CTC-GEOTEK, INC.
ENGINEERING TESTING INSPECTION

# SOIL AND FOUNDATION INVESTIGATION

Aurora Municipal Center
NW of E. Alameda Ave. & S. Chambers Rd.
Aurora, Colorado
(Revision 1)

PREPARED FOR:

The Weitz Company, Inc.
4725 South Monaco St., Suite 100
Denver, Colorado 80202-5669

Attention: Mr. Nick Gefrides

Project 212034 August 10, 2001

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## SUMMARY

(1) The subsurface conditions encountered in our exploratory borings generally consist of 6 to 14 feet of overburden soils, including existing fill, natural silty clays and weathered claystone, overlying claystone and sandstone bedrock to the maximum depth investigated, 45 feet below the existing grade. Groundwater was encountered in the exploratory borings..

(2) Due to presence of severe swelling soils, a straight shaft pier (caissons) foundation system is recommended.

(3) Slab-on-grade construction recommendations and associated risks are presented in the text of this report.

(4) A representative from our office should observe the construction operations discussed in this report.

## SCOPE OF STUDY

This report presents the results of a soil and foundation investigation for the proposed Aurora Municipal Center located at northwest comer of East Alameda Avenue and South Chambers Road, Aurora, 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, attempt to evaluate the risks of slab-on-grade construction, 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 at the site is to consist of a five(5)-story office building and a multi-story parking structure without below grade space. The office building floor slab elevation is anticipated to be at 5471 and the parking structure floor slab elevation is anticipated to be at 5465. Design loads are not known at the time of this report, but are anticipated to be moderate to high.

## FIELD INVESTIGATION

Thirteen (13) exploratory test borings were drilled on the site at the approximate locations shown on Plate 1. The borings were drilled with 4 diameter, continuous flight, solid-stem power auger 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 5.

## LABORATORY TESTING

All samples were inspected and classified in the laboratory by the project engineer. Natural water contents, dry unit weights, Atterberg Limits and partial gradations were obtained from relatively undisturbed drive samples obtained at the site (see Table 1).

Swell-consolidation tests were performed on typical specimens of potentially swelling materials (see Plates 7 and 8). These tests indicate the behavior of these materials upon loading and wetting.

## SUBSURFACE CONDITIONS

The subsurface conditions encountered in our exploratory borings generally consist of 6 to 14 feet of overburden soils, including existing fill, natural silty clays and weathered claystone, overlying claystone and sandstone bedrock to the maximum depth investigated, 45 feet below the existing grade.

The on-site existing fill typically consists of dark brown to brown, slightly moist to very moist, silty clay with some sand and occasionally mixed with claystone materials. As indicated by the penetration resistance values, the existing fill was firm to stiff.

The on-site natural clay is typically brown, moist to very moist, silty clay with some sand. As indicated by the penetration resistance values, the natural silty clay was firm to stiff.

The on-site sandstone bedrock was typically yellowish brown to brown, slightly moist to wet and fine to medium grained. As indicated by the penetration resistance values, the sandstone bedrock was medium hard to very hard.

The claystone bedrock encountered was typically yellowish gray to gray to brown and moist to very moist. As indicated by penetration resistance values, the claystone was medium hard to hard.

As shown on the swell-consolidation test results presented on Plates 7 and 8, the overburden soils exhibit a low to high potential upon wetting.

Groundwater was encountered in the exploratory borings ranging from 10 to 20 feet below the existing grade at the time of drilling and two(2) weeks subsequent to drilling. Groundwater levels are indicated on the exploratory boring logs (Plates 2 through 5). It **should be noted that groundwater levels can vary and perched water tables can be created with changes in precipitation, irrigation, drainage and land use.**

## FOUNDATIONS

Based upon the subsurface conditions encountered and the proposed construction, it is recommended that the building structure, including such items as escalators and elevators, and critical exterior slabs (such as entryways) at the site be founded on straight shaft piers (caissons) drilled at least 10 feet into the non-weathered bedrock strata and have a minimum pier length of 25 feet.

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 are based on the field and laboratory test results and the supporting capacity of the average of the softest materials encountered. The following maximum allowable end bearing and compressive side shear values are recommended for the portion of the drilled piers in **non-weathered** bedrocks.

Drilled Pier Design Data

| Penetration into non-weathered bedrocks (feet) | Maximum allowable end bearing pressure (psf) | Compressive side shear (psf) |
|---|---|---|
| 0 - 5 | ------- | 2,000 |
| 5 -10 | ------- | 2,500 |
| 10+ | 25,000 | 2,500 |

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

A design value of 60 percent of the compressive side shear may be used for tension shear.

If bedrocks are exposed at the subgrade, the top 3 feet of pier shaft should not be counted in side shear calculation.

A minimum pier diameter of 16 inches or 5 percent of the expected total shaft length, whichever is greater, should be used to facilitate proper cleaning and observation of the drilled pier holes.

To achieve full design pressures, 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 (side shear) percent 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.

For uniform building code seismic analysis (UBC 1997) the site can be considered to be in Seismic Zone 1 and Soil Profile Type $S_C$.

**Soft lenses and pockets of bedrocks were encountered in some exploratory borings at various depth. The end of the pier must not be supported on soft bedrock lenses or pockets. If soft zone is encountered at the design penetration, additional pier penetration is required to pass the soft. A representative from CTC-Geotek, Inc. must inspect the pier installation and verify the bedrock conditions.**

Lateral pier design parameters would be horizontal modulus of subgrade reaction values of 40 tcf and 200 tcf for the overburden materials and the bedrock, respectively. The modulus values are based on a pier diameter of 1 foot. Values used should be the preceding divided by the actual pier diameter in feet.

The following soil data are provided for the L-Pile program.

| | Overburden soils(including weathered bedrock) | Sandstone bedrock | Claystone bedrock |
|---|---|---|---|
| Wet Density (pcf) | 115 | 120 | 115 |
| Cohesion C (psf) | 2,000 | 0 | 4,000 |
| Friction Angle $\phi$ (degree) | 0 | 38 | 0 |
| $\varepsilon 50$ (in/in) | 0.007 | 0 | 0.005 |
| K (Cyclic) (pci) | 200 | 225 | 800 |
| K (Static) (pci) | 500 | 225 | 1,600 |

Note: The value of parameter K should be reduced to 125 pci when sandstone bedrock is submerged.

Piers are recommended for support of building related construction. However, site retaining walls which are not rigidly connected to a building, could be supported on shallow footing foundations. Based on the expansive potential of the subsurface soils, it is recommended that the retaining wall footings be placed on at least 4 feet of selected fill and be designed using a maximum allowable soil bearing pressure of 3,000 psf and as high of a dead-load pressure as possible. Potential movements would be settlement on the order of 1 inch and uplift movement on the order of 2 to 4 inches. If the underlying materials were not wetted in the future, the actual uplift movements would be very small. We do not recommend that any retaining wall footings be placed directly on the on-site existing soils. The estimated potential uplift movements of the retaining wall footings to be placed directly on the on-site existing soils may exceed 4 inches.

The top 12 inches of selected fill should be an imported, inherently non-swelling, granular soil, such as silty sand or "crusher reject". The on-site clays (in both natural or fill states), exclusive of topsoil, organic matter and claystone material, are suitable to be used as selected fill in the areas at least 12 inches below the bottom of the retaining wall footing, the on-site claystone bedrock including weathered claystone material and/or existing fill containing claystone material are not to be used as selected fill beneath the footings. The quantity of on-site soils that is available to be used as select fill is not known at the time of report. Import of select fill may be required.

Any selected fill **to be placed beneath the retaining wall footings** should be placed in uniform lifts, moisture conditioned and compacted to at least 98 percent of the maximum Standard Proctor density (per ASTM D-698) at a moisture content appropriate for the particular material. The specific moisture content will be determined by the geotechnical engineer at the start of construction and re-verified through the process.

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

Irrigation control provisions near the retaining walls are presented in the LATERAL EARTH PRESSURES Section of this report.

## FLOOR SLAB CONSTRUCTION

**Due to presence of severe swelling materials, floor slabs should ideally be structurally supported.** This is the safest way to construct slabs where expansive materials are present. In this case, the floors could be supported on void form, grade beams and piers, the same as the structure. A 12 inch minimum "void" or crawl space would be placed beneath the floor system. This would greatly reduce the risk of damage to slabs and interior partitions due to swelling soils.

Adequate precautions should be taken during construction to minimize excessive wetting of the subgrade soils and/or excessive drying of the fill material in the building pad. Adequate drainage for runoff and timely backfill of excavations in the building pad is recommended.

Provided the owner can accept some risk of slab movement, slab-on-grade construction is possible. If this alternate is chosen, it is recommended that the following measures be taken to help minimize, but not eliminate, the floor slab movements:

(A) The on-site soils and bedrocks should be excavated to a depth of at least 6 feet below the bottom of floor slabs.

(B) Floor slabs should be placed on at least 6 feet of select fill including at the top at least 12 inches of inherently non-swelling, relatively impervious soils(sand cap), such as imported silty sands or "crusher reject". The select fill should extend laterally to at least 6 feet outside the proposed foundation wall location. The select fill should be compacted to at least 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 the underlying clays preventing their drying after placement and prior to slab pouring.

(C) The select fill should consist of either inherently non-swelling material, such as imported sands, or at increased risk, materials which can be placed and maintained in such manner that its swell potential is minimized. Imported sand or "crusher reject" fill would ideally be used for all of the underslab fill. The on-site clays(in natural or fill states), exclusive of topsoil, organic matter and claystone material, are anticipated to be potentially the latter type. We recommend further laboratory testing be performed prior to reuse of the on-site clays. Any select fill (imported or on-site) should be approved by a geotechnical engineer prior to it's placement. Earth Work section of this report provides additional select fill selection and placement criteria.

The on-site claystone bedrock (including weathered claystone) and/or existing fill mixed claystone material is not to be used as select fill. We recommend that during the excavation or grading, clay material be separated from claystone material and stockpiled for future use. The quantity of the on-site soils that are available to be used as select fill is not known at the time of this report. Import of select fill may be required.

(D) Separate slabs from bearing members to allow their independent movement. Joints (construction joints/saw cuts) in the slabs at maximum spacings in accordance with ACI requirements.

(E) Place a minimum 3 inch "void" above, or preferably below non-bearing partitions in slab-on-grade areas. Door jambs, drywall, heating and cooling equipment, etc., should be similarly protected.

(F) **Keep any exposed clay and claystone subgrade moist during construction by occasional sprinkling.**

(G) No irrigation should occur for a distance of 6 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 building.

Water infiltration into floor slab subgrade(wetting of subgrade) will cause subgrade soils to swell and could cause slab damage. Therefore, it is important that surface water be drained away from any building structures by designed surface drainage or be collected and carried away by a subsurface drainage system. Lawn watering should be minimized or limited to the extent practical at the subject site.

(H) A polyethylene moisture barrier may be required by floor covering manufacturers below any slabs that are to receive relatively impermeable floor coverings. If this moisture barrier is to be installed, it must be continuous and would be placed shortly before concrete placement. Any moisture/vapor barrier used should be installed per recommendations of ASTM E-1643.

(I) Sewer lines beneath the building should have a sufficient 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. Utility lines 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.

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

The proceeding slab-on-grade precautions with 6 feet of select fill would generally limit potential movements to 1.0 to 1.5 inches with up to approximately 3 inches of heave possible in isolated areas. Damages associated with these types of movements could be significant. If the floor slab subsoils are not wetted, future movements would be minimized.

It should be noted that these floor slab comments and recommendations would also apply to the exterior slabs, and pedestrian walkway around building, particularly at critical areas such as attached sidewalks and entryways where doors could be inhibited.

## GROUNDWATER CONDITIONS AND PERIPHRAL DRAINAGE SYSTEM

Groundwater was encountered in the exploratory borings ranging from 10 to 20 feet below the existing grade at the time of drilling and two(2) weeks subsequent to drilling. Groundwater levels are indicated on the exploratory boring logs (Plates 2 through 5). Although present groundwater conditions are favorable for the anticipated floor slab elevations, based on the expansion characteristics of the on-site materials, it is recommended that precautionary subsurface drainage systems connected to sumps or other suitable outlets be provided. The systems consist of peripheral drainage along foundation walls(grade beams).

Peripheral drainage should consist of at least a 2 foot width of free drainage granular material and/or drainage board on the back side of the walls and a pipe drain surrounded by granular material at least 18 inches below the slab level. The pipe should be sloped a minimum of 3/4 percent to the outlet. Peripheral drainage system design should be coordinated with foundation wall backfill material selection.

## LATERAL EARTH PRESSURES

Foundation Walls or Grade Beams: Foundation walls or grade beams will be comparatively rigid and should, in our opinion, be designed for 'at rest' lateral soil pressures. If on-site clays are to be used as backfill, the lateral earth pressure design value would be estimated by an equivalent fluid density of 80 pcf. A imported granular sands with less than 15 percent of fines could be used as backfill. The imported sands, 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 55 pcf. The cost of imported granular sands should be considered if the equivalent fluid density of 55 pcf is to be used.

The upper 1 foot of backfill should be fairly impermeable to prevent surface water from entering the backfill. Any claystone material (including weathered claystone bedrock) should not be used as foundation wall backfill.

Temporary Excavation Bracing: Temporary bracing is not necessary for excavated areas if a 1.5 (horizontal) to 1 (vertical) slope is maintained. It may be noted that vertical excavations in the types of materials present on the site typically do not present problems during short-term construction time periods. However, the stability of vertical faces cannot be assured. Should bracing be necessary at some critical area or desirable for personnel safety we recommend that an "active" earth pressure of 40 x 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 x 12 - 150 = 330 psf per linear foot).

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

1) The minimum width of excavation recommended behind the retaining wall, in order to reduce earth pressures and lateral soil expansion pressures, can be estimated as follows: width of backfill =H/1.7 where H is the height of the retaining wall.

2) The lateral earth pressure may then be computed by using an equivalent fluid density of 40 pcf with imported granular sands with less than 15 percent of fines or 55 pcf with on-site clays.

3) Drainage should be provided to prevent water build-up behind site retaining walls. Weep holes would be a suitable drainage provision. Irrigation should ideally occur at least 6 feet away from the edge of retaining wall or limited to the extent practical at the site.

Resistance: Lateral pressures on retaining walls may be resisted by an ultimate passive equivalent fluid density of 200 pcf (natural soil or compacted fill). An ultimate coefficient of friction of 0.4 may also be used in the design.

## EARTH WORK

All topsoil, organic materials should be removed from within the proposed building and pavement areas. Topsoil depths may vary throughout the site but are anticipated to be on the order of 6 inches.

We recommend that 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.

Select fill refers to the fill material to be placed beneath the floor slabs, retaining wall footings, pavement sections and along foundation walls. The select fill should consist of either inherently non-swelling material, such as imported sands, or at increased risk, materials which can be placed and maintained in such manner that its swell potential is minimized. The on-site clays(in natural or fill states), exclusive of topsoil, organic matter and claystone material, are anticipated to be potentially the latter type. We recommend further laboratory testing be performed prior to reuse of the on-site clays. Any select fill (imported or on-site) should be approved by a geotechnical engineer prior to it's placement.

**The on-site claystone bedrock(including weathered claystone and/or existing fill mixed with claystone material) is not to be used as select fill. We recommend that during the excavation or grading, clay material be separated from claystone material and stockpiled for future use. The quantity of the on-site soils that are available to be used as select fill is not known at the time of this report. Import of select fill may be required.**

Select fill should be compacted to at least 95 percent(98 percent beneath retaining wall footings only) of the maximum Standard Proctor density (per ASTM D-698) at a moisture content appropriate for the particular material. We would expect that clay soils would require a moisture content of optimum to 3 percent above optimum in order to minimize swell potentials. The moisture content of essentially granular material such as sand would not be critical and could be placed within ±2 percent of the optimum moisture content.

**Please note that the on-site soils and bedrocks would exhibit higher swell potentials if they are allowed to become drier. This must not be allowed to occur during the construction period on any foundations, floor slabs, and/or pavement subgrade. Otherwise, the potential heaves would increase significantly.**

Fill in landscaped areas may be placed at a minimum of 88 percent of the maximum Standard Proctor density with no moisture control.

## DESIGN AND CONSTRUCTION DETAILS

Piers:

1) Piers should be reinforced longitudinally, with one No. 5 steel (Grade 60) rod for each 16 inches of pier perimeter (minimum two rods), to help prevent breakage of the piers due to uplift on their sides by swelling materials. The bedrock penetration portion of the pier hole(at least 10 feet below the top of pier) should be roughened artificially. The upper portion of the pier holes 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.

2) A 6 inch minimum "air space" should be provided beneath the portions of the grade beams that span between piers.

3) Groundwater was encountered in the exploratory borings, therefore use of temporary casing should be anticipated to install piers. If groundwater infiltration greater than 3 inches occurs in pier holes, pumping to remove water or to place concrete below the water would then be required. Concrete should be placed immediately after drilling and inspection in order to minimize water infiltration problems.

4) Concrete should be placed in drilled piers the same day they drilled. The presence of water may require that concrete be placed immediately after the drilled pier holes is completed. Failure to place concrete the day of drilling will normally result in a requirement for additional penetration. Concrete used in the drilled piers should be a fluid mix with a sufficient slump so it will fill the void between reinforcing steel and the drilled pier hole. Typically a slump on the order of 5 to 7 inches is considered adequate.

5) Backfill around the building should be a non-swelling material, moisture conditioned and compacted to at least 95 percent of the maximum Standard Proctor density (per ASTM D-698) at a moisture content sufficient to minimize swell potential. The exterior grades should be well sloped away from the structure. A minimum slope of 6 inches in the first 10 feet is recommended. However, a slope of 3.5 inches in 10 feet could be used in paved areas.

## CONCRETE

The sulfate concentration tests performed on the on-site soil samples indicated the sulfate concentration ranging from 0.028 to 0.052 percent. This concentration indicated the risk of sulfate attack on concrete is negligible. As a precautionary measure, we recommend that Type II cement be used in all concrete exposed to earth.

## 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 to observe the construction operations discussed in this report.

Respectfully submitted,

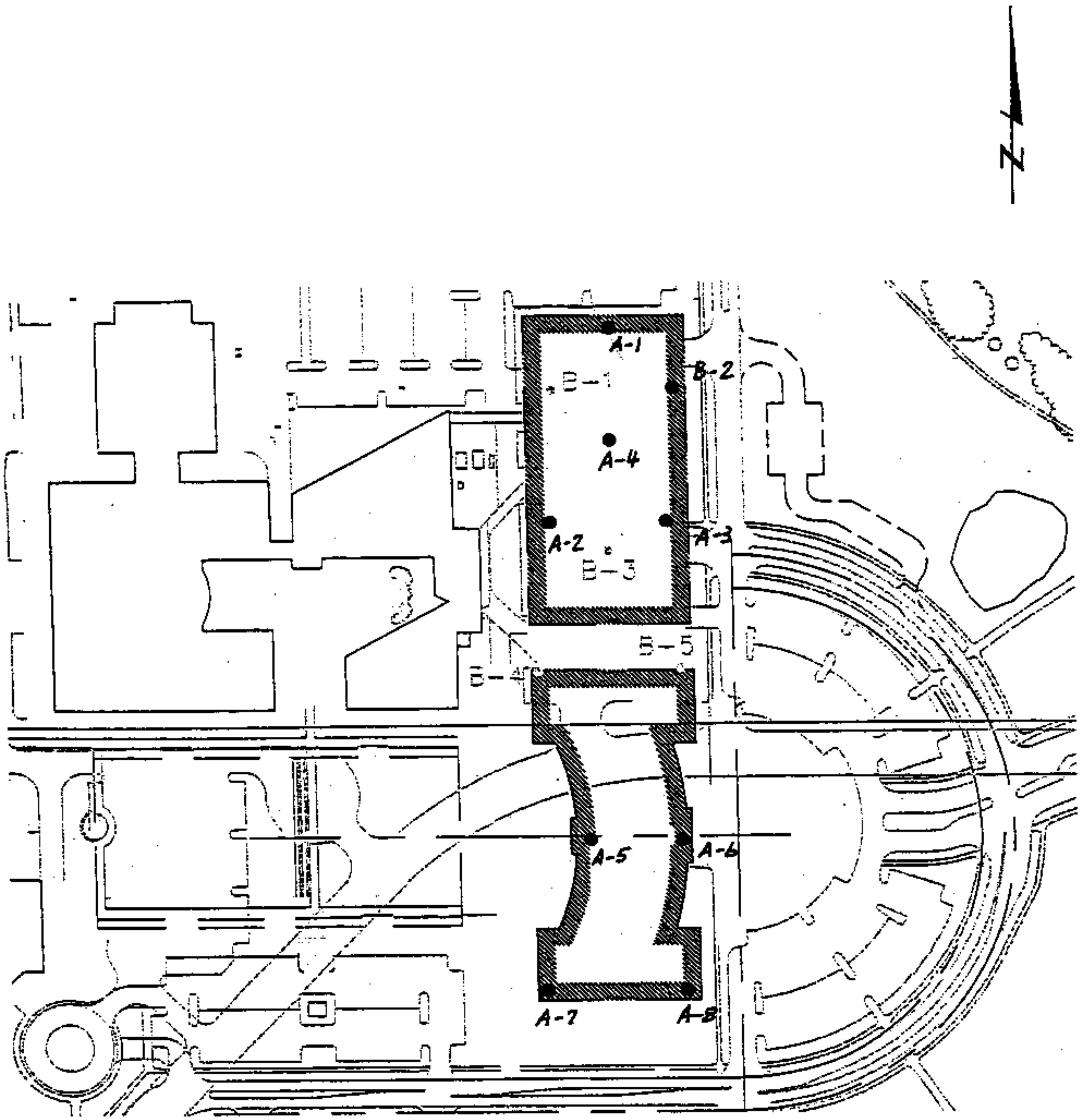
CTC-Geotek, Inc.

By: ______________________
Jie Yu, P.E.
Project Engineer

Reviewed by: ______________________
Robert Scavuzzo, P.E.
Senior Engineer

COLORADO REGISTERED PROFESSIONAL ENGINEER
ROBERT SCAVUZZO
27065

JY:RS
3 copies sent
6 copies to: Michael Barber Architecture
Attn: Mr. Paul Todd

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| BORING LOCATION PLAN | CTC-GEOTEK<br>ENGINEERING TESTING INSPECTION<br>155 S. Navajo • Denver, CO 80223 • 303-698-1050 |

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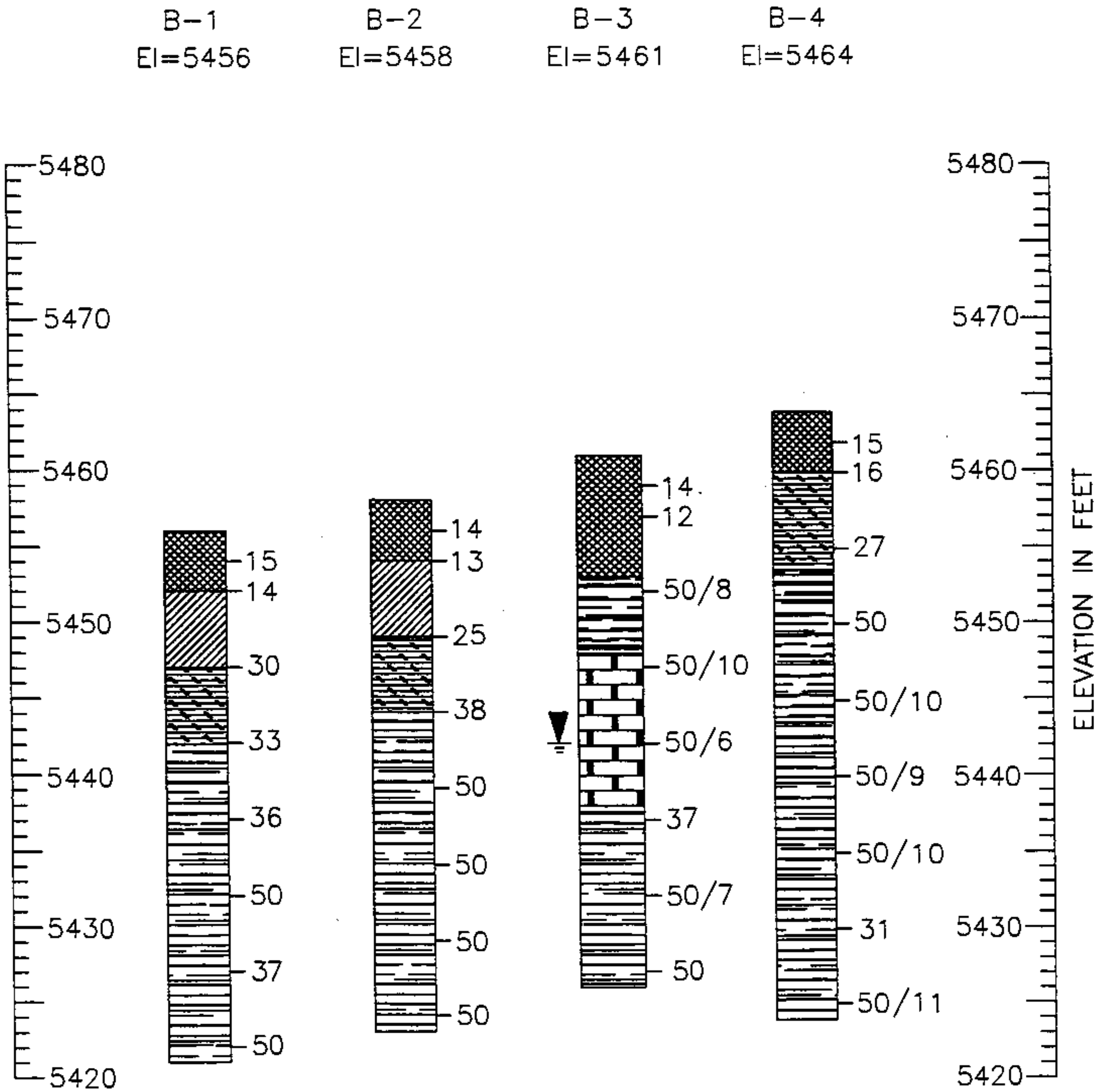


LOGS OF EXPLORATORY BORINGS

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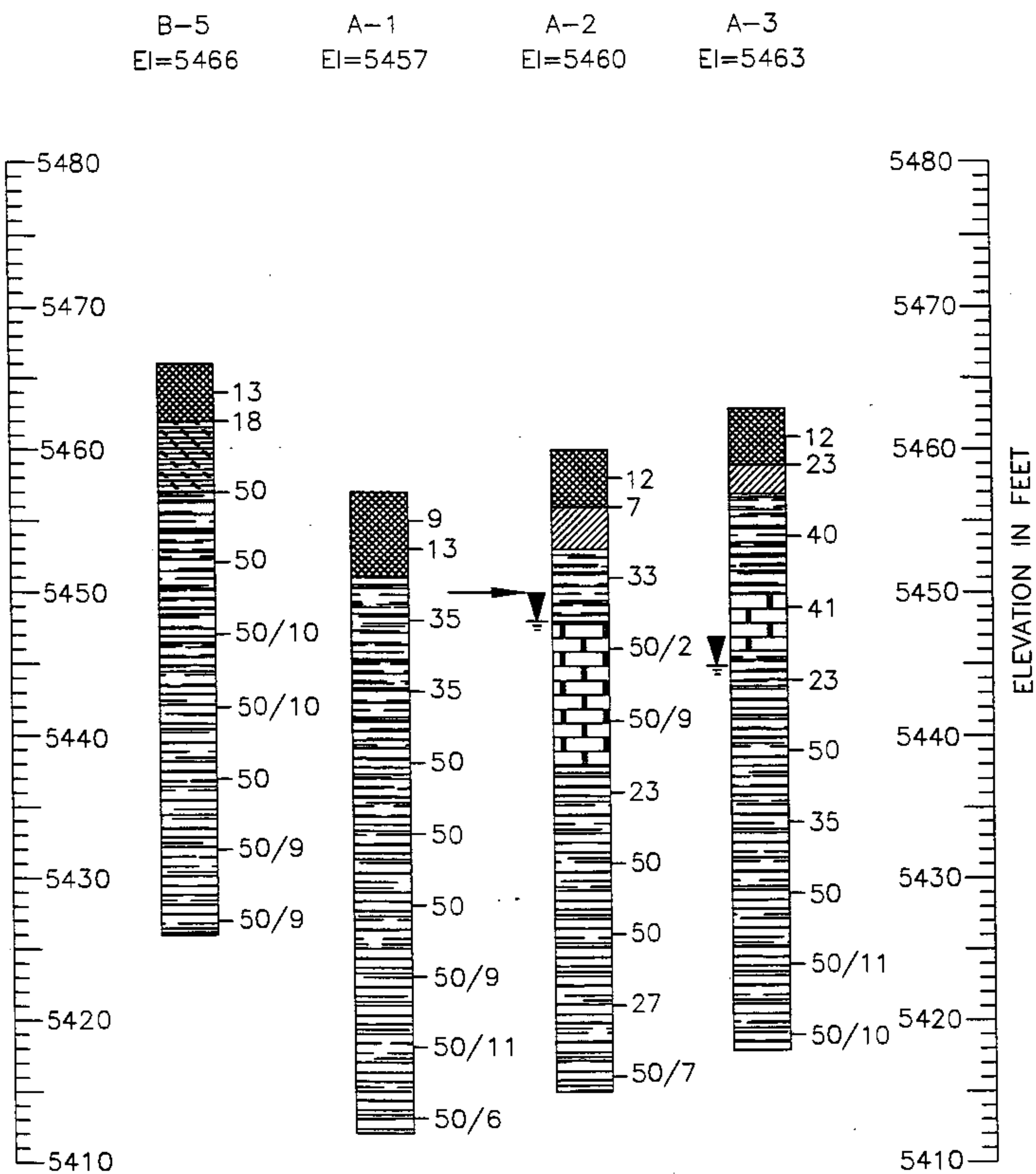


LOGS OF EXPLORATORY BORINGS

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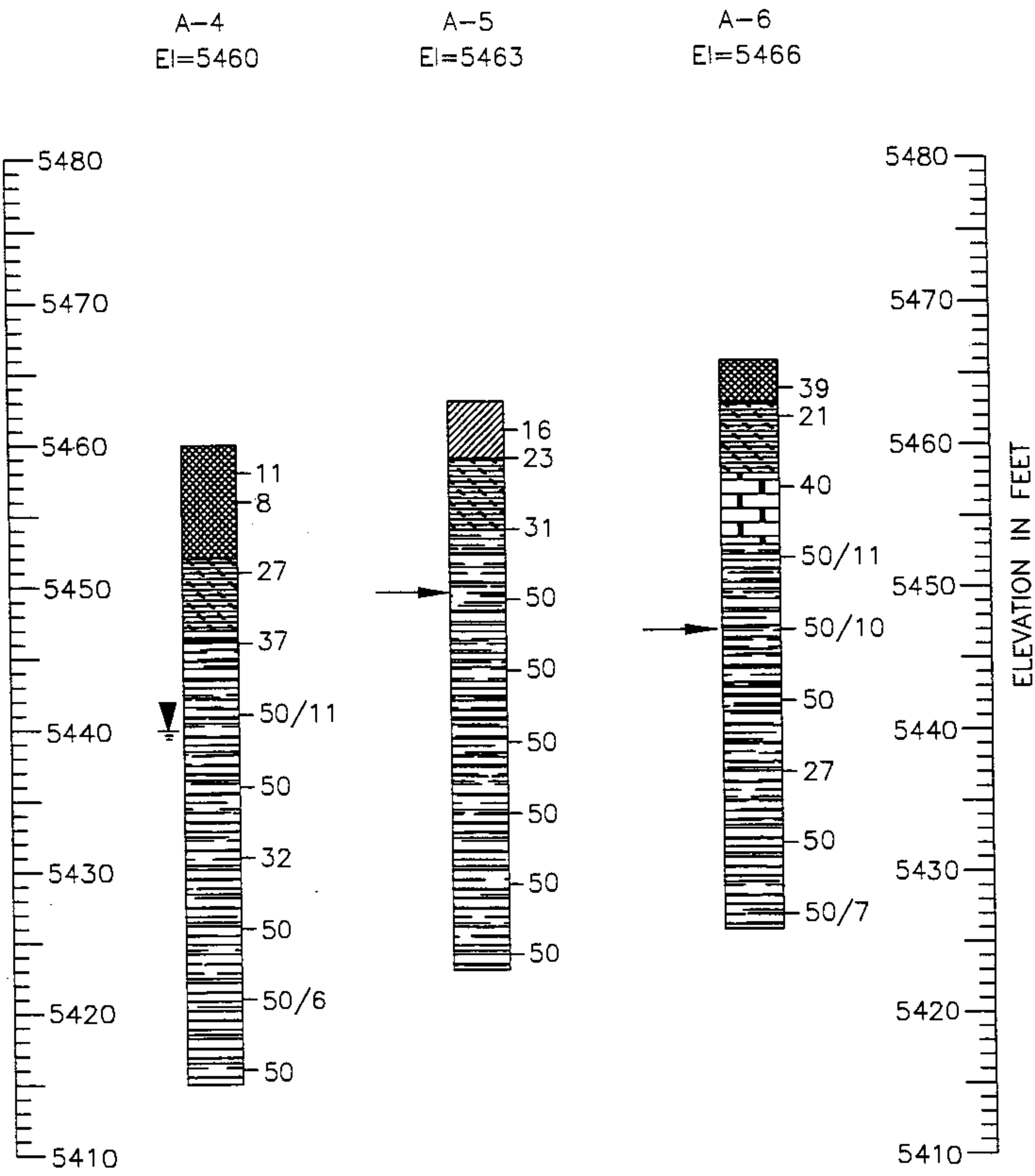
PLATE 3




LOGS OF EXPLORATORY BORINGS

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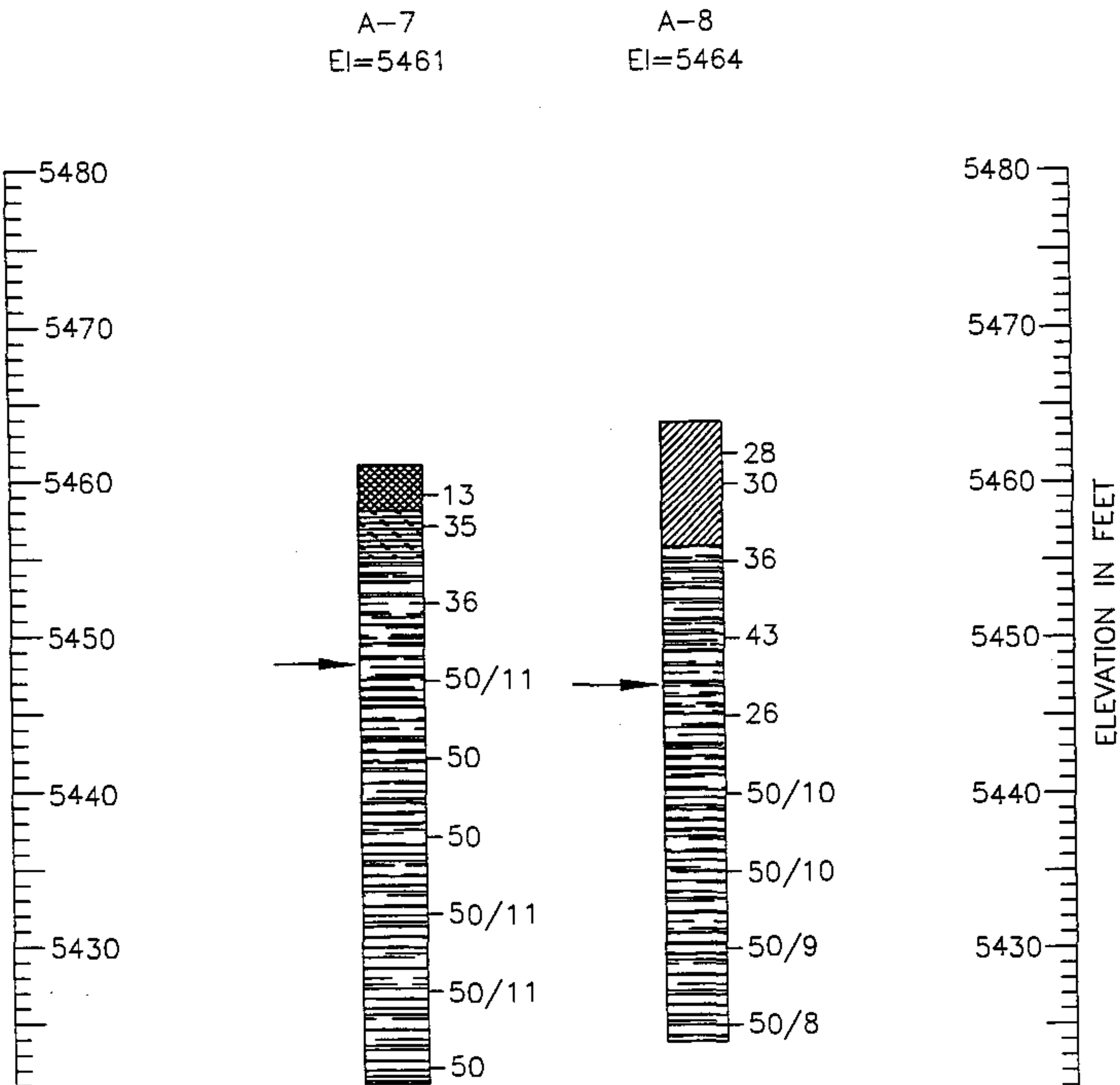


LOGS OF EXPLORATORY BORINGS

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| DATE: 7/12/01 | JOB NO. 212034 | PLATE 5 |

Existing fill, silty clay with some sand, occasionally mixed with claystone materials, dark brown to brown, slightly moist to very moist, firm to stiff.

Silty clay with some sand, natural, brown, moist to very moist, firm to stiff.

Sandstone bedrock, fine to medium grained, yellowish brown to brown, slightly moist to wet, medium hard to very hard.

Claystone bedrock, yellowish grey to grey to brown, moist to very moist, medium hard to hard.

Weathered claystone, grey to brown, moist, firm to stiff.

Groundwater level at the time of drilling

Groundwater level at approximately 2 weeks subsequent to drilling

## NOTES:

(1) Exploratory borings A–1 thru A–8 were drilled on June 15th and 16th, 2001 with 4–inch solid stem power augers. Borings B–1 thru B–5 were drilled in previous investigation.
(2) Stratification lines represent approximate boundaries, actual transition may be gradual.
(3) The logs only show conditions at the time and location indicated.
(4) Groundwater was encountered in the exploratory borings at the depth indicated.
(5) 15 indicates 15 blows of a 140–pound hammer falling 30 inches to drive the sampler 12 inches. 50/8 indicates the number of blows for 8 inches of penetration.

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| LEGEND AND NOTES | CTC–GEOTEK<br>ENGINEERING TESTING INSPECTION<br>155 S. Navajo • Denver, CO 80223 • 303–696–1050 | | |
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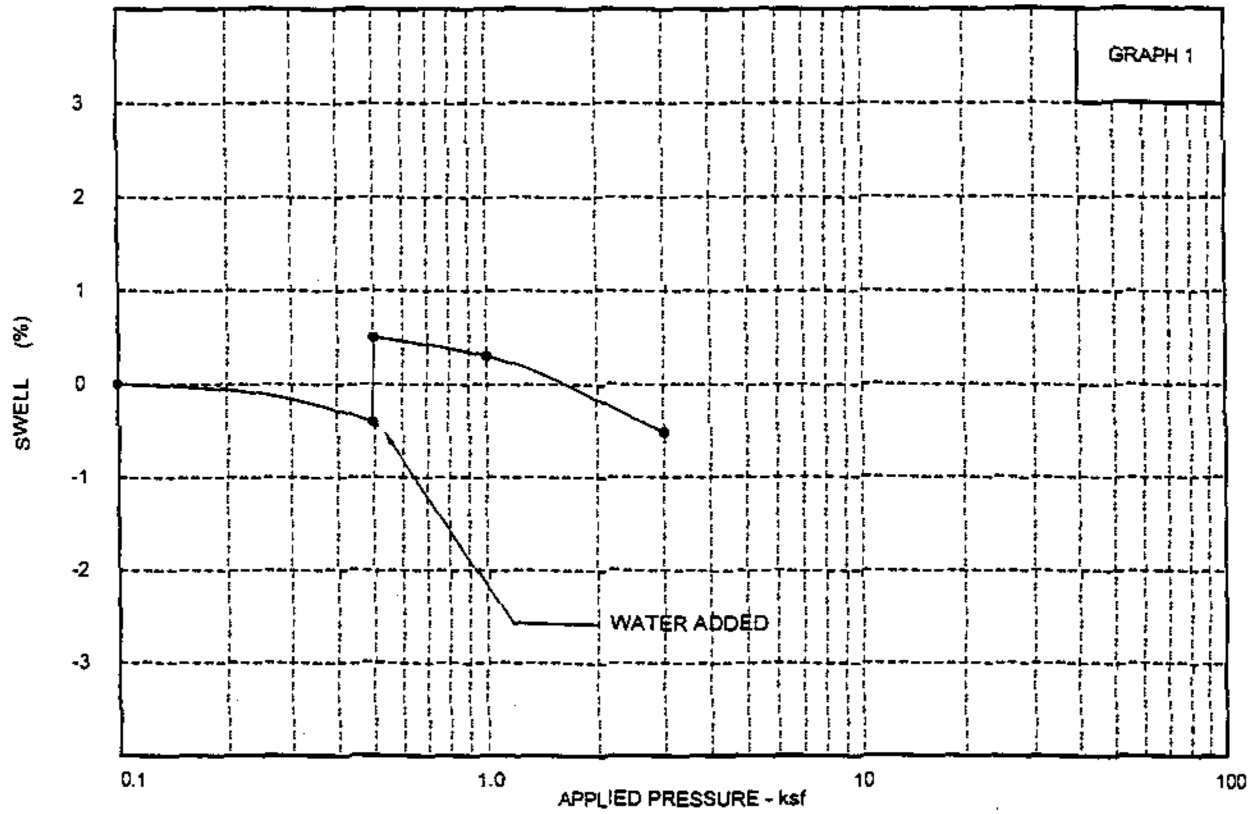


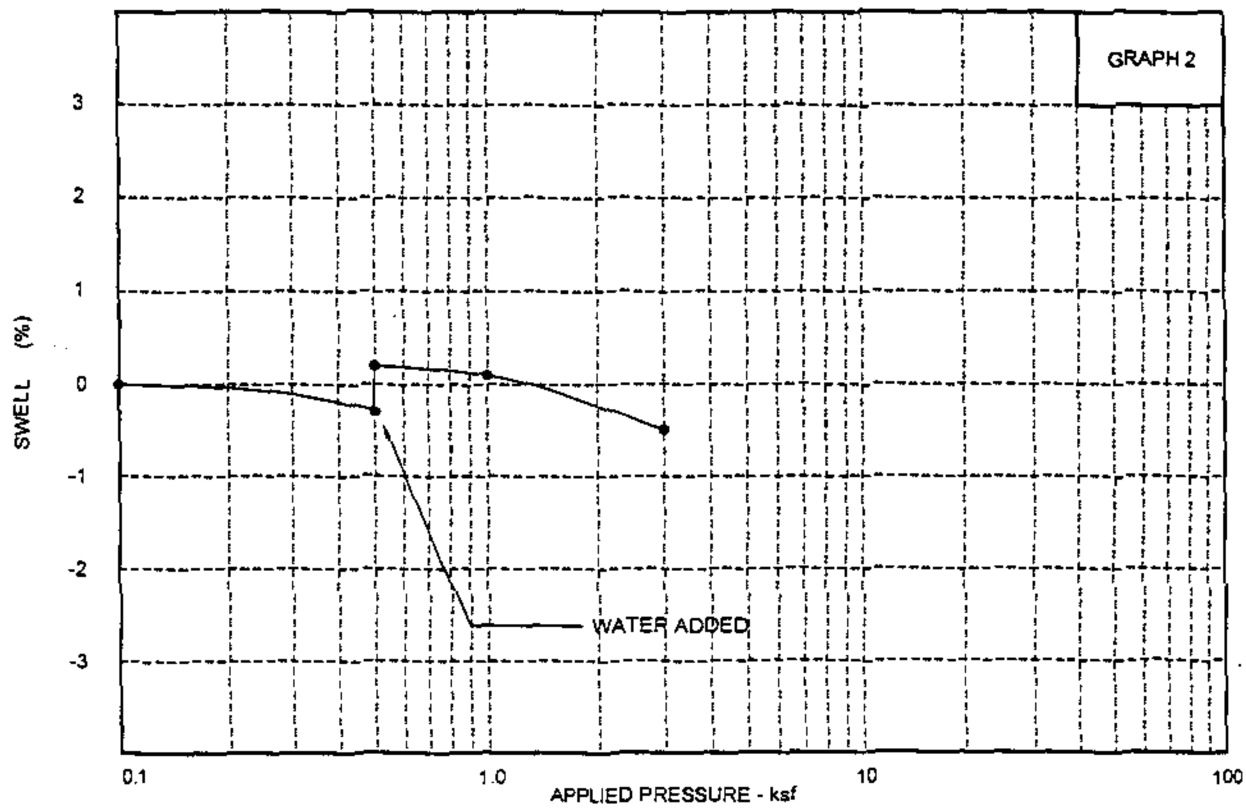


| GRAPH NO. | BORING NO. | SAMPLE NO. | DEPTH IN FEET | DRY DENSITY (PCF) | MOISTURE (%) | SOIL DESCRIPTION |
|---|---|---|---|---|---|---|
| 1 | A2 | C3 | 9 | 92.6 | 28.1 | Claystone |
| 2 | A3 | C2 | 4 | 83.8 | 35.6 | Claystone |

**CTC-GEOTEK**

SWELL - CONSOLIDATION TEST

| | |
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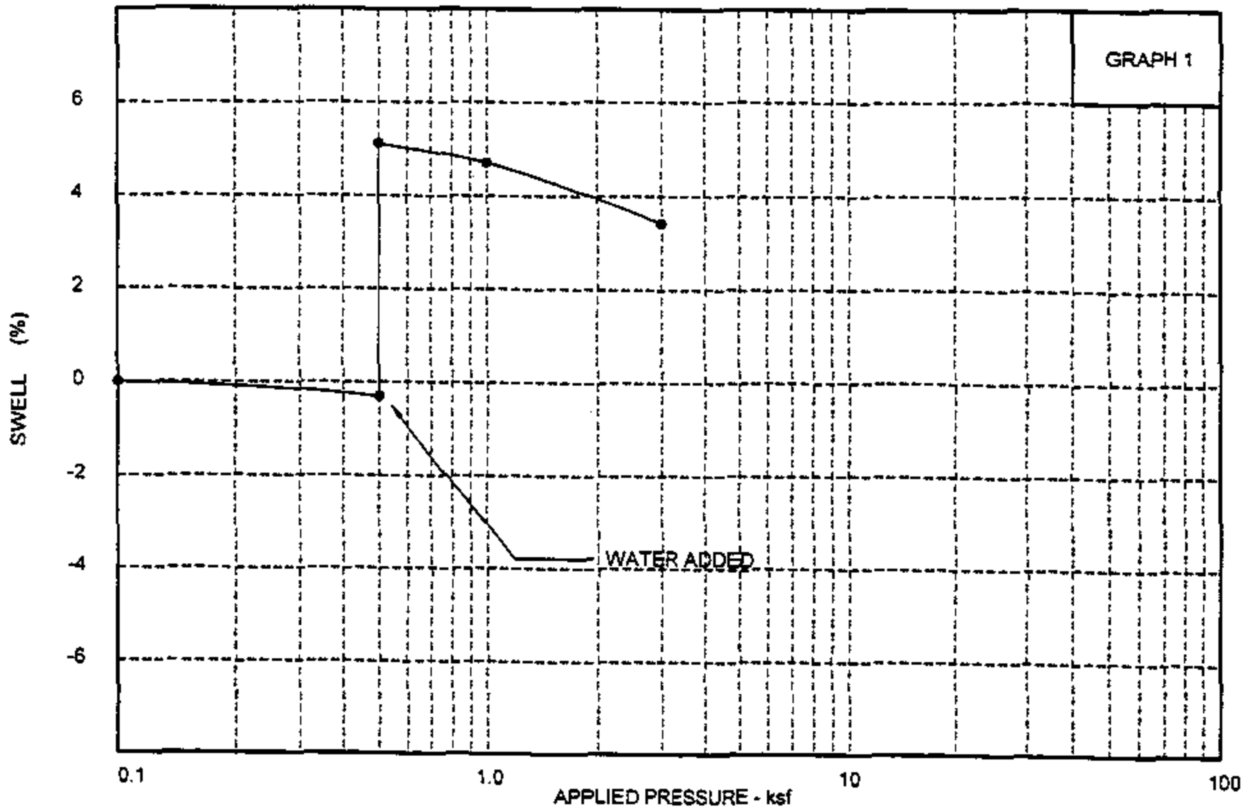
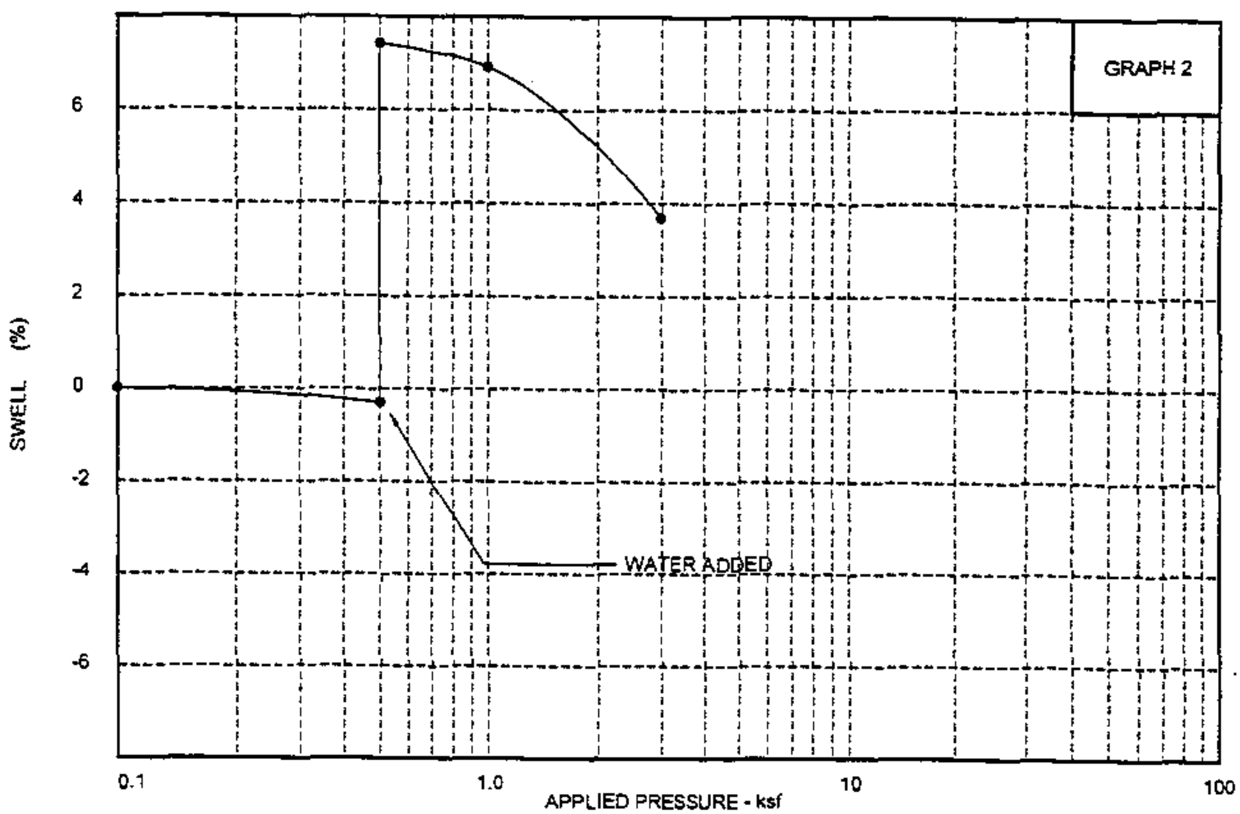
| DRAWN BY: JLW | JOB NO.: 212034 |
| DATE: 7-9-2001 | PLATE: 7 |




| GRAPH NO. | BORING NO. | SAMPLE NO. | DEPTH IN FEET | DRY DENSITY (PCF) | MOISTURE (%) | SOIL DESCRIPTION |
|---|---|---|---|---|---|---|
| 1 | A5 | C1 | 2 | 104.5 | 17.9 | Silty Clay, some sand |
| 2 | A7 | C2 | 4 | 107.9 | 18.2 | Silty Clay, little sand |

**CTC-GEOTEK**

SWELL - CONSOLIDATION TEST

| | |
|---|---|
| DRAWN BY: JLW | JOB NO.: 212034 |
| DATE: 7-9-2001 | PLATE: 8 |

**CTC-GEOTEK**
ENGINEERING TESTING INSPECTION

# SUMMARY OF LABORATORY TEST RESULTS

**Project No. 212034**

| 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 | C4 | 14 | CA | 96.2 | 26.5 | | | | | | $C_1$ 11151.5 | | | Claystone |
| B1 | C7 | 29 | CA | 102.2 | 21.8 | | | | | | $C_1$ 10543.5 | | | Claystone |
| B4 | C8 | 34 | CA | 97.9 | 24.0 | | | | | | $C_1$ 8061.5 | | | Claystone |
| B4 | C9 | 39 | CA | 106.3 | 19.6 | | | | | | $C_1$ 11224.3 | | | Claystone |
| A1 | C6 | 24 | CA | 104.4 | 21.4 | | | | | .052 | | | | Claystone |
| A2 | C3 | 9 | CA | 92.6 | 28.1 | | | | | | | SW | | Claystone |
| A2 | C3 | 39 | CA | 100.3 | 24.9 | | | | | | $C_1$ 10142.2 | | | Claystone |
| A3 | C9 | 4 | CA | 83.8 | 35.6 | 68 | 47 | 21 | 55.3 | | | SW | | Weathered Claystone A-7-6(22) CH |
| A3 | C5 | 19 | CA | 87.7 | 32.4 | | | | | | $C_1$ 7240.6 | | | Claystone |

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

$C_1$ - Unconfined Compression
$C_2$ - Miniature Vane Shear
$C_3$ - Pocket Penetrometer
$C_4$ - 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

**TABLE 1**

CTC-GEOTEK
ENGINEERING TESTING INSPECTION

# SUMMARY OF LABORATORY TEST RESULTS

**Project No. 212034**

| 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 | | | | | | |
| A3 | C7 | 29 | CA | 101.6 | 24.5 | | | | | | $C_1$ 10843.8 | | | Claystone |
| A4 | C9 | 29 | CA | 97.6 | 26.1 | | | | | | $C_1$ 9279.4 | | | Claystone |
| A5 | C1 | 2 | CA | 104.5 | 17.9 | | | | | | | SW | | Silty Clay, some sand |
| A5 | C9 | 39 | CA | 105.5 | 20.4 | | | | | | $C_1$ 10954.3 | | | Claystone |
| A7 | C2 | 4 | CA | 107.9 | 18.2 | 55 | 41 | 14 | 81.3 | | | SW | | Weathered Claystone A-7-6(33) CH |
| A8 | C3 | 9 | CA | 111.4 | 14.1 | | | | | .028 | | | | Claystone |
| A8 | C5 | 19 | CA | 81.2 | 38.4 | | | | | | $C_1$ 4305.8 | | | Claystone |

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

- $C_1$ - Unconfined Compression
- $C_2$ - Miniature Vane Shear
- $C_3$ - Pocket Penetrometer
- $C_4$ - 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

**TABLE 1**