DONALD HERZOG & ASSOCIATES, INC.

Consulting Soil and Foundation Engineers

REPORT
SOIL INVESTIGATION
790 BOLINAS ROAD
MARIN COUNTY, CALIFORNIA

DH&A Job No. 656.2

Prepared for

Mr. Louis Hawkins
30 North San Pedro Road
San Rafael, California 94903

by

DONALD HERZOG & ASSOCIATES, INC.

Donald Herzog,
Principal Engineer

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P. O. Box 707 - Mill Valley, California 94941

Telephone (415) 383-5517

INTRODUCTION

This report presents the results of the soil investigation we performed for your planned eleven lot subdivision to be located on the north side of Pine Mountain Tunnel Road, immediately west of 790 Bolinas Road, in Marin County, California. We understand that the lots are to be used for single-family dwellings located immediately adjacent to Pine Mountain Tunnel Road. The existing roadway is to be improved and minor widening is anticipated.

We previously performed a preliminary soil and geologic investigation for the site, and presented the results in our report of October 3, 1977, and in our Consulting Engineering Geologist's report of September 30, 1977.

The purpose of our soil investigation was to review our previous work and to explore the subsurface conditions to the extent of 24 test holes to develop the following soil engineering information:

1. A description of the soil conditions observed.
2. Areas judged geotechnically feasible for residential development.
3. Site grading recommendations.
4. Recommended foundation type(s) and design criteria.
5. Retaining wall design criteria.
6. Recommendations for slab-on-grade construction.
7. Soil engineering drainage recommendations.

WORK PERFORMED

We reviewed the results of our previous preliminary soil and geologic investigation. We inspected the site with the owners, architect, and civil engineer, and discussed geotechnical construction constraints.

On September 22 and October 9, 1978, we explored the subsurface conditions at the site to the extent of 13 backhoe test pits and 11 auger borings. The test pits were excavated in areas accessible to a track-mounted backhoe. In other areas, it was necessary to use a portable power auger. The test holes were located as indicated on the attached Test Hole Location Plan, Plate 1. Our engineer located the test holes, inspected the excavating and drilling, logged the conditions encountered, and obtained soil samples for visual examination and classification. The Logs of the Test Holes are presented on Plates 2 through 8. The soils are described in accordance with the Unified Soil Classification System presented on Plate 9.

SITE CONDITIONS

The site is the northern flank of a large west-east trending ridge. Pine Tunnel Mountain Road is an unimproved road following the ridge north of Bolinas Road. Pine Tunnel Mountain Road was constructed by cutting on the uphill side and placing fill on the downhill side. The cut banks range to several tens of feet high and are as steep as about 1/2 horizontal to 1 vertical (1/2:1). Numerous small sloughs and slides were observed in the cut banks. One large landslide located on the uphill side of Pine Tunnel Mountain Road near the east end of the development has apparently caused the road to be relocated in the past.

The material excavated from the uphill side of the road was apparently placed as fill on the downhill side. The fill banks are quite steep (on the order of 1:1), and range to several tens of feet high. The roadway was reportedly constructed near the turn of the century. Roadway fills of that era generally were not keyed into firm material beneath the surface soils, nor were they compacted. In several areas, the roadway exhibits longitudinal cracks indicative of lateral fill yielding. However, we did not observe evidence of major fill instability during our investigation.

Below the roadway fill, swales and spur ridges slope down to a creek located near the north property line. The natural slopes vary from about 1½:1 to 3:1. This area is very densely overgrown with trees and brush.

The spur ridges generally appear to be stable. However, hummocks, scarps, and depressions indicative of landsliding and severe soil creep were observed in the swales and on steep side slopes.

The test holes indicate that the site is blanketed by topsoil and sandy clay colluvium (slopewash). The colluvium is weak, porous, and compressible and varies from a few feet thick on the spur ridges to several feet thick in the swales. Topsoil and colluvium on hillsides typically experience slow downhill creep (on the order of a fraction of an inch per year). The colluvium varies from sandy silt along spur ridges, to "fat" silty clay in the swales. In some areas, the colluvium contains slickensides indicative of landsliding.

Beneath the colluvium, the borings encountered highly variable sandstone and shale bedrock of the Franciscan Melange Formation. The rock varied from hard, strong, sandstone and graywacke to weak, highly sheared, and altered black and gray shale gouge.

Seepage was encountered in Test Pit 13, which was excavated in the large landslide area on the uphill side of the road. The remaining test holes did not encounter free water. However, our exploration was performed near the end of the summer dry season, and we anticipate that seepage and groundwater vary with seasonal rainfall.

CONCLUSIONS

Based upon the results of our investigation, we judge that the areas indicated on Plate 1 are suitable for residential construction from a geotechnical standpoint.

The areas not judged suitable for construction have one or more of the following problems: (1) excessive steepness; (2) active landsliding; (3) excessive thickness of creeping soils; or (4) very weak underlying bedrock offering inadequate lateral resistance.

The soils above the rock are weak and compressible when wet; are expansive; are experiencing slow downhill creep (on the order of a fraction of an inch per year); and are unsuitable for foundation support. It will be necessary to extend foundations through these soils and into the firm underlying rock. It will be necessary to design the foundations to resist lateral forces caused by downhill creep of the soils above the rock.

We judge that it will be most feasible to platform structures from the slopes. The structures may then be supported on drilled, cast-in-place, reinforced concrete piers extending through the soils and well into the firm underlying rock. However, where buildings are excavated through the soil and into the rock, spread footings can be used.

Where structures are located on slopes or within 10 feet of the toe of slopes, it will be necessary to install catchment walls on the uphill side of the buildings to provide protection from slough debris from above.

The existing road was constructed many years ago without proper engineering. The roadway fills were placed on steep slopes, probably without benefit of keyways or compaction. Both the fill and cut banks are much steeper than permitted by present day standards. However, although longitudinal cracks indicative of lateral yielding are evident in the fill portion of the roadway, and minor sloughing and sliding are evident in the cuts, the road is apparently performing satisfactorily as are numerous similar roads in Marin County. Reconstructing the road to current standards would require expensive and disfiguring large cut and fill embankments, as well as numerous high and expensive retaining walls. We judge that in addition to being esthetically unacceptable, such construction would make the project economically unfeasible. We judge that a more realistic alternative would be to leave as much of the present road as possible intact; restrict widening to the uphill side of the existing road; accept frequent repair of yielding and settling pavement; and accept removal of slough debris and repair of sloughs. Future maintenance can be reduced by locating the pavement edge back a few feet from the downhill side, and by installing catchment type walls to intercept slough in particularly poor areas on the uphill side of the road.

There is an inherent risk of instability associated with all hillside construction. We judge that residences constructed in accordance with this report will be stable, and that the risk of future instability will be within the range generally associated with construction on steep hillsides in Marin County. We anticipate that roadway cut and fill banks will experience sloughing requiring repair and clean up, and that patching of settlement and yielding will be required. If this is unacceptable, it will be necessary to reconstruct the road with keyed 2:1 fill embankments and 1½:1 and 2:1 cut banks.

We believe that there are no active faults at the site and, therefore, little risk of fault related ground rupture during earthquakes. Like the entire Marin County Area, the site is subject to severe ground shaking during earthquakes. It will be necessary to design and construct the project in strict accordance with current standards for earthquake resistant construction.

RECOMMENDATIONS

Site Grading

Areas to be developed should be cleared of vegetation and of the upper few inches of soil containing organic matter. The strippings should be removed or stockpiled for reuse as topsoil. Excavation can then be performed as necessary. We anticipate that with the exception of organic matter and of rocks or lumps larger than six inches in diameter, the excavated material will be suitable for reuse as compacted fill.

The amount of grading required will depend upon the roadway performance acceptable. If fill yielding and settlement with resultant maintenance are acceptable, the existing road fills may remain. If performance in accordance with current standards of practice is required, it will be necessary to overexcavate the existing roadway fill, and to flatten or retain the cut banks. Otherwise, grading should be restricted to the uphill side of the roadway and to areas indicated as suitable for development which are flatter than 3:1.

Areas to receive fill should be prepared by cutting level keyways extending into firm residual soil or rock. Where evidence of seepage is observed, and/or where fill is to extend beneath structures, subsurface drainage facilities should be installed at the rear of keyways as directed by the Soil Engineer.

The keyways should be prepared by scarifying to a depth of six inches, moisture conditioning as necessary, and compacting to at least 90 percent of the maximum dry density of the materials as determined by the ASTM D-1557-70(C) laboratory compaction test procedure. Fill material should then be spread in eight inch thick loose lifts, moisture conditioned as necessary, and compacted to at least 90 percent relative compaction. As the fill continues upslope, it should be continually keyed into firm soil or rock.

Roadway cut banks where sloughing is acceptable may be as steep as 1:1 in firm rock and 1½:1 in soft rock and soil. Other new cut and fill slopes should be no steeper than 2:1. Where steeper banks are required, retaining walls should be used. Slopes should be planted with fast growing, deep rooted ground cover to reduce sloughing or erosion.

Foundations

1. Spread Footings - Spread footings should only be used in areas excavated into firm rock. Spread footings should be at least 16 inches wide and should extend at least 18 inches into firm rock. The footings should be stepped as necessary to produce level tops and bottoms and should be deepened as necessary to provide at least eight feet of horizontal confinement between the footing bottoms and the face of the nearest slope.

Footings installed in accordance with these recommendations may impose dead loads, dead plus real live loads, and total loads of 2500, 3000, and 3500 pounds per square foot (psf), respectively.

2. Drilled Piers - Drilled, cast-in-place, reinforced concrete piers should be at least 18 inches in diameter and should extend at least eight feet into firm rock. The piers should be designed and reinforced to resist creep forces equivalent to an active equivalent fluid pressure of 50 pcf acting on two pier diameters. The thicknesses of the design creep zones are indicated on Plate 1. The piers should be interconnected with grade beams and tie beams to support building loads and to redistribute stresses imposed by the creeping soils.

The grade beams should be designed by the Project Engineer to support the imposed structural loads. Tie beams should be 12 inches square and should be reinforced with 2 #5 bars. Upslope-downslope grade beams and tie beams should be no more than 20 feet apart.

The portion of the piers extending into firm rock below the design creep zone may impose a passive equivalent fluid pressure of 200 pcf acting on two pier diameters, and vertical dead plus real live loads of 850 psf in skin friction. End bearing should be neglected because of the difficulty of cleaning out small diameter pier holes and the uncertainty of mobilizing end bearing and skin friction simultaneously.

If ground water is encountered, it may be necessary to dewater the holes and/or to place the concrete by the tremmie method. If caving soils are encountered, it may be necessary to case the holes. Hard drilling may be required to achieve the required penetration.

Retaining Walls

Where buildings are located on slopes or within 10 feet of the toe of slopes, a retaining wall on the uphill side of the building should extend at least 24 inches above the backfill to provide protection from slough debris. Retaining walls supporting level backfill should be designed to resist an active equivalent fluid pressure of 40 pounds per cubic foot (pcf) acting in a triangular pressure distribution. Where the backfill slopes up steeper than 3:1, the walls should be designed for an active equivalent fluid pressure of 55 pcf. Where retaining wall backfill is subject to truck vehicular traffic, the walls should be designed to resist an additional surcharge pressure equivalent to 2 feet of additional backfill. Retaining wall heights should be calculated from the top of catchment areas.

The portion of retaining wall foundations extending into firm rock at least seven horizontal feet from the face of the nearest slope, may impose a passive equivalent fluid pressure and a friction factor of 400 pcf and 0.45 respectively to resist sliding. Passive pressure on piers may be assumed to act on two pier diameters.

Retaining walls should be fully backdrained. The backdrains should consist of four inch diameter perforated pipe sloped to drain to outlets by gravity and of clean, free draining crushed rock or gravel. The crushed rock or gravel should extend to within one foot of the surface. The upper one foot should be backfilled with compacted soil to exclude surface water. The ground surface behind retaining walls should be sloped to drain.

Where migration of moisture through retaining walls would be detrimental, retaining walls should be waterproofed.

Slab-on-Grade

Slab-on-grade subgrade should be rolled to produce a dense, uniform surface. The slabs should be underlain with a capillary moisture break consisting of at least four inches of clean, free draining crushed rock or gravel at least 1/4 inch and no larger than 3/4 inch in size. Where migration of moisture vapor through slabs would be detrimental, an impermeable membrane moisture vapor barrier should be provided between the drainrock and the slabs. Slabs should be reinforced to reduce cracking.

The future expansion potential of the subgrade soils should be reduced by thoroughly presoaking the slab subgrade prior to concrete placement.

An outlet should be provided from the slab drainrock.

Soil Engineering Drainage

Surface water should be diverted away from slopes and foundations.

Roofs should be provided with gutters and the downspouts should be connected to closed conduits discharging well away from foundations and slopes.

Foundation drains should be provided adjacent to all perimeter foundations except the downhill side. Foundation drains should consist of trenches at least 18 inches deep and sloped to drain by gravity. Three inch diameter perforated pipe sloped to drain to outlets by gravity should be placed in the bottom of the trenches. The trenches should be back-filled to within six inches of the surface with clean, free draining crushed rock or gravel. The upper six inches should be backfilled with compacted soil to exclude surface water. The ground surface should be sloped to drain away from foundations.

Where retaining walls are used for perimeter foundations, retaining wall backdrains may be used in lieu of foundation drains.

Roof downspouts and surface drains must be maintained entirely separate from foundation drains and retaining wall backdrains.

LIMITATIONS

Subsurface conditions are complex and may differ from those indicated by surface features and those encountered at the test hole locations. Therefore, we are unable to guarantee the performance of any site or foundation system. For houses constructed on hillsides, we recommend that mudflow and earthquake insurance be obtained where available.

If conditions different from those described in this report are encountered during construction, we should be notified immediately, so that we may modify our recommendations if warranted.

SUPPLEMENTAL SERVICES

We should review the final plans for conformance with the intent of our recommendations. During construction, we should inspect construction excavations and pier drilling operations to observe the conditions encountered and to modify our recommendations, if warranted. We should also be notified to provide inspection and testing during fill placement operations to ascertain that the specified compaction is attained.

LIST OF PLATES

LOG OF TEST HOLES

	Depth	Condition
Log of Test Pit 1		
	0 '- 4½'	RED BROWN CLAYEY SILT (ML) dry, medium stiff, friable
	4½'-10 '+	SANDSTONE low strength, low hardness, highly fractured, deeply weathered, coarse grained
Log of Test Pit 2		
	0 '- 4 '	BROWN SANDY SILT (ML) medium stiff, dry, with abundant rock fragments
	4 '-10 '+	META-SHALE low hardness, low strength, deeply weathered, highly fractured, with sandstone and semi-rounded graywacke boulders in meta-shale matrix.
Log of Test Pit 3		
	0 '- 1 '	DARK BROWN SANDY SILT (ML) soft, dry, porous topsoil
	1 '- 3 '	RED BROWN CLAYEY SILT (ML) medium stiff, wet
	3 '- 8 '	GRAY GREEN SILTY CLAY (CL) soft, saturated, with landslide slickensides
	8 '-10 '+	GRAYWACKE hard, strong, little weathered

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Depth	Condition

Log of Test Pit 4

0 '- 1 '	DARK BROWN SANDY SILT (ML) medium stiff, dry, porous topsoil
1 '- 2 '	GRAY BROWN GRAVELLY SILT (ML) medium stiff, moist
2 '-10 '	GRAY CLAY (CL) stiff, wet, with landslide slickensides
10 '-12 '+	META-CHERT moderately strong, moderately hard, highly fractured, deeply weathered with abundant quartz.

Log of Test Pit 5

0 '- 1½'	DARK BROWN SANDY SILT (ML) medium stiff, dry, porous topsoil
1½'- 5 '	BROWN SANDY SILT (ML) medium stiff, dry, with abundant rock fragments
5 '- 5½'	GRAY BROWN CLAY (CL) medium stiff, wet
5½'-11 '+	BROWN SANDSTONE low strength, low hardness, highly fractured, deeply weathered, with thick lenses of dessicated clay

Log of Test Pit 6

0 '- 1½'	DARK BROWN SANDY SILT (ML) medium stiff, dry, porous topsoil
1½'- 5 '	BROWN SILT (ML) stiff, moist
5 '- 5½'	GRAY BROWN SILTY CLAY (CL) stiff, moist
5½'-11'+	BROWN SANDSTONE low strength, low hardness, deeply weathered, highly fractured

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LOG OF TEST HOLES
790 Bolinas Road
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PLATE 3

Depth	Condition

Log of Test Pit 7

0 '- 3 '	BROWN SANDY SILT (ML) medium stiff, dry, with abundant rock fragments
3 '- 6 '	BROWN SANDSTONE very weak, low hardness, deeply weathered, large grained, highly fractured
6 '- 8 '	RED SILTY CLAY (CL) very stiff, moist
8 '-10 '+	RED SANDSTONE AND SHALE moderately, strong, moderately hard, deeply weathered, little fractured

Log of Test Pit 8

0 '- 1½'	BROWN SANDY SILT (ML) medium stiff, dry, with abundant rock fragments
1½'- 6 '+	BROWN SHALE moderately strong, moderately hard, moderately weathered, highly fractured

Log of Test Pit 9

0 '- 1½'	BROWN SANDY SILT (ML) medium stiff, dry, with abundant rock fragments
1½'- 5 '+	BROWN SHALE moderately strong, moderately hard, moderately weathered, highly fractured

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Depth	Condition
Log of Test Pit 10	
0 '- 3 '	BROWN SANDY SILT (ML) medium stiff, dry, porous topsoil
3 '- 6 '	DARK GRAY SILTY CLAY (CL) very stiff, moist, dessicated
6 '- 9 '+	GRAY SHALE weak, soft, deeply weathered, highly altered, highly fractured, with sandstone and talc
Log of Test Pit 11	
0 '- 1 '	DARK BROWN SANDY SILT (ML) medium stiff, dry, porous topsoil
1 '- 4 '	BROWN SANDY SILT (ML) medium stiff, moist, with abundant rock fragments
4 '-11 '+	BROWN SANDSTONE AND SHALE low strength, soft, deeply weathered, with abundant silt, coarse grained
Log of Test Pit 12	
0 '- 3 '	BROWN SANDY SILT (ML) medium stiff, moist, with rock fragments
3 '-10 '+	BROWN SANDSTONE AND SHALE low strength, low hardness, deeply weathered, highly fractured, with silt
Log of Test Pit 13	
0 '- 7 '	DARK BROWN SANDY SILT (ML) soft, moist, porous topsoil
7 '-12 '+	MOTTLED ORANGE BROWN SANDY SILT (ML) soft, wet (saturated below 9')

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Depth	Condition
Log of Test Boring 14	
0 '- 7 '	GRAY SILTY CLAY (CL) medium stiff, moist, with rock fragments
7 '-10 '	BROWN SANDY SILT (ML) medium stiff, moist, with rock fragments
10'	GRAYWACKE very strong, very hard, little weathered
Log of Test Boring 15	
0 '- 2 '	LIGHT BROWN SANDY SILT (ML) medium stiff, dry, porous
2 '- 6 '+	BROWN SANDSTONE moderately strong, moderately hard, fractured, with moderate amounts of silt
Log of Test Boring 16	
0 '- 2 '	LIGHT BROWN SANDY SILT (ML) medium stiff, dry, porous topsoil
2 '- 3 '	BROWN SANDY SILT (ML) medium stiff, dry
3 '- 9 '+	BROWN SANDSTONE low strength, soft, highly fractured with silt, becoming fine below 6'
Log of Test Boring 17	
0 '- 2 '	BROWN SANDY SILT (ML) medium stiff, dry, porous topsoil with rock fragments
2 '- 4 '+	BROWN SANDSTONE AND SHALE moderately strong, moderately hard, little fractured, moderately weathered, dry

Depth	Condition

Log of Test Boring 18

0 '- 3 '	BROWN SANDY SILT (ML) medium stiff, dry, porous topsoil
3 '- 6 '+	BROWN SANDSTONE AND SHALE low strength, soft, highly fractured, deeply weathered

Log of Test Boring 19

0 '- 2 '	GRAY SANDY SILT (ML) medium stiff, dry, porous with shale fragments
2'+	GRAYWACKE strong, hard, little weathered

Log of Test Boring 20

0 '- 1 '	FRACTURED ROCK RUBBLE
1'+	BROWN SANDSTONE strong, hard, little weathered Refusal at 3'

Log of Test Boring 21

0 '- 5 '	GRAY CLAY (CL) very stiff, moist
5 '- 6 '+	GRAYWACKE strong, hard, little weathered, little fractured

Log of Test Boring 22

0 '- 5 '	DARK GRAY SILT CLAY (CL) medium stiff, moist
5 '-	Refusal on probable boulder (Three additional holes also encountered refusal on boulders at varying depths to 5')

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LOG OF TEST HOLES
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PLATE 7

Depth	Condition

Log of Test Boring 23

0 '- 3 '	SILTY SANDSTONE RUBBLE
3 '- 9 '+	BROWN SANDSTONE AND SHALE low strength, moderately hard, deeply weathered, very highly fractured

Log of Test Boring 24

0 '- 3 '	BROWN SANDY SILT (ML) medium stiff, moist
3 '- 5 '	BROWN SANDSTONE moderately strong, moderately hard, deeply weathered, highly fractured

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LOG OF TEST HOLES
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PLATE 8

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS MORE THAN HALF IS LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL - SAND MIXTURES
			GP	POORLY GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, POORLY GRADED GRAVEL - SAND - SILT MIXTURES
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL - SAND - CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
			SP	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, POORLY GRADED SAND - SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN HALF IS SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS

UNIFIED SOIL CLASSIFICATION SYSTEM

Shear Strength, psf
Confining Pressure, psf

Consol	Consolidation	*Tx	320 (2600)		Unconsolidated Undrained Triaxial
LL	Liquid Limit (in %)	TxCU	320 (2600)		Consolidated Undrained Triaxial
PL	Plastic Limit (in %)	DS	2750 (2000)		Consolidated Drained Direct Shear
G_s	Specific Gravity	FVS	470		Field Vane Shear
SA	Sieve Analysis	*UC	2000		Unconfined Compression
■	"Undisturbed" Sample	LVS	700		Laboratory Vane Shear
⊠	Bulk Sample				

Notes: (1) All strength tests on 2.8" or 2.4" diameter samples unless otherwise indicated.

(2) * Indicates 1.4" diameter sample.

KEY TO TEST DATA

DONALD HERZOG & ASSOCIATES Consulting Soil & Foundation Engineers	SOIL CLASSIFICATION CHART AND KEY TO TEST DATA	PLATE
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