

CTC-GEOTEK, INC

ENGINEERING TESTING INSPECTION

SOIL AND FOUNDATION INVESTIGATION

IBS Building
Southeast Corner of 15th Street & Grandview Avenue
Boulder, Colorado

PREPARED FOR:

University of Colorado
Dept. of Facilities Management – Office of Planning, Design & Construction
Campus Box 453 UCB
1540 30th Street, 3rd Floor
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Attention: Mr. Larry Krook

Project 282014 May 12, 2008

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SUMMARY

- (1) The subsurface conditions at the site are somewhat variable. Our borings generally encountered up to 6 feet of existing fill over various, generally granular, soil mixtures and 1 to 4 feet of severely weathered claystone. Bedrock was initially encountered at depths of 7 to 12 feet with shale at depths of 26 to 29. Present groundwater levels are at depths of deeper than 20 feet.
- (2) In our opinion, a straight shaft pier (caisson) foundation system is recommended for the support of the building structure at the site. The piers should be drilled at least 6 feet into bedrock and have shaft lengths of at least 18 feet. Footings are discussed for features, such as site retaining walls, that are not connected to the building structure
- (3) Slab-on-grade construction on new structural fill is generally suitable as discussed. At least some removal of existing fill must occur below slabs and pavements. Groundwater levels are suitable for the proposed below-grade level.
- (4) A representative from our firm should observe the construction operations discussed in this report.

SCOPE OF STUDY

This report represents the results of a soil and foundation investigation at the site of the proposed new IBS building on the University of Colorado campus at the southeast corner of 15th Street and Grandview Avenue in Boulder, Colorado.

The purpose of this study was to explore the subsurface conditions, obtain some data of the pertinent engineering characteristics of the underlying strata, recommend the most appropriate foundation system, develop foundation design criteria, attempt to evaluate the risks of slab-on-grade construction, provide pavement requirements, and address other geotechnical factors in the proposed development.

It should be understood that economic and practical constraints limit our sampling and laboratory testing to only a minuscule fraction of the total mass of soil and/or bedrock which lies within the zone of influence of the proposed structure.

Our analyses, conclusions and recommendations are based upon the assumption that the samples of subsurface strata, which we observed and tested, are representative of the entire soil mass.

PROPOSED CONSTRUCTION

As we understand, the proposed development on the site will consist of a four-story building with a partial basement at the northern end. The maximum column loads foundation loads are expected to be on the order of 900 kips with 450 kip dead-load.

In addition to the structure, various site development (pavements, utilities, etc.) measures will be undertaken on the site.

FIELD INVESTIGATION

Six (6) exploratory test borings were drilled on the site, at the locations shown on Plate 1. The borings were drilled with 4-inch diameter, continuous flight, solid-stem power augers using a truck-mounted drill rig.

At regular intervals the drilling tools were removed from the boreholes and soil samples were obtained with a 2-inch I.D. California Spoon Sampler. The sampler was driven into the various subsoil strata with blows of a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler one foot, or a fraction thereof, constitutes the penetration test. This field test is similar to the standard penetration test described by ASTM Method D-1586. Penetration resistance values, when properly evaluated, are an index to the soil strength and density. The depths at which the samples were taken and the penetration resistance values from the borings are shown on the Logs of Exploratory Borings, Plates 2 and 3.

LABORATORY TESTING

All samples were carefully inspected and classified in the laboratory by the project engineer. Natural water contents, dry unit weights, Atterburg limits, partial gradations (percents passing the U.S. No. 200 sieve), and percents water soluble sulfates were obtained from relatively undisturbed drive samples of typical materials encountered (see Table 1).

Swell-consolidation tests were performed on typical specimens of potentially swelling and/or consolidating material (see Plate 5 through 7). This is to indicate the behavior of these materials upon wetting and loading.

SUBSURFACE CONDITIONS

The initial soils at the site consist of 1 to 6 feet of existing fill consisting of various sand, gravel, clay and silt mixtures. These materials are generally medium moist to moist and moderately compact to compact. These materials appear to exhibit no more than a very low swell potential.

Below the fill, the natural soils consist of various mixtures and layers of sand, clay and silt which are occasionally gravelly. These soils are medium moist to moist. The sands are non-expansive but clay layers would possess swell potentials.

Claystone bedrock was initially encountered in the borings at depths of 7 to 12 feet. The initial bedrock is somewhat weathered and is generally medium hard to very hard. The claystone possesses a low to moderate swell potential. The bedrock is a very hard "Denver Blue Shale" below depths of 26 to 29 feet.

The present groundwater levels are at depths below 20 feet. It should be noted that groundwater levels can vary with changes in precipitation, irrigation, drainage and land use.

FOUNDATIONS

It is recommended that building structure at the site be founded on straight shaft piers (caissons) drilled at least 6 feet into the bedrock strata and in all cases at least 18 feet in total length. The 18-foot minimum length would be considered from any basement elevation when piers are directly adjacent to basement walls or wall backfill areas.

Using the drilled pier type of foundation, each column is supported on a single drilled pier or building walls are supported on a grade beam founded on a series of drilled piers. Load applied to a pier of this type is transmitted to the bedrock, partially through peripheral shear stresses, which develop on the sides of the pier, and partially through end bearing pressure.

Design values provided are based on the field and laboratory test results and the supporting capacity of the average of the softer materials encountered. For the portion of the pier in bedrock, and below the upper 3 feet of the pier shaft (i.e. bedrock penetration shall not be counted within 3 feet of the pier top), we recommend the maximum allowable end bearing and compressive side shear values shown in Table A. Estimated pier settlements at the site are less than 1/2 inch.

Table A - Pier Design Values

Depth Into Bedrock (Feet)	End Bearing Pressure (PSF)	Compressive Side Shear (PSF)
0 - 6	35,000	2,500
6 - 16	35,000	4,000
16 +	50,000	5,000

A design value of 60 percent of the compressive side shear may be used for the tension shear. Pier end bearing and side shear capacities may be increased by one-third for short term transient loads such as wind and seismic forces.

A minimum dead-load of 20,000 psf times the piers cross sectional area should be utilized. If the minimum dead-load criteria cannot be met, additional bedrock penetration utilizing the tension shear should be used along with additional steel reinforcing.

A minimum pier diameter of 18 inches should be used. The use of temporary casing during drilling will generally be necessary. The length to diameter ratio of the piers should not exceed 30. Piers should be spaced at least two diameters, edge-to-edge, apart. If closer piers must be used, design pressures will need to be adjusted. The allowable design pressures would be a linear relationship from 100 percent at two diameters apart down to 75 (end bearing) and 67 percent (side shear) at no diameters apart, that is with

edges touching. If two nearby piers are of different diameters, the spacing ratio should be determined based on the smaller diameter of the two.

Lateral pier design parameters are horizontal modulus of subgrade reaction values of 40 pci (soil and fill) and 300 pci (bedrock). The modulus values are based on a pier diameter of 1 foot. Values used should be the proceeding divided by the actual pier diameter in feet. Lateral pier design parameters based on allowable passive equivalent fluid density values of 160 pcf and 500 pcf may be used for the upper soil material and the bedrock, respectively. Other pier design and 'L Pile' parameters are provided in Table A.

Table B - Soil Strength Design Parameters

Soil Desc.	Soil Type	Dry Density (lb/in ³)	Average Undrained Shear Strength (lb/in ²)	Average Friction Angle (Deg)	Strain @ 50% Max. Strength	Modulus of Subgrade Reaction (lb/in ³)
Upper Soils	Varies, generally SM, SC, or SP	0.062	8.0	20.0	0.015	40
Bedrock	CL	0.065	75.0	0	0.004	300

For lateral load conditions, the following reduction factors, to the trailing pier in the direction of loading, due to group action should be used in the design.

Table C – Lateral Load Reduction Parameters

Pier Spacing (D – pier diameter)	Reduction (%)
0.6D	80
0.8D	65
1D	50
2D	35
3D	20
4D	16
5D	12
6D	8
7D	4
8D	0

Piers are recommended for support of building related construction. However, with the assumption of some risk of movement, features such as site retaining walls that are not directly related to the building, could be supported on shallow footing foundations. The footings should be placed on at least 1 foot of newly placed structural fill. These foundations would be designed using a maximum allowable soil bearing pressure of 2,500 psf and as high of a dead-load pressure possible. Potential movements would be settlement of up to 3/4 inch and a maximum heave on the order of 1 to 2-1/2 inches. If the underlying soils were not wetted in the future, the actual movements would be very small. The potential maximum heave is dependent upon the depth of swelling material and it's possible expansion. Any existing fill in footing areas must be removed from below the footing foundations. Footing excavations should be carefully observed. Any particularly poor subgrade material would need to be removed and replaced from below footings. Any clayey soils exposed in footing excavations or overexcavations should be kept moist during construction by occasional sprinkling.

The use of frost depth of 3 feet is appropriate in the foundation design. However, grade beams with a void below them are not subject to frost heave and thus only need a depth that is structurally appropriate.

The soils at the site are not particularly prone to liquefaction. Site conditions do not require the use of structural ties between individual foundation elements. The site specific design spectral response acceleration parameters at short periods, S_{DS} , and 1-second period, S_{D1} , are 0.213 and 0.096, respectively.

Geophysical (seismic) work was done on the site. That work was done by Zonge Geosciences (report previously provided). An average shear wave velocity of 1,959 ft/sec was found during that study. That value allows the use of Site Class C in IBC 2006 seismic analysis and design.

FLOOR SLAB CONSTRUCTION

The site is suitable for slab-on-grade construction assuming the owner can accept the risk of some potential floor slab movements. It is recommended that the following measures be taken to help minimize, but not eliminate, movement should wetting occur:

A) Floor slabs should be placed on a total of at least 4 feet of new fill, including at the top, at least 12 inches of inherently non-swelling relatively impervious soils, such as silty sand or “crusher reject”. The material should be approved by the geotechnical engineer prior to its placement. In any case where slabs overlie existing fill, at least 2 feet, and possibly more if the fill is particularly poor, of the fill must be removed and replaced. Ideally all existing fill would be removed and should be if the owner desires maximum risk reduction. The new fill should be compacted to 95 percent of the maximum Standard

Proctor density (per ASTM D-698) at a moisture content sufficient to minimize swell potential. The purpose of the upper sand fill is to insulate any underlying clays and thus prevent their drying after placement and prior to slab pouring. At the basement level, which will be at or close to the more highly swelling claystone, it is recommended to place 6 feet of fill, the minimum 12 inches of select non-swelling fill plus 5 feet of reworked on-site material, below the floor. The earthwork section of this report provides additional fill placement criteria.

B) Generally separate slabs from bearing members to allow their independent movement. However, slabs could be connected to foundation elements where significant structural benefits occur. Joints (construction joints/saw cuts) should be provided in the slabs at no greater than the maximum spacing of ACI requirements.

C) Place a minimum 1-1/2 inch "void" above, or preferably below non-bearing partitions in slab-on-grade areas. Doorjamb, drywall, heating and cooling equipment, etc., should be similarly protected.

D) Keep any exposed clays or claystones moist during construction by occasional sprinkling.

E) No irrigation should occur for a distance of 5 feet beyond the building limits. Those areas may be covered with decorative gravel or artificial lawn, or preferably pavement. All exterior joints (building-sidewalk, curb pavement, etc.) should be well sealed. Roof downspouts should discharge on splashblocks, downspout extensions, or pavements to beyond the limits of the foundation backfill but not less than 6 feet from the buildings.

E) A polyethylene moisture barrier is desirable below any slabs that are to receive relatively impermeable floor coverings. This moisture barrier would be placed shortly before concrete placement in a manner consistent with the recommendations of ASTM E-1643. Areas without relatively impermeable floor coverings would not need a moisture barrier.

F) The floor slabs can be designed using a modulus of subgrade reaction value of 150 pounds per cubic inch.

G) Sewer lines beneath the structure should have a good slope (minimum 1-1/2 percent). All utility lines should be provided with flexible joints or oversized sleeves where they enter the building to prevent breakage caused by differential movement. Waterlines within the building should, as much as possible, be overhead rather than below the slab. All utility lines throughout the site should be carefully leak tested, in order to minimize future wetting of the underlying clays and claystone once construction is complete.

The preceding slab-on-grade precautions, with the new structural fill, will generally limit potential movements to less than 3/4 inch. However, greater movements, on the order of up to 1-1/2 inches could be possible in some isolated locations, particularly at the basement level. If the owner cannot accept the risks of those potential movements, structurally supported slabs over at least an 8-inch "void" or crawl space should be used.

It should be noted that these floor slab comments and recommendations would also apply to exterior slabs, particularly at critical areas such as attached sidewalks or entryways.

EARTHWORK

We recommend that permanent cut and fill slopes generally be no steeper than 2 (horizontal) to 1 (vertical). Steeper slopes may be suitable but will need to be individually considered. Slopes will need to be protected against erosion. Vegetation, benched timber walls, rock walls, rip-rap, etc. would all be suitable measures.

Existing fills on the site will need some reworking below new footings, slabs and pavements. Below any footing foundations, all fill should be removed. Below slabs and pavements, at least some and possibly all prior fills should be removed and any remaining fill observed for suitability during earthwork operations. Any particularly poor pockets noted would require deeper overexcavation. If any existing organic and/or concentrated or large rubble fill is encountered it must be removed and cannot be reused on-site.

Structural fill can generally consist of either inherently non-swelling material such as sands or materials which can be placed and maintained in such manner that their swell potential is minimized. The sands at the site are of the first type while the clayey on-site materials are of the later type. Any imported material should be approved prior to its use. Note that only granular (sand) fill should be used in the upper 1-foot below any footings and floor slabs. Reuse of claystone as fill in building areas and in the upper 4 feet of pavement areas should be avoided.

Any select import fill materials for use below slabs or footings should have a liquid limit of less than 30, a plasticity index of less than 10, a maximum particle size of 2 inches, and a percent passing the U.S. No. 200 sieve of between 10 and 30 percent.

Structural fill should be compacted to at least 95 percent of the maximum Standard Proctor density (per ASTM D-698) at a moisture content appropriate for the particular

material. We would expect that any on-site clay would require a moisture content on the order of from 2 percent below the optimum to 2 percent above in order to minimize swell potentials. **The specific minimum moisture content of each on-site material encountered will be determined by the geotechnical engineer during construction.** The specific minimum moisture content for each clay material would be that at which a maximum swell of 1 percent occurred under a 150-psf loading. The swell tests would be run as each proctor test was performed. The necessary moisture content of any imported material would be determined at the time of approval. The moisture content of essentially granular material such as sand would not be critical.

Please note that any on-site clays could exhibit higher swell potentials if allowed to become drier. This must not be allowed to occur during the construction period below any foundations and slabs-on-grade. Otherwise, the potential heaves could increase.

Fill in landscaped areas may be placed at a minimum of 90 percent of the maximum Standard Proctor density at a moisture content of within 3 percent of the optimum.

Typically it is expected that the existing fill materials will be suitable for reuse as new general fill material, but perhaps not as granular fill. However, any organics, trash and any large (greater than 12 inches) rubble should not be reused at all. Rubble larger than 4 inches should not be reused within 3 feet of the surface.

An earthwork shrinkage factor on the order of 10 to 13 percent is suggested.

Conventional equipment, including ripping, would be expected to typically accomplish excavation to the depths expected of most of the bedrock encountered. However, it can

not be ruled out that some, small, isolated lenses of very to extremely hard sandstone could require additional effort to remove and/or drill through.

GROUNDWATER

Present groundwater conditions are generally favorable for a below-grade level, from existing grades. However, a water table rise and/or new, higher perched water tables could occur on top of less permeable subsurface layers in the future. A precautionary perimeter subsurface drainage system suitably connected to a sump or other acceptable outlet must therefore be provided around (wall drainage board and/or gravel “chimney”) and at least 12 inches below the slab level at the perimeter (perforated pipe and gravel) of any underground spaces such as the proposed partial basement at the northern end of the structure.

DESIGN AND CONSTRUCTION DETAILS

1) All piers should be reinforced longitudinally, with at least one No. 5 steel (Grade 60) rod for each 18 inches of pier perimeter (minimum two rods), to help prevent breakage of the piers due to uplift on their sides by any swelling materials. Additional reinforcing may be needed for other structural reasons. The bedrock penetration portion of the pier holes should be roughened artificially with a side tooth added to the auger after drilling and prior to cleaning in order to assure a good bond between the concrete and the bedrock. The roughening should consist of at least 1 inch by 1-1/2 inch high grooves at a vertical spacing of 18 inches. The upper portion of the piers should be kept smooth to reduce the adhesion between the swelling materials and the piers. Enlargement of the tops of the piers (mushrooming) must be avoided. In order to insure a consistent pier diameter, the contractor should be prepared to pour the top of pier with sonatube as needed. A 4-inch minimum “air space” or void should be provided beneath the portions of the grade beams that span between piers.

2) Temporary casing of the pier holes is expected to perhaps be necessary to control caving at some pier locations. Excessive water infiltration from within bedrock, greater than 4 inches, is possible, but not likely. Pumping to remove water or to place concrete below the water would then be required. Concrete with a design slump of 5 to 7 inches should be placed immediately after drilling and inspection in order to minimize water infiltration problems.

3) **Precautions should be taken against drying the foundation soils during construction or wetting thereafter.** Backfill around the building should be moistened and well-compacted (95 percent of Standard Proctor). A minimum slope of 8 inches in the first 10 feet is recommended. Flatter slopes would be acceptable in hard-surfaced areas such as pavements, etc.

4) Care should be taken in excavating for any footing foundations so as to avoid disturbing the subsoils. Any soils disturbed during footing excavation or preparation should be removed or recompacted prior to placing concrete.

5) Footing foundation walls should be well reinforced, both top and bottom and particularly around openings. This is to give them sufficient strength to resist slight differential movements that may occur in the bearing strata below foundation levels.

6) We recommend that Type II or Type I/II cement with a tricalcium aluminate content of less than 5 percent be used in all concrete exposed to the earth.

7) No special corrosion protection measures are necessary for below-grade utilities other than sleeving the fittings of metal water piping with polyethylene.

LATERAL EARTH PRESSURES

Basement Building Walls: Building walls will be comparatively rigid and should, in our opinion, be designed for 'at rest' lateral soil pressures. If on-site backfill is to be used, the lateral earth pressure design value would 60 pcf. As an alternative, an imported granular, relatively clean (less than 10 percent passing the U.S. No. 200 sieve), non-swelling material could be used as backfill. The imported gravel, if that alternative is selected, must be present within an area defined by a line extending upward from the base of the wall at an angle of 30 degrees from the wall. The lateral earth pressure may then be estimated by using an equivalent fluid density of 45 pcf. The upper 1-1/2 feet of backfill should be fairly impermeable to prevent surface water from entering the backfill. A precautionary subsurface drain should be provided around the perimeter of any below grade interior space. If significantly upward sloping backfill (steeper than 3 horizontal to 1 vertical) is to be used we should be contacted to provide additional recommendations.

Temporary Excavation Bracing: No temporary bracing is necessary for excavated areas if a 2 (horizontal) to 1 (vertical) slope is maintained. Should bracing be necessary at some critical area or desirable for personnel safety, we recommend that an "active" earth pressure of $40 \times Z - 150$ psf be used, where Z=depth of excavation (for example, if a 12 foot excavation is planned, the temporary bracing should be designed for a lateral earth pressure of $40 \times 12 - 150 = 330$ psf per linear foot).

Site Retaining Walls: The data presented in the section on basement walls is also generally applicable to site retaining walls with the following modifications:

1) The active lateral earth pressure may then be computed by using an equivalent fluid density of 30 pcf with imported, relatively clean, granular backfill or 45 pcf with on-site backfill.

2) Drainage should be provided to prevent water build-up behind site retaining walls. Generally, weep holes would be a suitable drainage provision.

Resistance: Lateral pressures on basement, and/or other retaining walls may be resisted by an ultimate passive equivalent fluid density of 280 pcf (natural soil or compacted fill). A coefficient of friction of 0.5 may also be used in the design.

PAVEMENT RECOMMENDATIONS

The pavement subgrade materials are generally expected to be various sand, silt, clay and gravel mixtures. These materials would typically be rated as fair to good subgrade soils.

Based on the expected subgrade soils and the assumed traffic loadings, the following minimum pavement sections are recommended:

1) AUTO AND/OR LIGHT TRUCK PARKING AREAS (EDLA of 5)

- 1) 5.0 inches of full-depth asphalt; **OR**
- 2) 3.0 inches of asphalt overlying 6 inches granular base course; **OR**
- 3) 4.5 inches of concrete

2) MAIN ACCESS DRIVES AND TRUCK TRAFFIC AREAS (EDLA of 10)

- 1) 8.0 inches of full-depth asphalt; **OR**
- 2) 5.0 inches of asphalt overlying 9 inches granular base course; **OR**
- 3) 6.0 inches of concrete

We highly recommend that the 6-inch concrete pavement be used for loading areas, and areas where truck-turning movements are concentrated, including trash dumpster areas.

Prior to placement of new pavement sections, the entire area should be stripped of all organic matter and debris. The exposed surface should then be proof rolled with a heavy pneumatic-tired vehicle, such as a loaded dump truck of approximately 20,000 pounds. Any soils, which are noted to be pumping or deforming excessively under the moving wheel loads, should be removed and replaced with a properly compacted and approved material. It is recommended that a geotechnical engineer observe this operation.

It should be understood that attempting to place and compact any asphalt pavement on a soft and deflecting subgrade generally results in unsatisfactory density of the base course and asphalt and/or formation of cracks in the asphalt. Either of these occurrences can severely weaken the pavement section.

Where pavements overlie existing fill, at least 2 feet, and possibly more if the fill is particularly poor, of the fill should be removed and replaced. Ideally all existing fill would be removed and should be if the owner desires maximum risk reduction.

All new fill in pavement areas should be compacted to at least 95 percent of the maximum Standard Proctor density (per ASTM D-698) from 1 percent below to 3 percent above the optimum moisture content. In cut areas, the upper 1-foot of subgrade materials should also be compacted to the 95 percent criteria. In existing fill areas, at least the upper 2 feet of the subgrade materials should be removed and recompacted to the 95 percent criteria.

The granular base course should meet Colorado Department of Transportation (CDOT) specifications. The use of Class 6 (3/4-inch) aggregate is suggested. The base course should be compacted to at least 95 percent of the maximum Modified Proctor density (per ASTM D-1557).

The asphalt (hot plant mix) should meet CDOT specifications. We suggest a specific job mix formula meeting grading S (3/4-inch) be used. The asphalt mix should have a minimum Marshall stability of 1,800 pounds and a flow of between 8 and 18 (ASTM D-1559). The asphalt should be compacted to at least 95 percent of the density obtained from laboratory specimens made in accordance with the Marshall Method (per ASTM D-1559).

Concrete used for pavements should also meet CDOT specifications. We suggest the use of Class P concrete.

The lighter pavement sections are not designed to carry repeated heavy construction traffic. Therefore, construction operations subsequent to paving must be planned to avoid those paved areas.

Adequate surface drainage provisions should be made so as to prevent water flow into the subgrade soils beneath the pavements. The life of any pavement structure is greatly diminished by improper drainage.

MISCELLANEOUS

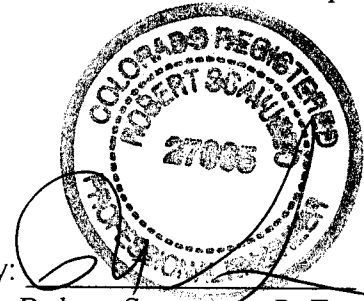
In any geotechnical investigation it is necessary to assume that subsurface conditions do not change greatly from those indicated by our exploratory borings. However, our experience has shown that anomalies do sometimes become apparent during construction.

For that reason, we recommend that a representative from our firm who is familiar with the subsurface conditions observe the construction operations discussed in this report.

Respectfully submitted,
CTC-Geotek, Inc.

By: Michael T. Bogan
Michael T. Bogan
Sr. Geotechnical Consultant

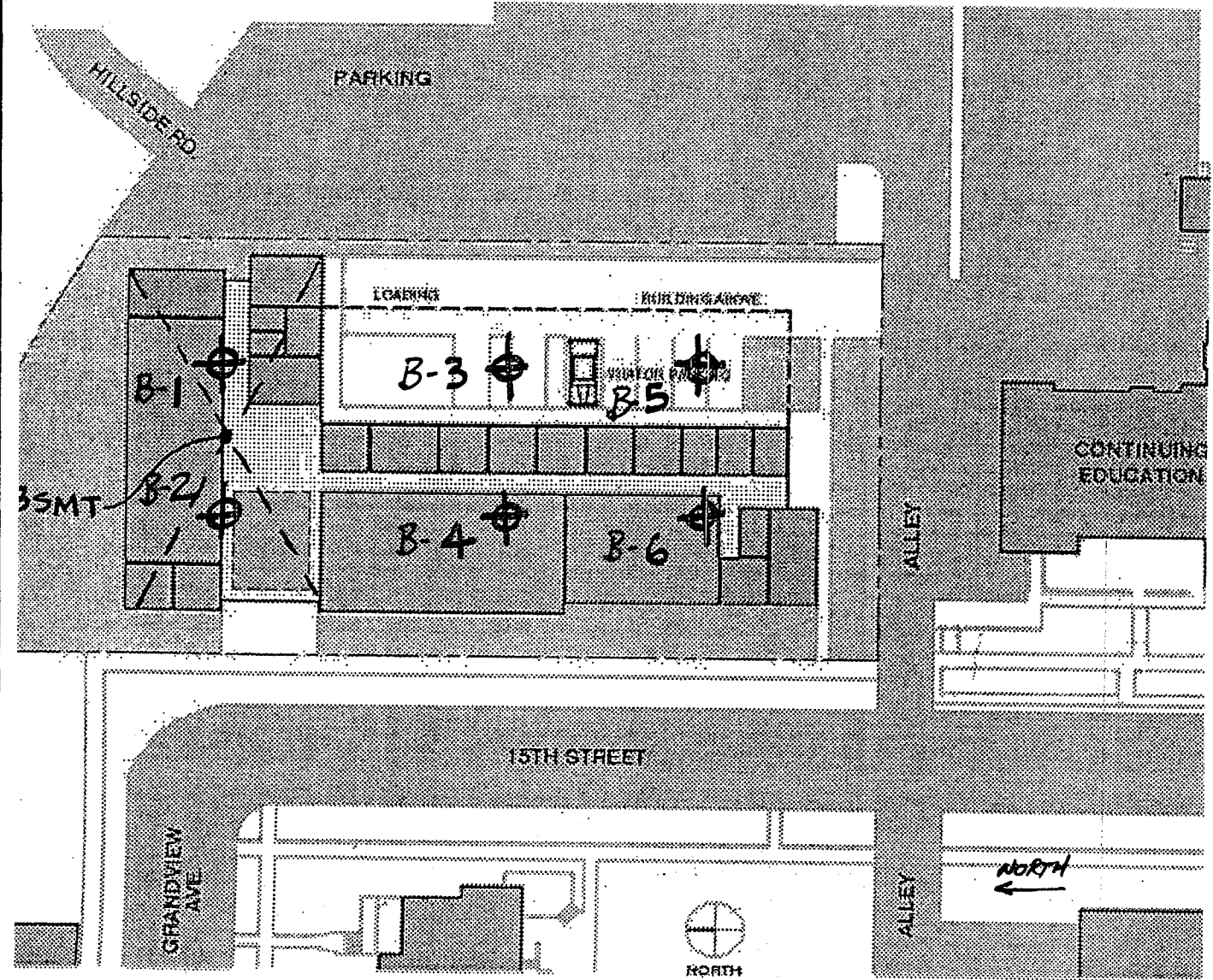
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BORING LOCATION PLAN

CTC-GEOTEK
ENGINEERING TESTING INSPECTION

155 S. Navajo Denver, CO 80223 303-698-1050

IBS Building
SE Corner of 15th Street & Grandview Avenue
Boulder, Boulder County, Colorado

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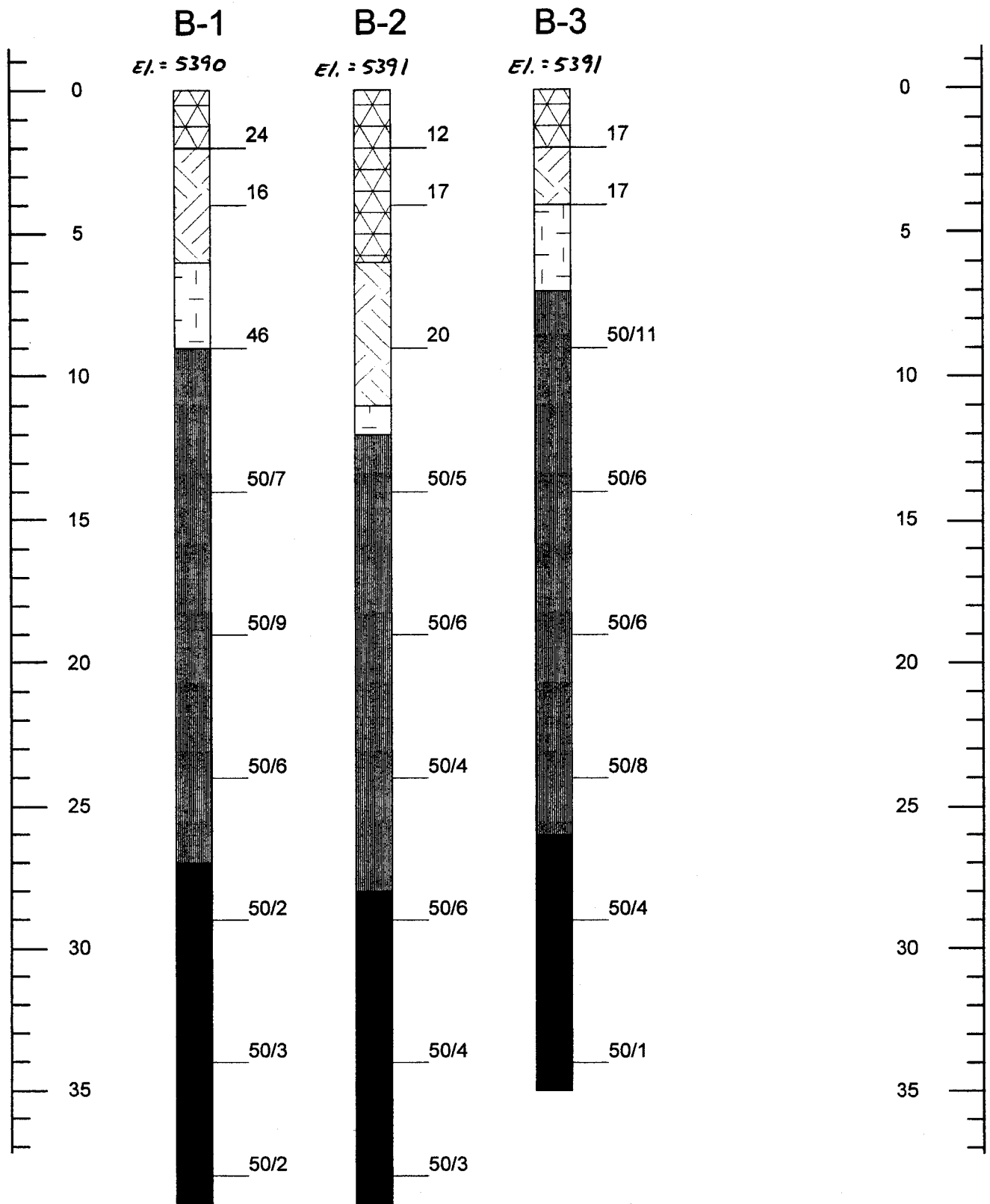
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JOB NO. 282014

PLATE 1



LOGS OF EXPLORATORY BORINGS

CTC-GEOTEK
ENGINEERING TESTING INSPECTION

155 S. Navajo Denver, CO 80223 303-698-1050

IBS Building
SE Corner of 15th Street & Grandview Avenue
Boulder, Boulder County, Colorado

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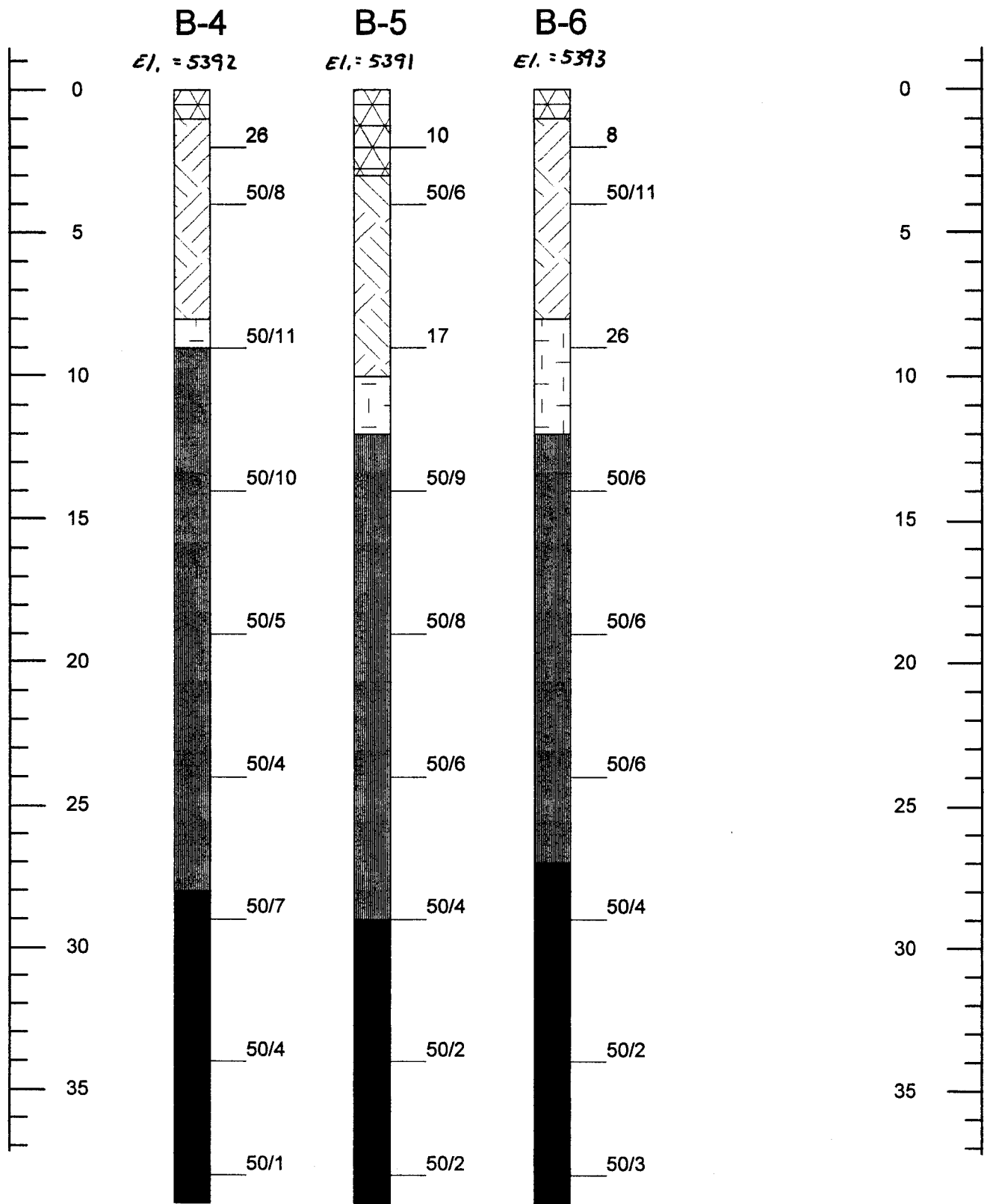
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PLATE 2



LOGS OF EXPLORATORY BORINGS

CTC-GEOTEK
ENGINEERING TESTING INSPECTION

155 S. Navajo Denver, CO 80223 303-698-1050

IBS Building
SE Corner of 15th Street & Grandview Avenue
Boulder, Boulder County, Colorado

DRAWN BY: mtb

SCALE: Vertical 1" = 5'
Horizontal

CHECKED BY:

DATE: 4-22-08

JOB NO. 282014

PLATE 3



Fill - sand, gravel, clay and silt, various mixtures, moderately compact to compact, medium moist to moist, brown, red-brown



Sand, clay and silt, various mixtures and layers, occasionally gravelly, medium moist to moist, brown to brownish red



Clay (severely weathered claystone), silty, very stiff, medium moist to moist, brown to dark brown, occasionally calcareous



Claystone bedrock, weathered, silty, medium hard to very hard, medium moist, gray, yellow-brown, brown



Claystone bedrock (shale), silty, occasionally sandy, very hard, medium moist, gray, gray-blue, black

No water noted at the time of drilling

NOTES: (1) Borings were drilled on April 15 & 16, 2008 with 4-inch, solid-stem, power augers.

(2) 24 indicates that 24 blows of a 140-pound hammer falling 30 inches were required to drive the sampler 12 inches. 50/7 indicates 50 blows for 7 inches.

(3) Stratification lines are approximate and transitions may be gradual.

(4) The logs only show conditions at the times and locations indicated.

LEGEND AND NOTES

CTC-GEOTEK
ENGINEERING TESTING INSPECTION

155 S. Navajo Denver, CO 80223 303-698-1050

IBS Building
SE Corner of 15th Street & Grandview Avenue
Boulder, Boulder County, Colorado

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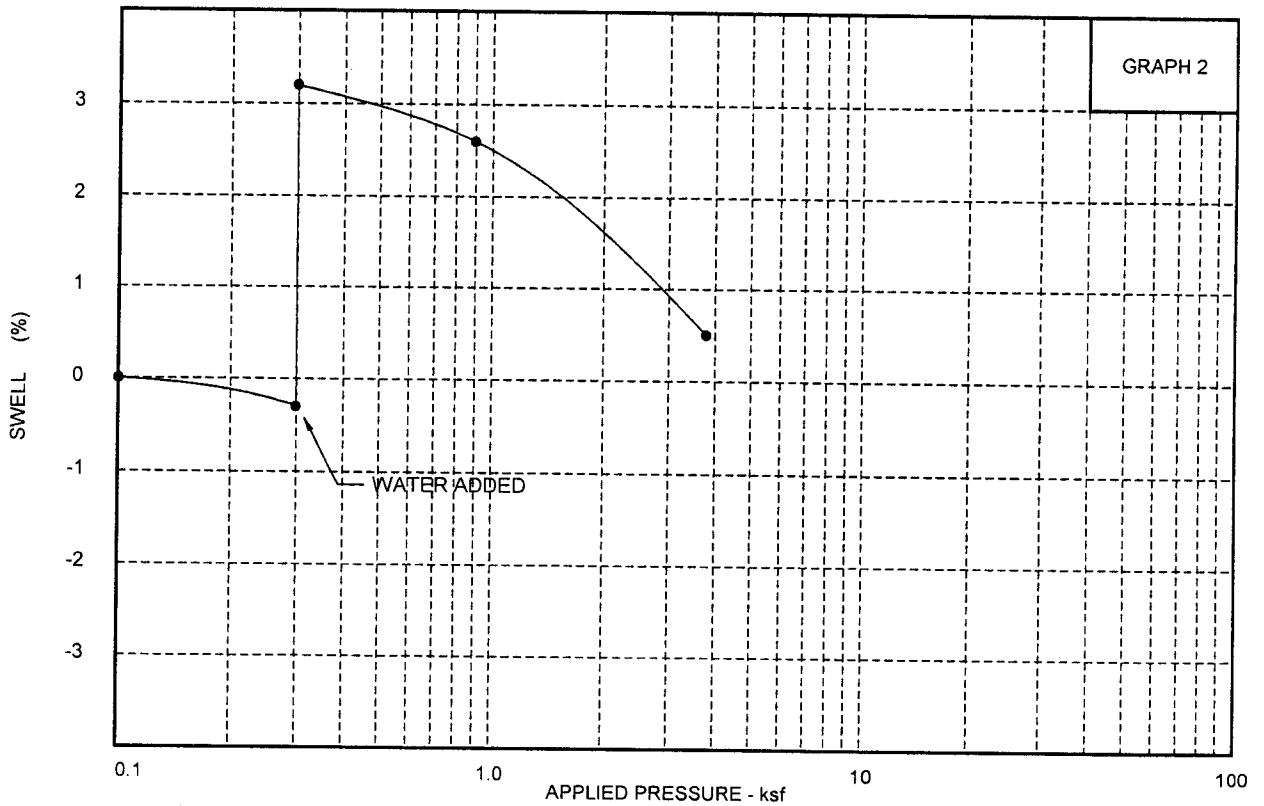
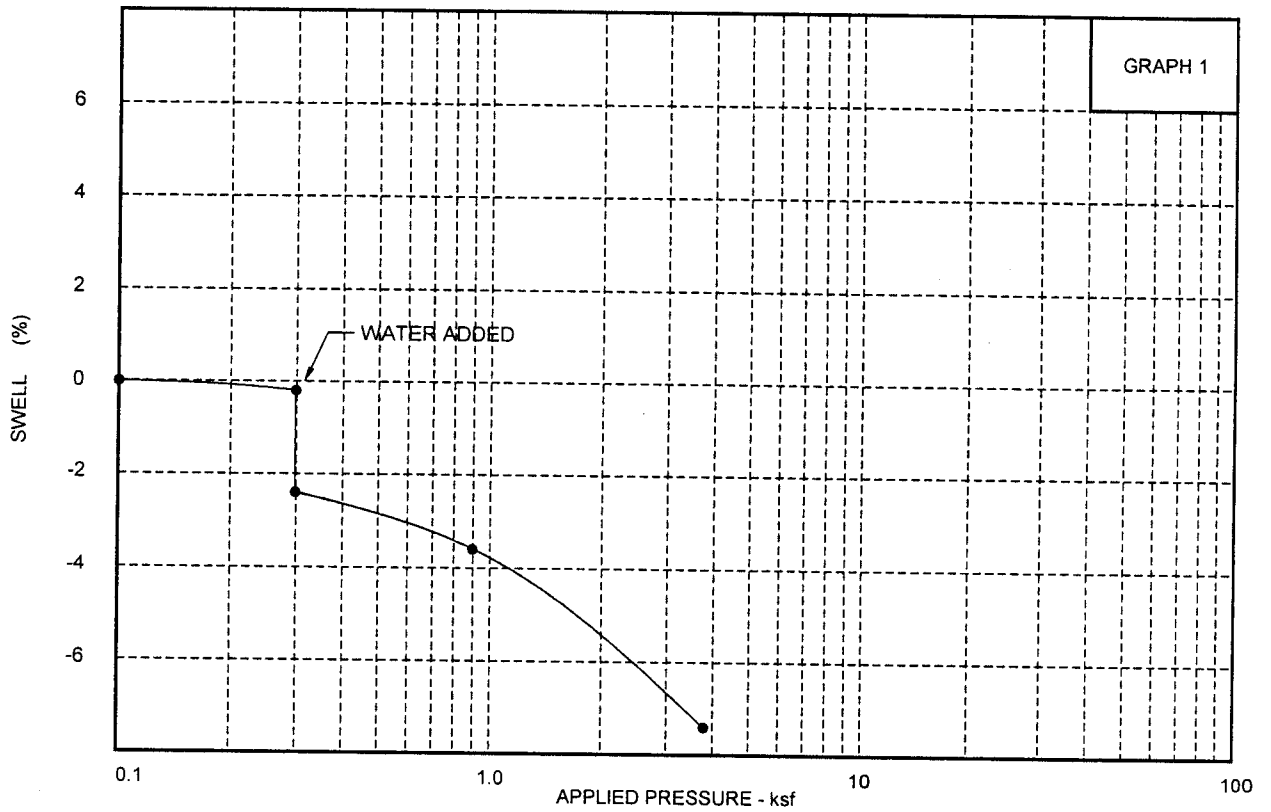
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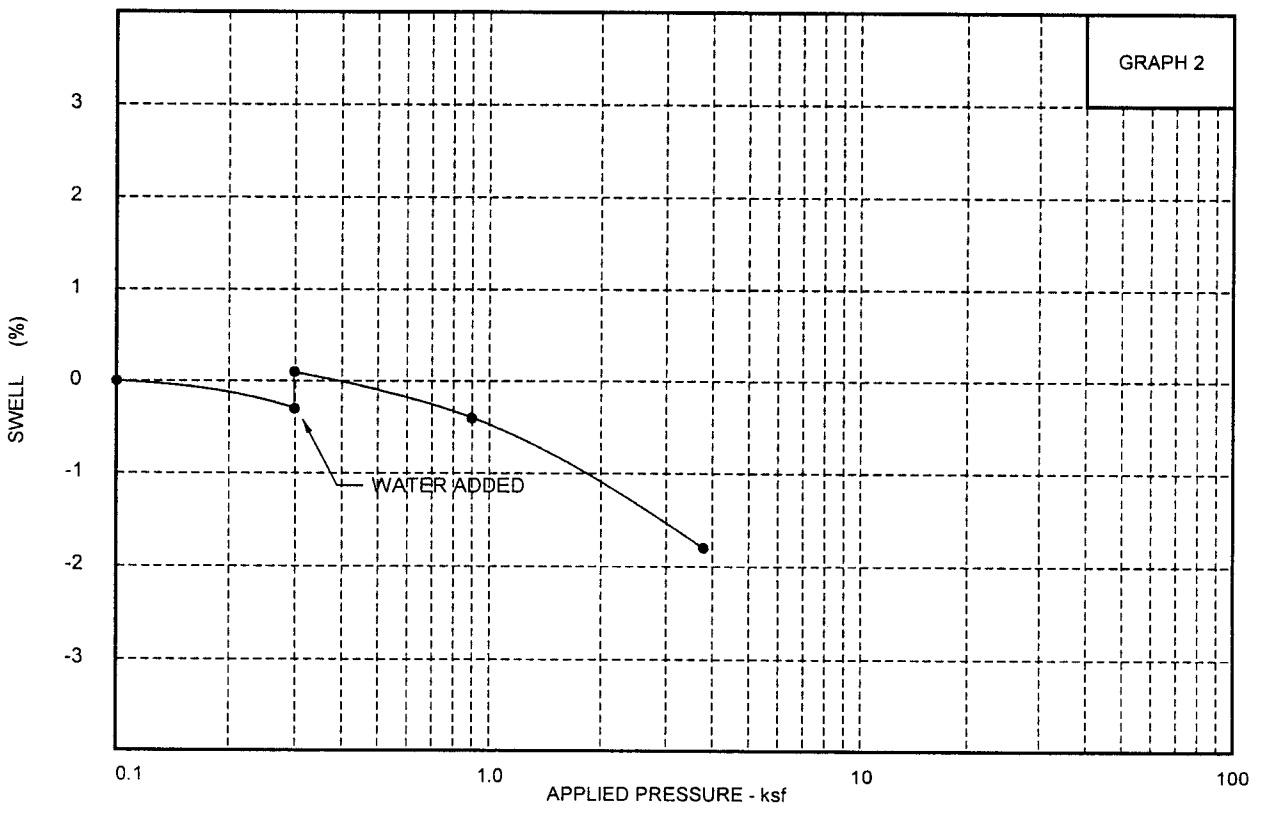
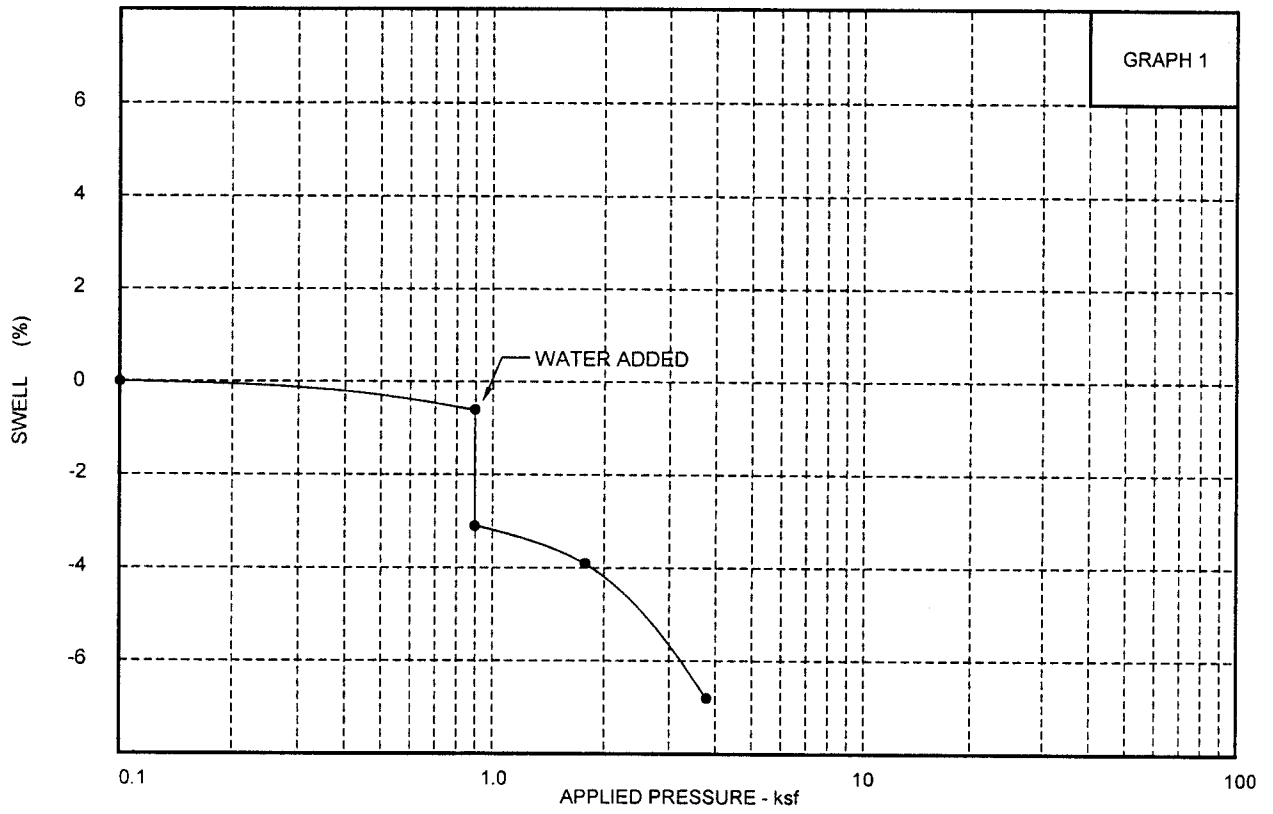
DATE: 4-22-08

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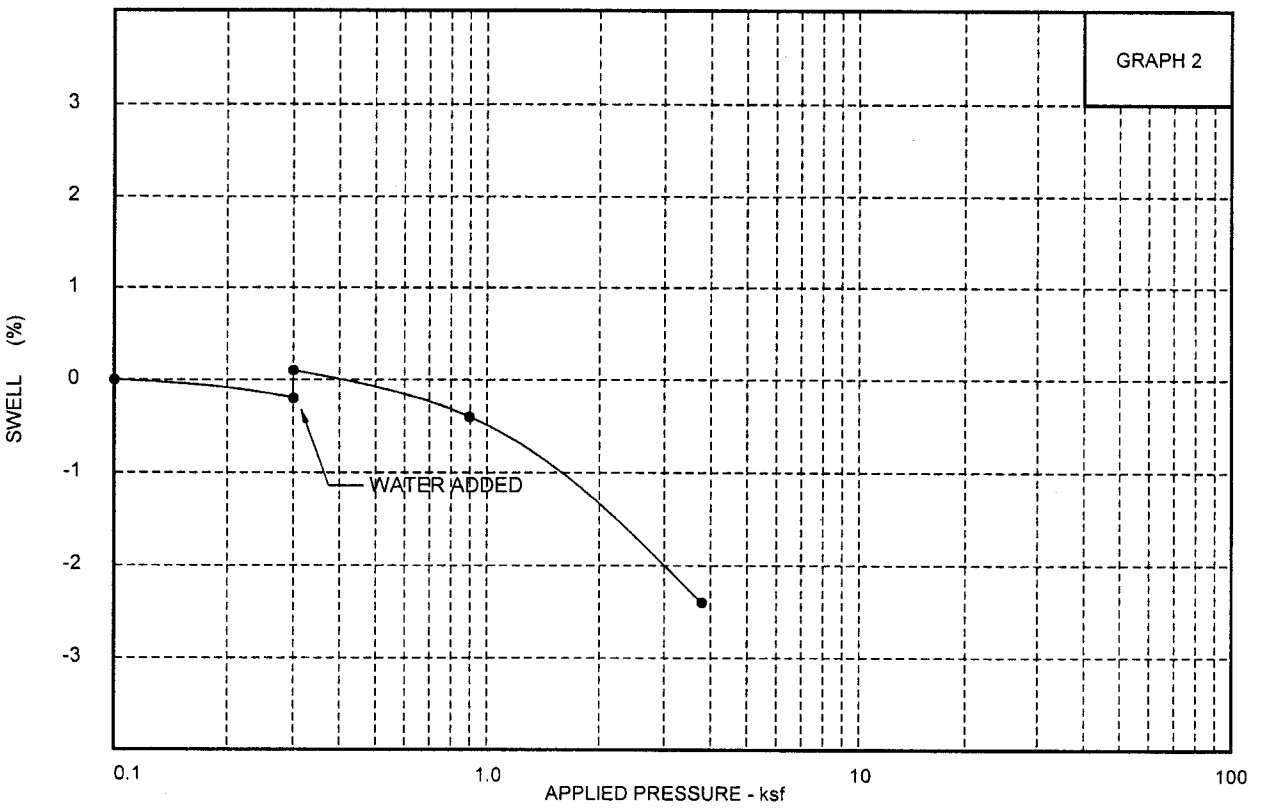
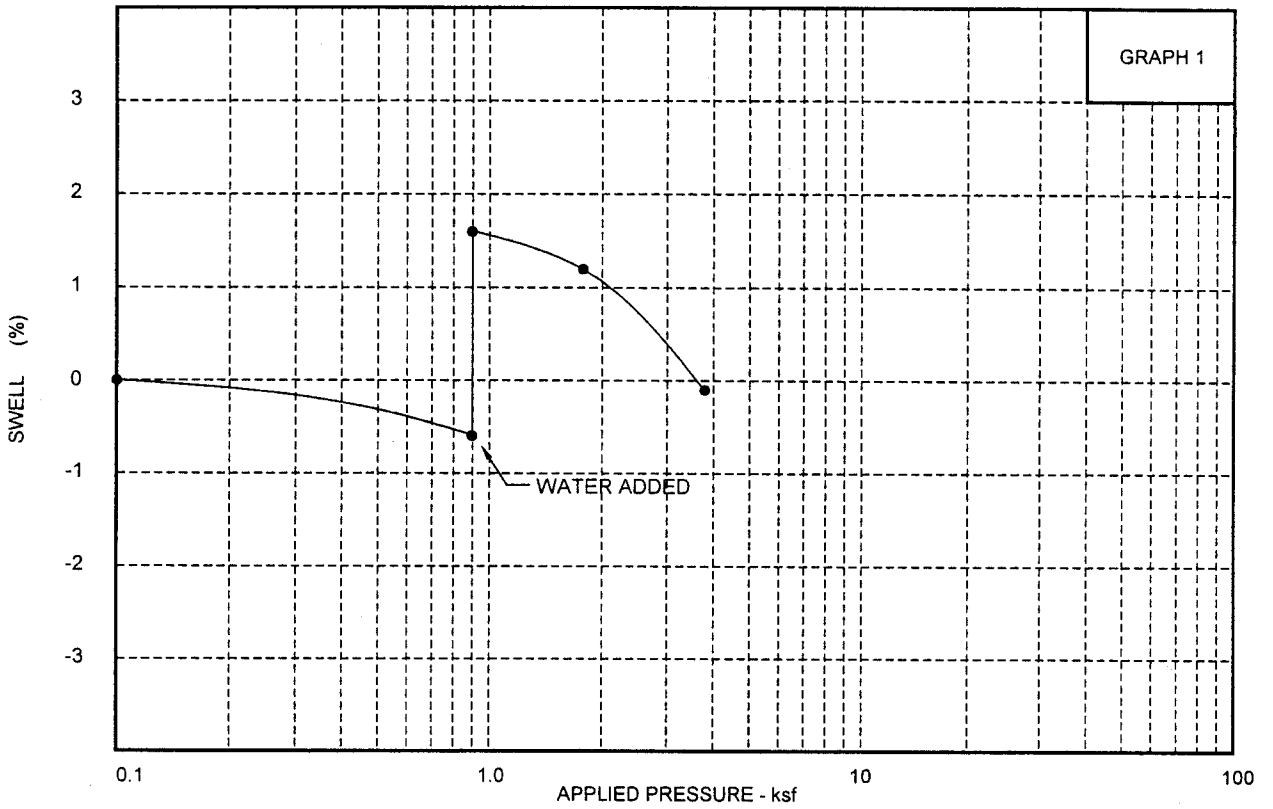
PLATE 4



GRAPH NO.	BORING NO.	SAMPLE NO.	DEPTH IN FEET	DRY DENSITY (PCF)	MOISTURE (%)	SOIL DESCRIPTION	GTC-GEOTEK	
1	B1		4	77.8	11.6	Silty Clay, and sand, trace gravel	SWELL - CONSOLIDATION TEST	
2	B1		9	114.5	13.3	Claystone	DRAWN BY: JLW	JOB NO.: 282014
							DATE: 4-29-2008	PLATE: 5



GRAPH NO.	BORING NO.	SAMPLE NO.	DEPTH IN FEET	DRY DENSITY (PCF)	MOISTURE (%)	SOIL DESCRIPTION	CTC-GEOTEK	
1	B2		14	96.5	11.4	Claystone	SWELL - CONSOLIDATION TEST	
2	B3		4	102.1	19.0	Claystone	DRAWN BY: JLW	JOB NO.: 282014
							DATE: 4-29-2008	PLATE: 6



GRAPH NO.	BORING NO.	SAMPLE NO.	DEPTH IN FEET	DRY DENSITY (PCF)	MOISTURE (%)	SOIL DESCRIPTION	CTC-GEOTEK	
1	B4		9	120.6	14.1	Claystone	SWELL - CONSOLIDATION TEST	
2	B6		2	106.3	16.7	Silty Clay, and sand, trace gravel	DRAWN BY: JLW	JOB NO.: 282014
							DATE: 4-29-2008	PLATE: 7

SUMMARY OF LABORATORY TEST RESULTS

Project No. 282014

BORING NO.	SAMPLE NO.	DEPTH IN FEET	SAMPLE TYPE (NOTE 1)	DRY DENSITY (PCF)	MOISTURE (%)	ATTERBERG LIMITS			% FINES	WATER SOLUBLE SULFATES (%)	SHEAR STRENGTH (PSF) (NOTE 2)	ADDITIONAL TEST RESULTS ATTACHED (NOTE 3)	SOIL DESCRIPTION
						LL	PI	PL					
B1		4	CA	77.8	11.6						SW	Silty Clay, and sand, trace gravel	
B1		9	CA	114.5	13.3						SW	Claystone	
B2		2	CA	88.9	14.8	45	22	23	37.2			Sand, and silty clay, trace gravel A-7-6(3) SC	
B2		14	CA	96.5	11.4						SW	Claystone	
B3		4	CA	102.1	19.0				.004		SW	Claystone	
B4		9	CA	120.6	14.1				.005		SW	Claystone	
B6		2	CA	106.3	16.7						SW	Silty Clay, and sand, trace gravel	
B6		9	CA	112.6	17.9	49	33	16	97.2			Claystone A-7-6(34) CL	

NOTE 1 - SAMPLE TYPE

- AD - Air Dried
- AS - Auger Sample
- BS - Bag Sample
- CA - California Sample
- HD - Hand Drive
- RM - Remolded Sample
- ST - Shelby Tube Sample

NOTE 2 - SHEAR STRENGTH TESTS

- C₁ - Unconfined Compression
- C₂ - Miniature Vane Shear
- C₃ - Pocket Penetrometer
- C₄ - Pocket Vane

NOTE 3 - ADDITIONAL TEST RESULTS ATTACHED

- CT - Consolidation Test
- GA - Gradation Analysis
- PT - Proctor
- RV - R-Value
- SW - Swell-Consolidation Test
- TT - Triaxial Test