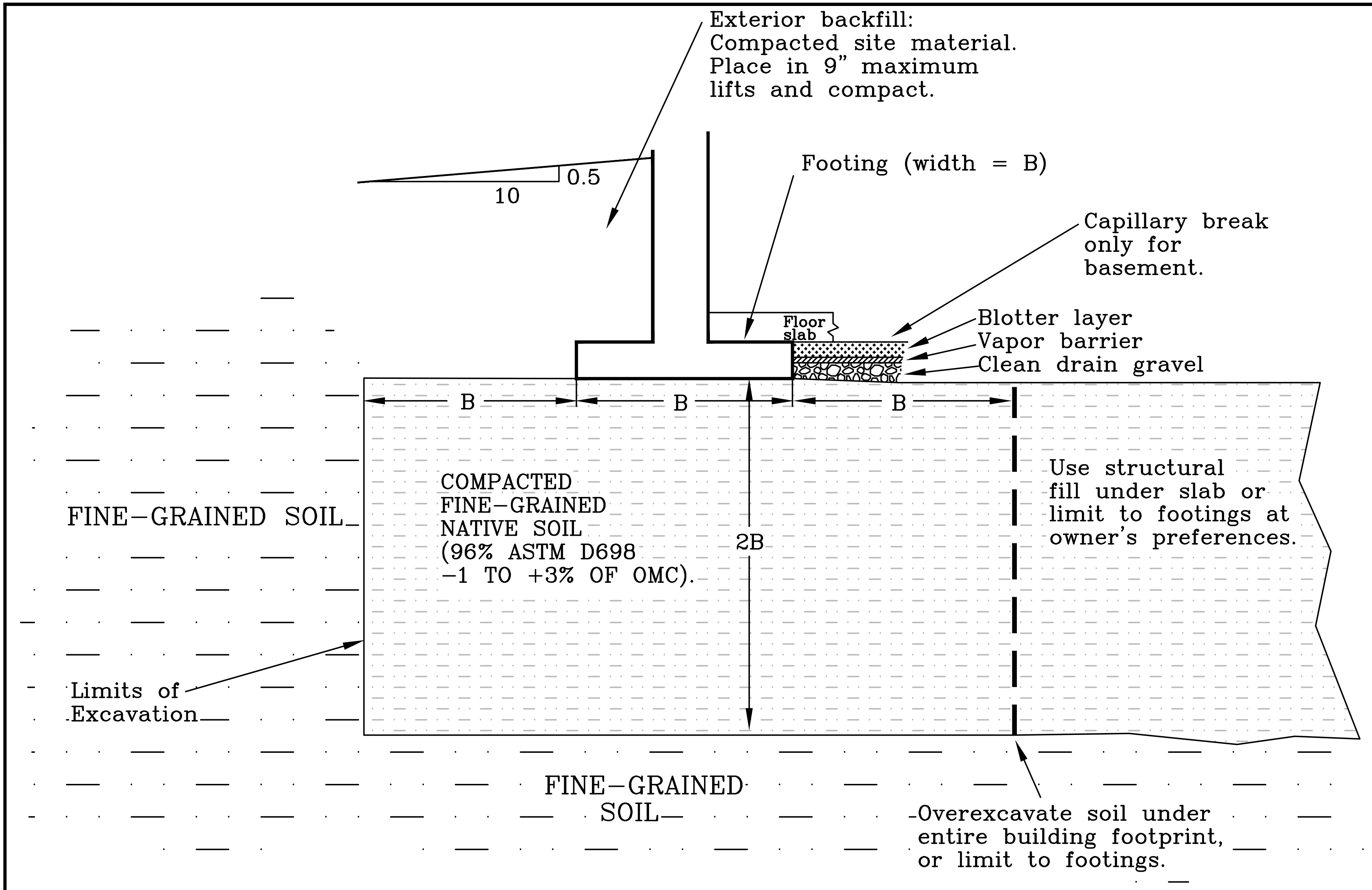
If deep foundations are not employed, building foundations should still be placed on engineered fills. This office recommends over-excavating the fine-grained soil and re-compacting with careful moisture-density control. **It should be noted that this method should only be performed with great care as moisture control and compaction is very difficult. An excavation contractor familiar with such a process should be selected.** Also, heavy foundation loads imposed by design snow loads and large footings are not typically compatible with over-excavation. This office should be provided foundation plans for review.

If construction begins in the spring or early summer, snowmelt and surface water runoff may be problematical. Freezing temperatures in the fall or winter also pose problems with moisture control. The most common cause of foundation failure is wetting of soils below foundations. 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.

Topsoil and loess should be over-excavated from beneath the footings and at least one footing width on the outside of the footings, as shown in Figure 4. It may be easier and certainly safer to over-excavate below the entire footprint of the building; i.e., below both footings and slabs Contractors often prefer to use pit run as replacement fill because pit run is likely to be easier to compact and less sensitive to moisture content. **However, the pit run may act as a moisture sink and cause wetting of the adjacent fine-grained soil.**

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. If natural soils are compacted, a minimum dry density of 96% ASTM D 698 is recommended in the zones of influence of foundations. It may be difficult to meet the standard in the first lift or two because the compactor will be working against uncompacted soil. Testing should occur in each compacted lift.

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. It is also important to provide density testing and supervision during fill placement, which this office is available to provide.



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5370 Woodchuck Road
Lot 10 River Meadows Subdivision
Teton County, Wyoming

Over-Excavation Detail

FIGURE
4

6.2 Bearing Capacity

Bearing capacity of soil refers to its ability to resist shear failure under load. The bearing capacity of re-compacted loess (should foundation option 6.1.2 be chosen) was estimated using Terzaghi's bearing capacity equation for strip footings (Bowles, 1996) and is 3,000 psf and 5,500 psf for footings buried at 4 feet bgs and 8 feet bgs, respectively. This assumes that 16-inch wide strip footings are used and that the soil has a unit weight of 110 pcf and an internal friction angle of 28 degrees. **If the final design has conditions other than those calculated, please contact this office to reassess the new bearing capacities.**

6.3 Lateral Loads on Foundation Walls

Table 6-1: Lateral Pressure Parameters for Compacted Exterior Loess Backfill

Condition	Coefficient of Earth Pressure	γK (equivalent fluid pressure)
Static Conditions Level Backfill*	$K_o = 0.53$ $K_a = 0.36$ $K_p = 2.77$	$\gamma K_o = 58$ pcf $\gamma K_a = 40$ pcf $\gamma K_p = 305$ pcf
Earthquake Conditions Level Backfill*	$K_{ae} = 0.43$ $K_{pe} = 2.58$	$\gamma K_{ae} = 48$ pcf $\gamma K_{pe} = 284$ pcf

*assumes a soil unit weight of 110 pcf with a friction angle of 28 degrees.

Research has indicated that lateral pressures due to earthquakes are non-hydrostatic in distribution, and the resultant acts above the lower third-point of the wall (Bakeer, et al., 1990). Accordingly, active soil pressures have been divided into two components that act at different wall heights. The static force (defined by $\gamma K_a = 40$ pcf) acts at the lower third-point. The Mononobe-Okabe equations were used to estimate dynamic forces against retaining walls and applied using half the maximum horizontal acceleration (Bowles, 1996; Hynes and Franklin, 1984; Whitman, 1990). The dynamic component is estimated as $0.5*(\gamma K_{ae} - \gamma K_a)$, which is approximately 4.0 H^2 pounds per foot of wall applied at 60% of the wall height.

Passive earth pressures were calculated using the Coulomb and Mononobe-Okabe equations (Bowles, 1996). At rest earth pressures (defined by $\gamma K_o = 58$ pcf) assume a horizontal ground surface behind the foundation wall. Use the at-rest pressure or the active pressure under seismic conditions for foundation wall design, whichever results in the higher resultant loads.

6.4 Soil Friction

A friction factor of 0.53, which is the tangent of 28 degrees, is suggested to calculate soil friction for design of concrete structures in contact with fine-grained (loess) subgrade. 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 loess subgrade material at the site as Type A soil. Simple cut slopes should be no steeper than 0.75H:1V (53 degrees). If the loess soil is found to be fissured, as is often the case, cut slopes should be no steeper than 1H:1V (45 degrees). According to OSHA regulations, any cut slope greater than 20 feet in height would require additional analysis. The contractor shall be responsible for adherence to OSHA and other safety regulations.

6.6 Final Grading

Properly compacted backfill and site drainage are extremely important. Final grading should provide positive drainage of at least 0.5 foot in the first 10 feet away from the structure. Adequate gutters are strongly recommended. Roof runoff should be discharged at least 3 feet away from the building or exterior slabs. Swales or other moisture collection points should be avoided if possible within 20 feet of the footings. Drainage swales should slope a minimum of 2%. There should be no irrigation within 5 feet of foundation walls. Irrigation pipes should be pressure tested when installed and checked annually for leaks.

Exterior backfill around buildings should consist of site materials placed in lifts and compacted to a standard of at least 92% Standard Proctor (ASTM D 698) and moisture-conditioned. Exterior fills should be placed as early as possible. Finer-grained material should be used in the upper 2 feet of the exterior backfill to provide a lower permeability cap. Utility trenches should also be backfilled in lifts and compacted. Fine-grained soils require a sheepsfoot or padfoot roller. Final grading should provide protection from frost. Do not over-compact exterior backfills against “green” foundation walls.

6.7 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.

If interior slabs are placed over loess, an impermeable layer (usually plastic) is suggested beneath the slab. The slab shall be 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 curl as it dries. Three articles from the American Concrete Institute (ACI) that discuss these options are attached in Appendix C.

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, compactable, 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.

6.8 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. 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 as 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.

The silty loess soils may cause particularly severe frost damage. It may be reasonable to simply assume that exterior slabs will require periodic replacement as a maintenance item. exterior slabs bearing on the loess at this site may be improved by over-excavation and re-compaction of 2-ft of native material with tight moisture control (at least 95% ASTM D698 between -2 and +3% of optimum moisture) and seating the slab on at least 6 inches (preferably 12 inches) of road mix gravel. A lightweight filter fabric may be used to separate the gravel from silt, loess or organic topsoil. Alternatively, a more conservative, and probably more effective, approach would be to remove all of the organic topsoil and silty material from beneath the exterior slabs and replace with structural fill (sandy gravel pit run). 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 by 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.

6.9 Foundation or Basement Drains

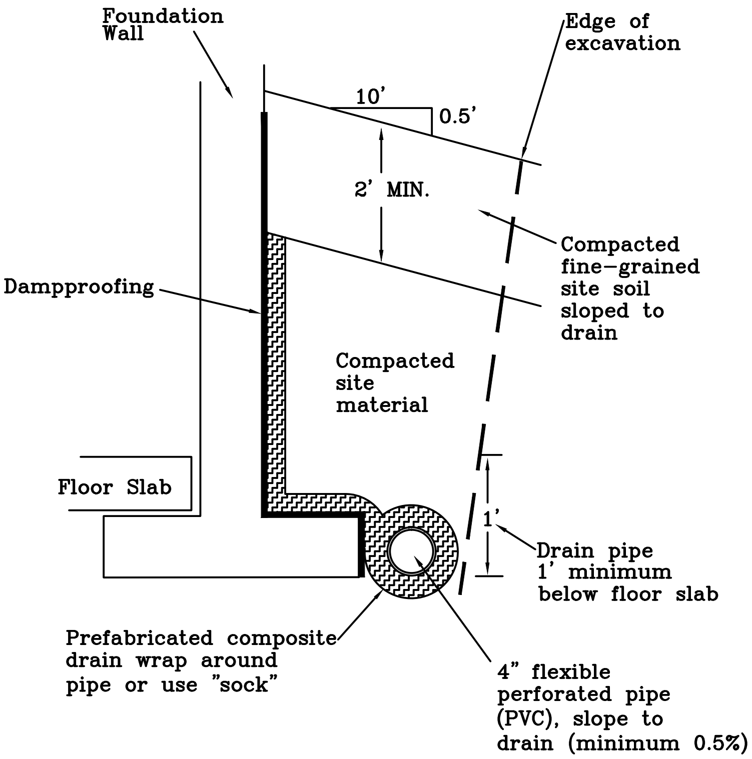
Foundation or basement drains are strongly recommended because loess drains poorly and tends to collect moisture. Two drainage alternatives are illustrated in Figure 5. One alternative is a prefabricated composite drain, which consists of an open wick layer laminated to filter fabric to reduce infiltration 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.

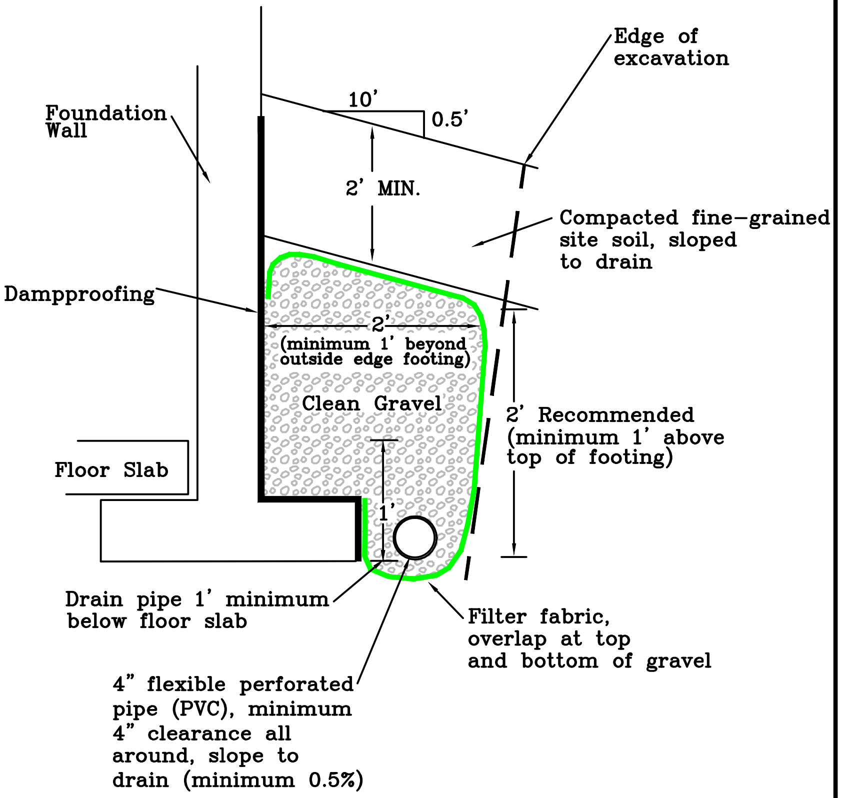
A second alternative involves placement of clean angular drain gravel or crushed stone between the foundation wall and the edge of the excavation. Drainage tiles, perforated pipe, or other approved systems should be installed at or below the area to be protected and should discharge by gravity or mechanical means into an approved drainage system. The drain pipe should 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.

UPDATE TIME: / / \chl / / \w\c\jh\ankeny\Fig 5 Found Drain

ALTERNATIVE #1:
USING PREFABRICATED COMPOSITE DRAIN



ALTERNATIVE #2:
USING GRAVEL DRAIN AND FILTER



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5370 Woodchuck Road
Lot 10 River Meadows Subdivision
Teton County, Wyoming

Foundation Drain
Detail

FIGURE
5

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. Review final plans to assure that everything drains properly. **Drains that are placed too shallow or with insufficient gradient may fail to perform.**

6.10 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.

6.11 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.

6.12 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.

7.0 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.

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APPENDIX A

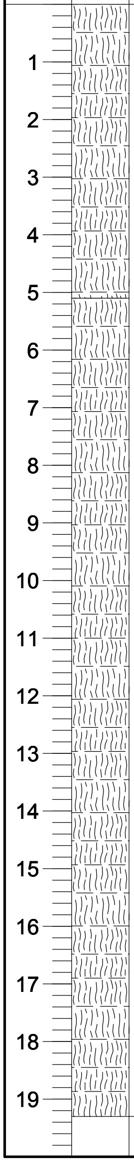
Test Pit Logs

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

TEST HOLE LOG

PAGE 1 OF 1

PROJECT NAME: Lot 10 River Meadows, 5370 Woodchuck
DATE: 4/28/15
PROJECT LOCATION: Teton County, Wyoming
HOLE NO.:   TP-1
TEST HOLE LOCATION: See site map
ELEVATION G.S. (ft.):
TOTAL DEPTH (ft.):   19.3
GROUNDWATER LEVEL (ft.):   NA
MEASURED FROM: Surface
DRILL TYPE: Hitachi Zaxis 160 LC
HAMMER:
DRILL CO: Fish Creek Excavation
DRILLER: Bill
LOGGED BY: chl



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
								0.0-5.1ft Clayey SILT: Very moist becoming dryer with depth, dark brown, very soft, massive, pinhole voids, some fine sand, roots to ~4.5' [LOESS] 2.4ft Becomes moist and soft					
								5.1-19.3ft Clayey SILT: Moist, buff/tan, soft to medium stiff with depth, massive, with pinhole voids [LOESS] Note: Stopped at limit of equipment. No groundwater observed at time of digging. Backfilled pit with spoils.					

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Jackson, WY 83002
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Fax: 307 733-5187

TEST HOLE LOG

PAGE 1 OF 1

PROJECT NAME: Lot 10 River Meadows, 5370 Woodchuck
DATE: 4/28/15
PROJECT LOCATION: Teton County, Wyoming
HOLE NO.:   TP-2
TEST HOLE LOCATION: See site map
ELEVATION G.S. (ft.):
TOTAL DEPTH (ft.):   17.8
GROUNDWATER LEVEL (ft.):   NA
MEASURED FROM: Surface
DRILL TYPE: Hitachi Zaxis 160 LC
HAMMER:
DRILL CO: Fish Creek Excavation
DRILLER: Bill
LOGGED BY: chl

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
1								0.0-17.8ft Clayey SILT: Very moist to moist, dark brown to buff/tan, very soft to medium stiff, massive, with pinhole voids, some fine sand [LOESS]					
2													
3													
4													
5													
6													
7													
8													
9													
10													
11													
12													
13													
14													
15								Note: Soil becomes drier, lighter in color and stiffer with depth. Stopped at limit of equipment. No groundwater observed at time of digging. Backfilled pit with spoils.					
16													
17													
18													
19													

APPENDIX B

Loess Article

Know More About Loess

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

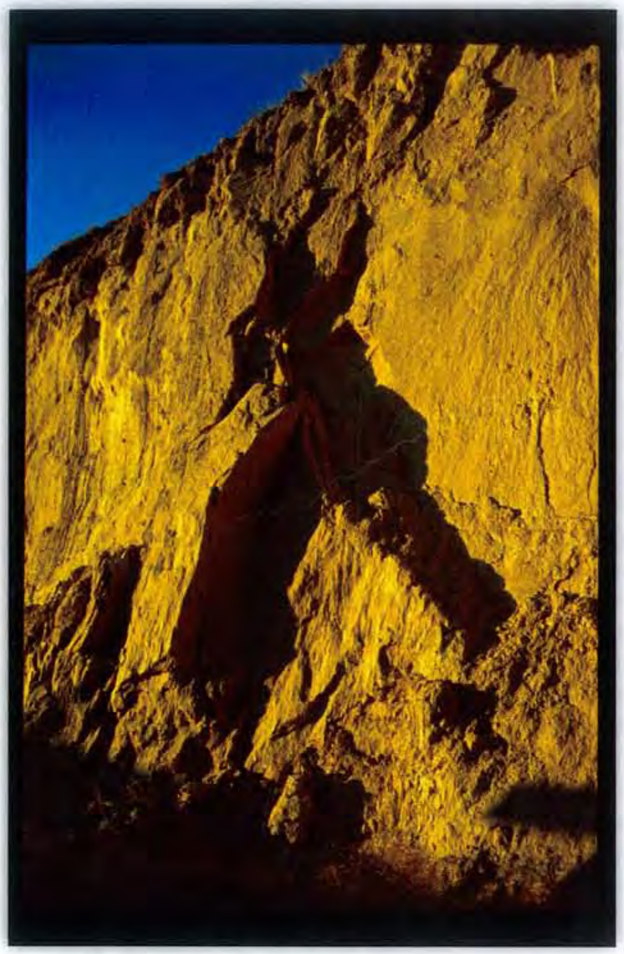


Figure 1. Near-vertical loess bluff face.

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.

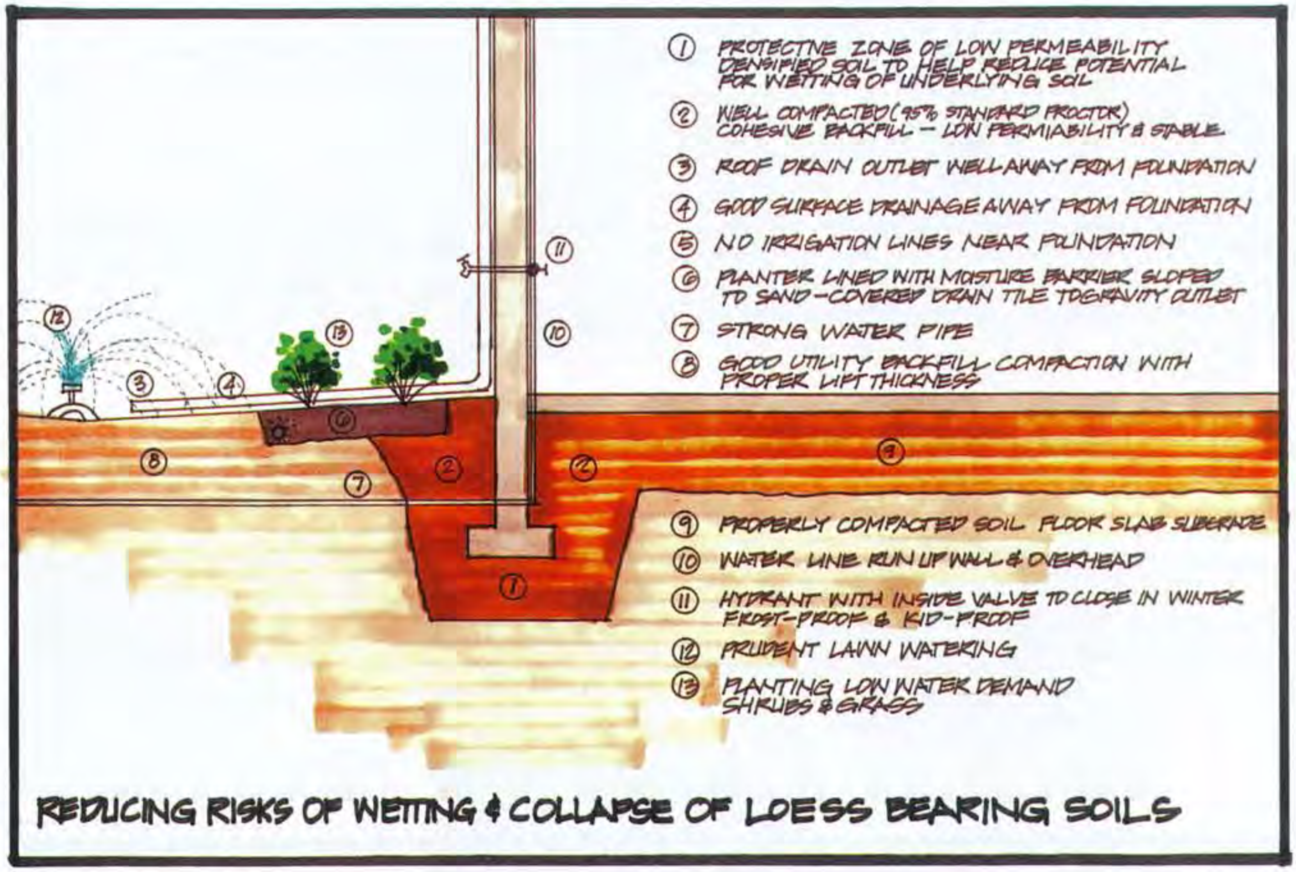


Figure 5. Common measures used to reduce wetting risks.

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

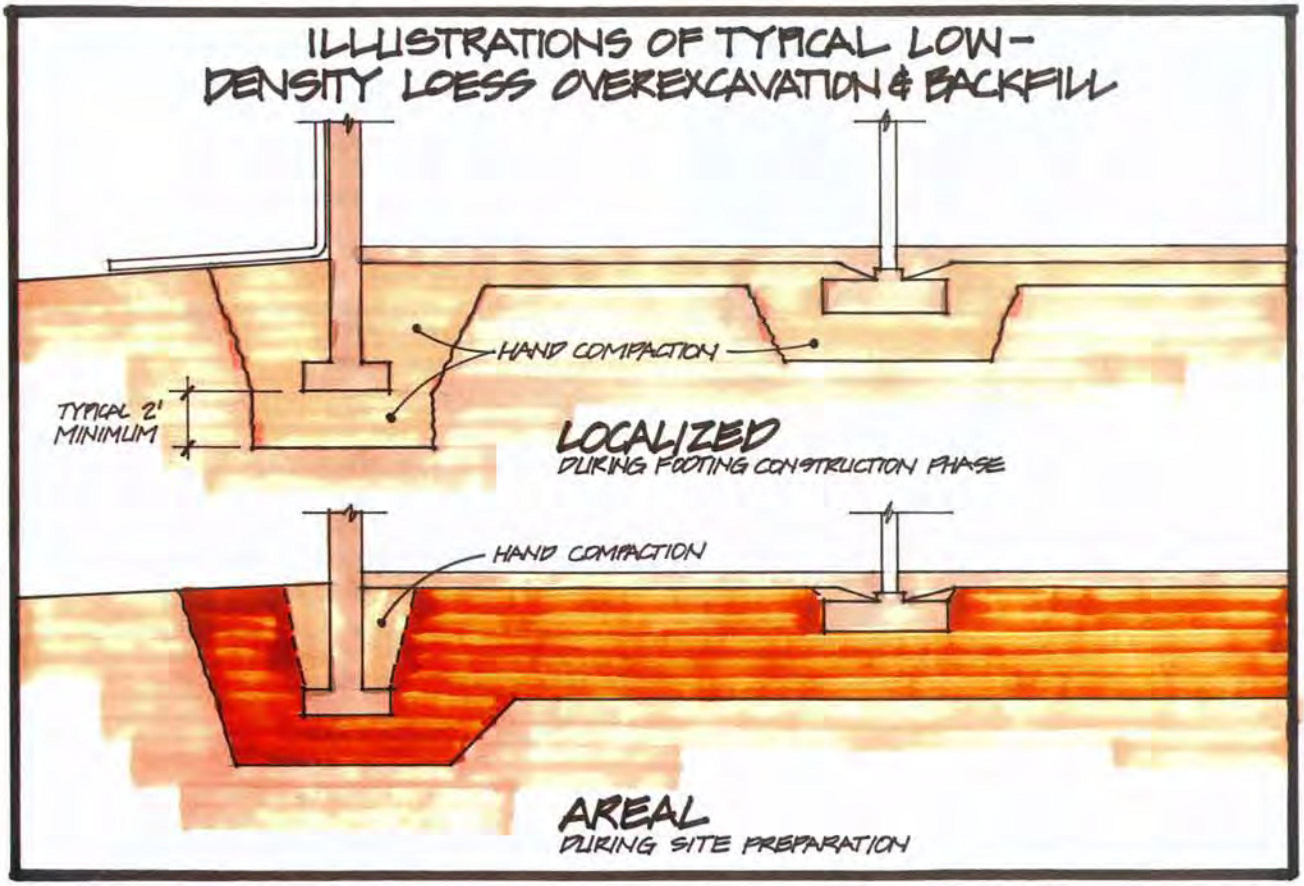


Figure 6. Cross-sections illustrating the partial excavation concept.

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.

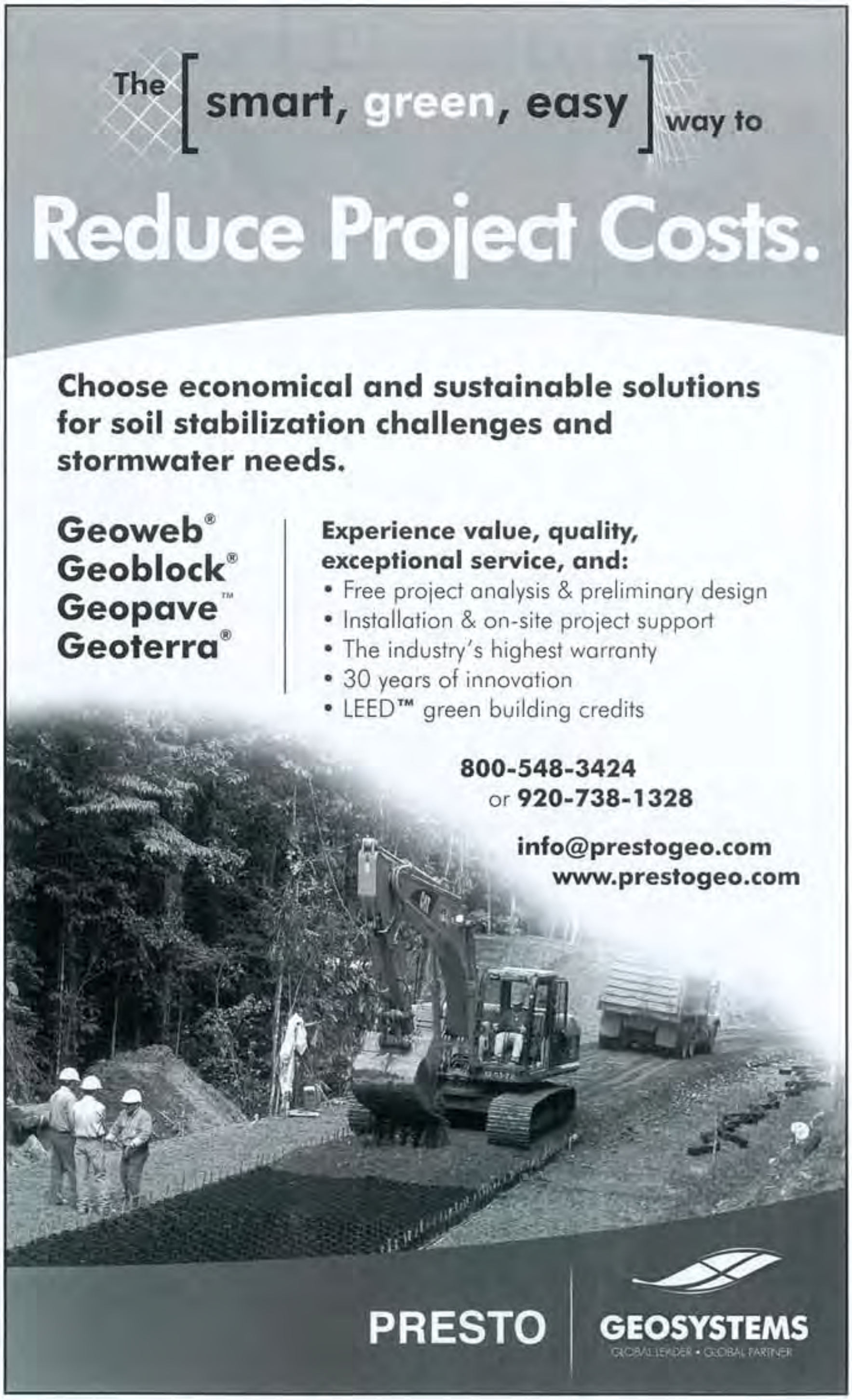
A more reliable method of reducing the risk posed by the collapsible soils is to derive support of the structure below the depth of the collapsible soils, or below the depth of anticipated wetting potential if the collapsible soils extend to a great depth. This solution is often impractical for light 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 C

Concrete 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.

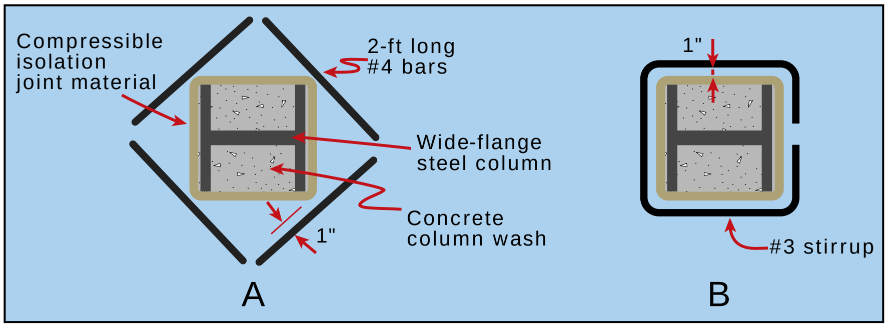
Rebar in concrete slabs placed directly on a vapor retarder help to control slab curling and cracking. Use supported deformed bars no smaller than #4, and space the bars far enough apart so workers can step between them.

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.

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

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.

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.

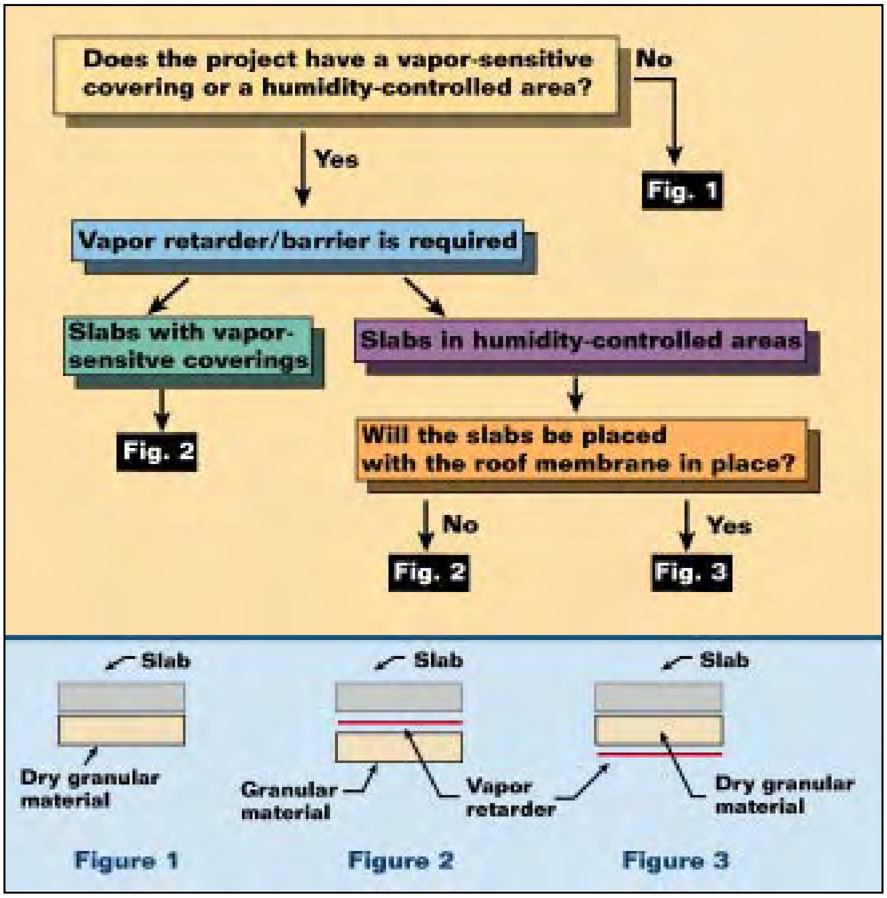


Figure 2. This flow chart helps designers decide if a vapor retarder or barrier is needed and where it should be placed.

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.*

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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

	Dry density (pcf)/moisture content (%)			Compaction test (standard Proctor)	Soil
Soil sample	No sand	1-in. sand	2-in. sand	Density/moisture	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-

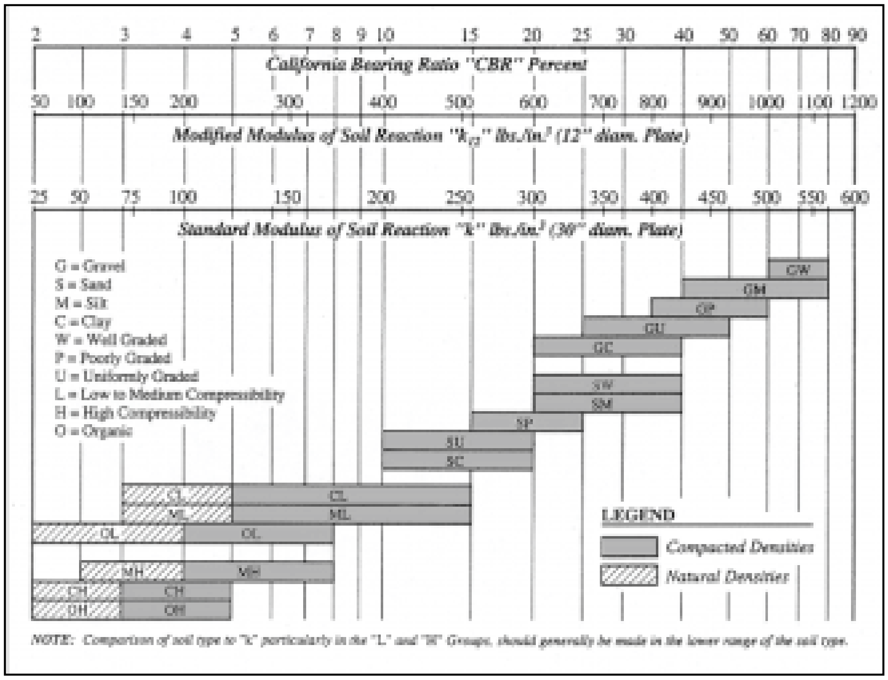

Figure 2. Interrelationships of CBR, k-values and soil classification (from Ref. 2).

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.

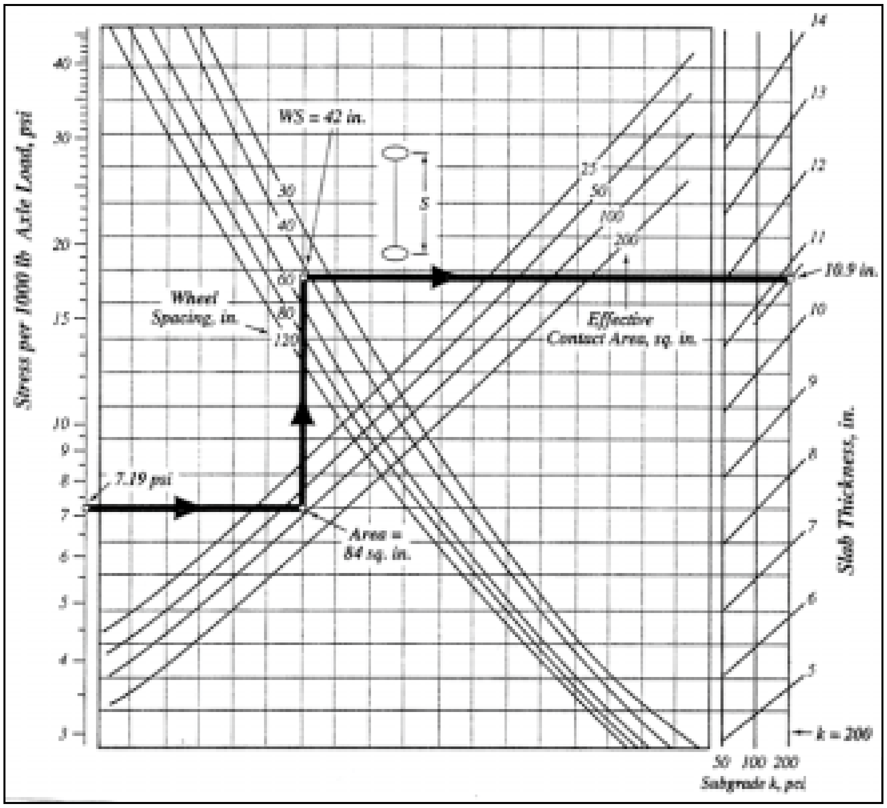

Figure 3. The example in this chart shows that decreasing the k-value from 200 to 50 increases the required slab thickness about an inch. For lighter loadings that yield a thinner slab, the same k-value reduction would still increase thickness about an inch.

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).

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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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