**Chen & Associates**
Consulting Geotechnical Engineers

3405 North El Paso
Colorado Springs, CO 80907
303/636-3304

Casper
Denver
Fort Collins
Glenwood Springs
Rock Springs
Salt Lake City
San Antonio



# PRELIMINARY GEOTECHNICAL INVESTIGATION
# PROPOSED WASTEWATER TREATMENT PLANT
# BIRDSALL ROAD
# EL PASO COUNTY, COLORADO

PREPARED FOR:

GILBERT, MEYER AND SAMS, INC.
611 NORTH WEBER, SUITE 300
COLORADO SPRINGS, COLORADO 80903-1032

ATTN: EDWARD MEYER, P.E.

Job Number 2-411-86

November 5, 1986

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TABLE I - SUMMARY OF LABORATORY TEST RESULTS

## CONCLUSIONS

(1) Beneath the existing topsoil, the subsoil conditions encountered consist of about 16.5 to over 36 feet of slightly sandy to sandy clay. Claystone bedrock was encountered in two test holes at depths of 16.5 and 35 feet to the maximum depth explored, 41 feet. Groundwater was encountered in one test hole.

(2) Spread footings bearing on the natural nonexpansive clays or a layer of structural fill would be feasible at the site. Straight shaft piers drilled into the underlying bedrock may also be feasible.

(3) Additional investigations will be required to provide specific design criteria for foundations, floor slabs, pavements and other soil related activities.

## PURPOSE AND SCOPE OF STUDY

This report presents the results of a preliminary geotechnical investigation for a proposed wastewater treatment plant to be located on the south side of Birdsall Road approximately three-quarters of a mile east of Hanover Road in El Paso County, Colorado (Secs. 27, 28, 33 and 34, T.16S., R.65W., Buttes Quadrangle). The project site is shown on Figure 1. The study was conducted in accordance with our proposal to Gilbert, Meyer and Sams, Inc., dated August 15, 1986.

This report has been prepared to summarize the data obtained and to present our conclusions and preliminary recommendations based on the subsurface conditions encountered. Preliminary design parameters and a discussion of geotechnical engineering considerations related to construction at the site are included.

## PROPOSED CONSTRUCTION

We understand the proposed wastewater treatment facility will consist of a two-million gallon per day oxidation ditch, secondary clarifiers, an operations building, sludge digester and other appurtenances. It is anticipated that the structures will be of reinforced concrete construction.

## SITE CONDITIONS

The site is currently vacant and vegetated with native grasses and weeds. The ground surface on the site slopes gently down to the south and east. The maximum elevation difference across the site is on the order of 60 feet.

There is an existing irrigation ditch in the southern portior of the site which crosses the site from the northwest down to the southeast. The ditch is no longer in use and has been eroded to a maximum depth of 25 feet. Several small lateral drainages extend out from the ditch to the northeast and southwest. The drainages and irrigation ditch were dry at the time of our observation.

## SUBSURFACE CONDITIONS

The subsurface conditions were investigated by drilling ten test holes at the approximate locations shown on Figure 1. Logs of the exploratory holes are shown on Figures 2 and 3, and Table I presents a summary of laboratory test

results. Generally the subsurface conditions encountered beneath the existing topsoil consist of slightly sandy to sandy clay. Claystone bedrock was encountered in two of the test holes.

Slightly sandy to sandy clay was encountered in all the test holes to depths ranging from 16.5 to over 36 feet. Penetration test results indicate the clays are stiff to hard in consistency. Some of the clays have a porous structure. The results of swell-consolidation testing presented on Figures 4 thru 6 indicate the clays have a nil to moderate swell potential. One of the samples of clay consolidated or collapsed upon loading and wetting. The tendency to collapse is typical of soils with a porous structure.

Claystone bedrock was encountered in Test Holes 3 and 6 at depths of 31 and 16.5 feet to the maximum depth explored, 41 feet. The results of penetration testing indicates the bedrock is hard to very hard. The results of a swell-consolidation test presented on Figure 5 indicates the bedrock has a high swell potential.

Groundwater was encountered in Test Hole 3 at a depth of 20 feet at the time of drilling and at a depth of 16.5 feet when measurements were taken one day later. The remaining holes were dry and open to near their completion depths.

FOUNDATION TYPES

The clays encountered on the site have erratic swell-consolidation characteristics. Some of the near surficial clays have a porous structure and tend to collapse upon loading and wetting. Because of their tendency to collapse, these materials have a relatively low bearing capacity. Some of the clays possess a low to moderate swell potential. Shallow foundations placed on or near these soils would be subject to movement should the soils become wetted and expand. We believe spread footings will be a feasible type of foundation system if the risk of some foundation movement can be tolerated. Placing the footings on a layer of nonexpansive fill would help to reduce the potential movement caused by the collapsible or expansive materials. This would require overexcavation of these materials from beneath foundation areas and replacing them with nonexpansive fill. We anticipate the depth of overexcavtion would be on the order of 3 to 5 feet. It may also be feasible to place spread footings directly on the low swelling clays or clays that have a low collapse potential.

We anticipate allowable bearing pressures for footings will be in the range 1200 to 3000 psf. The lower bearing capacities would be for footings placed directly on the collapsible materials. Minimum dead load pressures would be required for footings placed on the expansive clays.

Where bedrock is relatively shallow, straight shaft piers drilled into the bedrock may be a feasible type of foundation system. A drilled pier foundation system would place the bottom of the piers in a zone of relatively stable moisture content and make it possible to load the piers sufficiently to resist uplift movements caused by the expansive overburden soils or bedrock. We anticipate straight shaft piers drilled into the bedrock may be designed for an allowable end bearing pressure in the range of 20,000 to 40,000 psf and a skin friction for that portion of the pier in bedrock of about 10% of the end bearing pressure. Minimum pier lengths and minimum dead load pressures would be required for drilled pier foundations. Casing and dewatering of the pier holes may be required in some areas.

FLOOR SLABS

Slab-on-grade construction appears to be feasible on the site. However, there is a risk of movement where the slabs are supported by expansive or collapsible materials. Overexcavation of these materials and replacement with a layer of nonexpansive fill would help to reduce the risk of movement caused by the collapsible or expansive soils. Where feasible, the slabs should also be constructed as free floating. If free floating slabs are not feasible, the layer of nonexpansive fill may be increased in depth. The

only way to eliminate the risk of slab movement due to the collapsible or expansive materials would be the construction of structural slabs above crawl spaces.

UNDERDRAIN SYSTEM

All below ground levels extending near the existing water level or bedrock should be protected by an underdrain system, or waterproofed and designed to withstand hydrostatic uplift. We anticipate underdrain systems will be required where the lower level extends to within 5 feet of the existing water level or bedrock. The underdrain system would consist of free draining granular material beneath the floor slabs connected to perimeter drains.

SITE GRADING

The extent of site grading necessary for the project had not been determined at the time of this study. The general criteria presented below may be used for developing site grading plans. We should review the final site grading plans once they are completed.

Cut and Fill Slopes: Permanent, unretained cuts less than 15 feet in height should be stable on a slope of 2 horizontal to 1 vertical if well drained. Higher slopes may be feasible but should be investigated on an individual basis. If seepage is encountered in permanent cuts, an investigation should be conducted to determine if the seepage will adversely affect slope stability.

Fills up to 15 feet in height can be used if the fill slopes do not exceed 2 horizontal to 1 vertical, and the fills are properly compacted and drained. The ground surface underlying all fill areas should be prepared by removing organics, scarifying to a depth of 8 inches and recompacting to 95% of the maximum standard Proctor density (ASTM D-698) at a moisture content near optimum. Fill should be benched into hillsides steeper than 4 horizontal to 1 vertical.

Good surface drainage should be provided around all permanent cuts and fills to direct surface drainage runoff away from the cut or fill face. The grading plans should also provide positive surface drainage away from all buildings and pavement areas. Cut and fill slopes, and other stripped areas should be protected by revegetation or other means.

Fill Materials and Placement: All stuctural fill placed beneath the building areas should be a nonexpansive granular material approved by the soil engineer. Fill should be free of trash, debris, organics and other deleterious substances. The on-site clays and claystone bedrock will be expansive when placed in a compacted condition and are not suitable for use as a structural fill beneath building areas.

Fill placed for support of the foundations should be compacted to at least 100% of the maximum standard Proctor

density at a moisture content near optimum. Fill placed for support of floor slabs and pavements should be compacted to at least 95% of the maximum standard Proctor density.

Excavation Considerations: Excavation into the upper overburden soils can be accomplished with conventional excavating equipment. Excavation into the underlying bedrock will be more difficult and may require the use of heavy duty excavating equipment.

When excavating near the existing water level, the soils may pump or deform under the wheel loads of construction equipment. The use of track-mounted equipment is normally recommended since they generally exert lower contact pressures than pneumatic tires. Disturbed areas will require stabilization or removal prior to placing fill material.

## ADDITIONAL INVESTIGATIONS

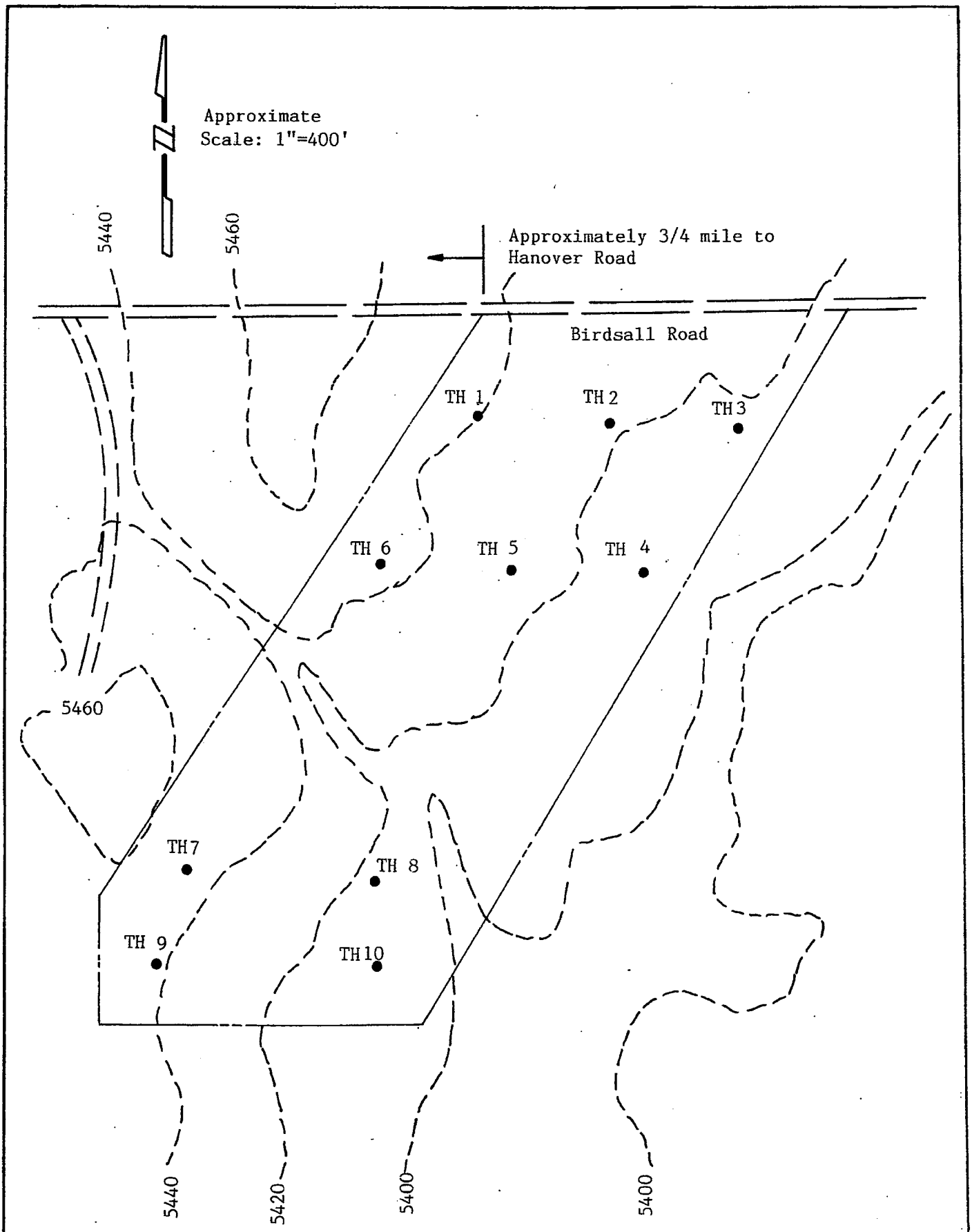
This report has been prepared in accordance with generally accepted geotechnical engineering practices in this area for use by the client for preliminary design and planning purposes. The conclusions and preliminary recommendations submitted in this report are based upon the data obtained from widely spaced exploratory borings drilled at the locations indicated on the exploratory hole plan. Additional investigations must be conducted to provide final recommendations once building locations and floor slab elevations have been determined.


CHEN & ASSOCIATES, INC.

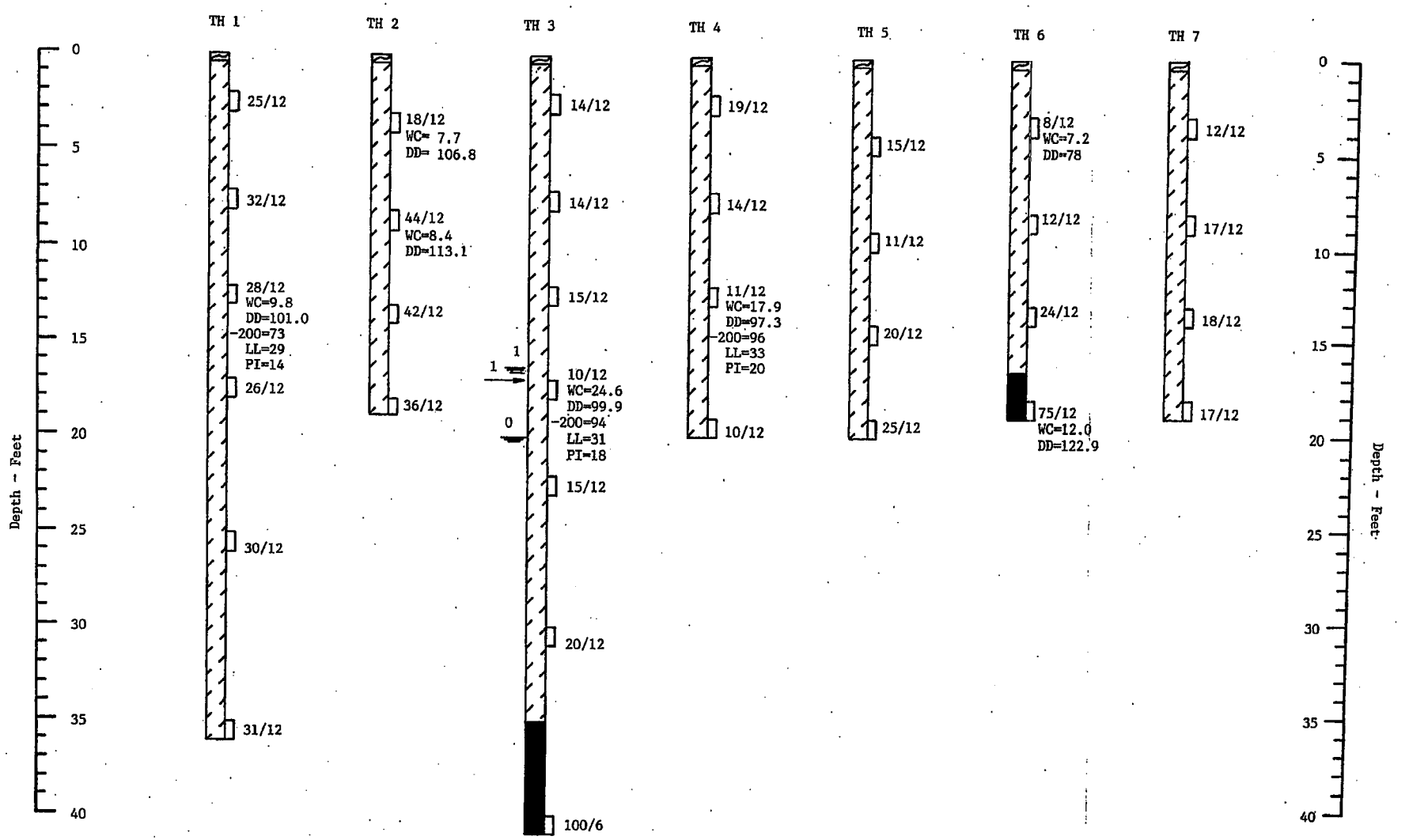
By Mark R. Matheny
Mark R. Matheny, P.E.

Reviewed by Bruce E. Berends
Bruce E. Berends, P.E.

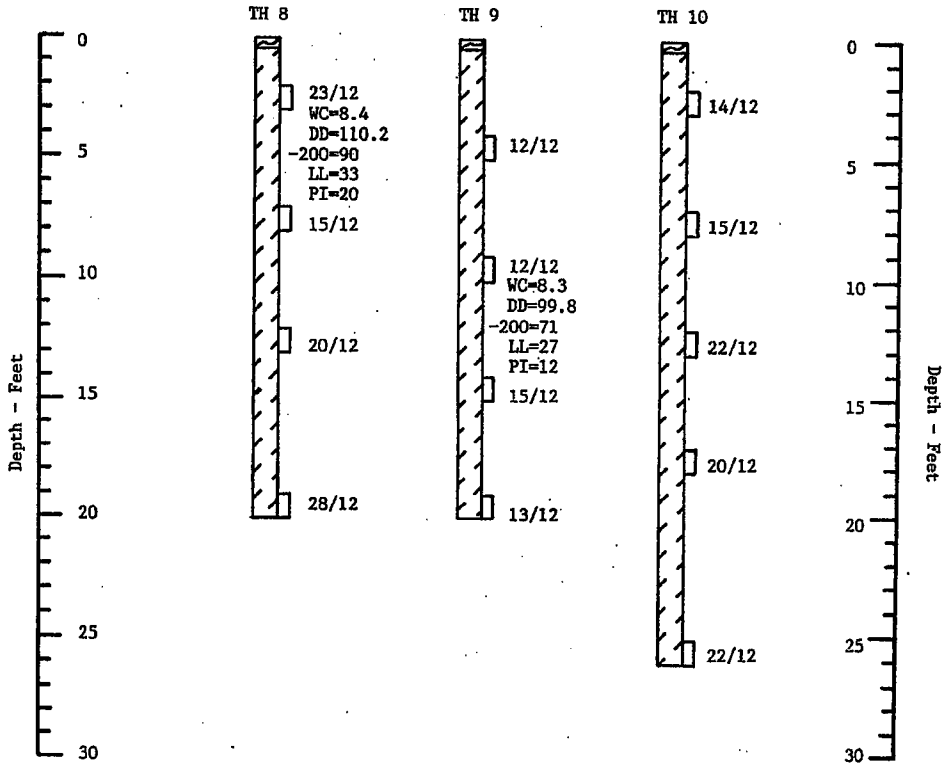
MRM:lm




| 2-411-86 | **Chen & Associates** | LOCATION OF EXPLORATORY HOLES | **Fig.** 1 |
|---|---|---|---|




| 2-411-86 | Chen & Associates | LOGS OF EXPLORATORY HOLES | Fig. 2 |
|---|---|---|---|




LEGEND

Topsoil; Clay (CL), sandy, slightly moist, brown.

Clay (CL), slightly sandy to sandy, occasional clayey sand lenses, locally porous and calcareous, stiff to hard, dry to wet, brown.

Claystone bedrock, hard to very hard, slightly moist to moist, olive to brownish gray.

Drive sample, 2-inch I.D. California liner sample.

25/12 Drive sample blow count, indicates that 25 blows of a 140-pound hammer falling 30 inches were required to drive the California sampler 12 inches.

1 / 1 Depth to water level and number of days after drilling measurement was taken.

→ Depth at which test hole caved and number of days after drilling measurement was taken.

Laboratory Test Results:

WC=Water Content (%);
DD=Dry Density (%);
-200=Percentage passing No. #200 sieve;
LL=Liquid Limit (%);
PI=Plasticity Index (%).

NOTES

1. Test holes were drilled on 10-22-86 with a 4-inch diameter continuous flight power auger.
2. Locations of test holes were provided by Gilbert, Meyer and Sams.
3. Elevations of test holes were not measured and logs of test holes are drawn to depth.
4. The lines between materials shown on the test hole logs represent the approximate boundaries between material types and the transitions may be gradual.
5. Water level readings shown on the logs were made at the time and under conditions indicated. Fluctuations in the water level may occur with time.

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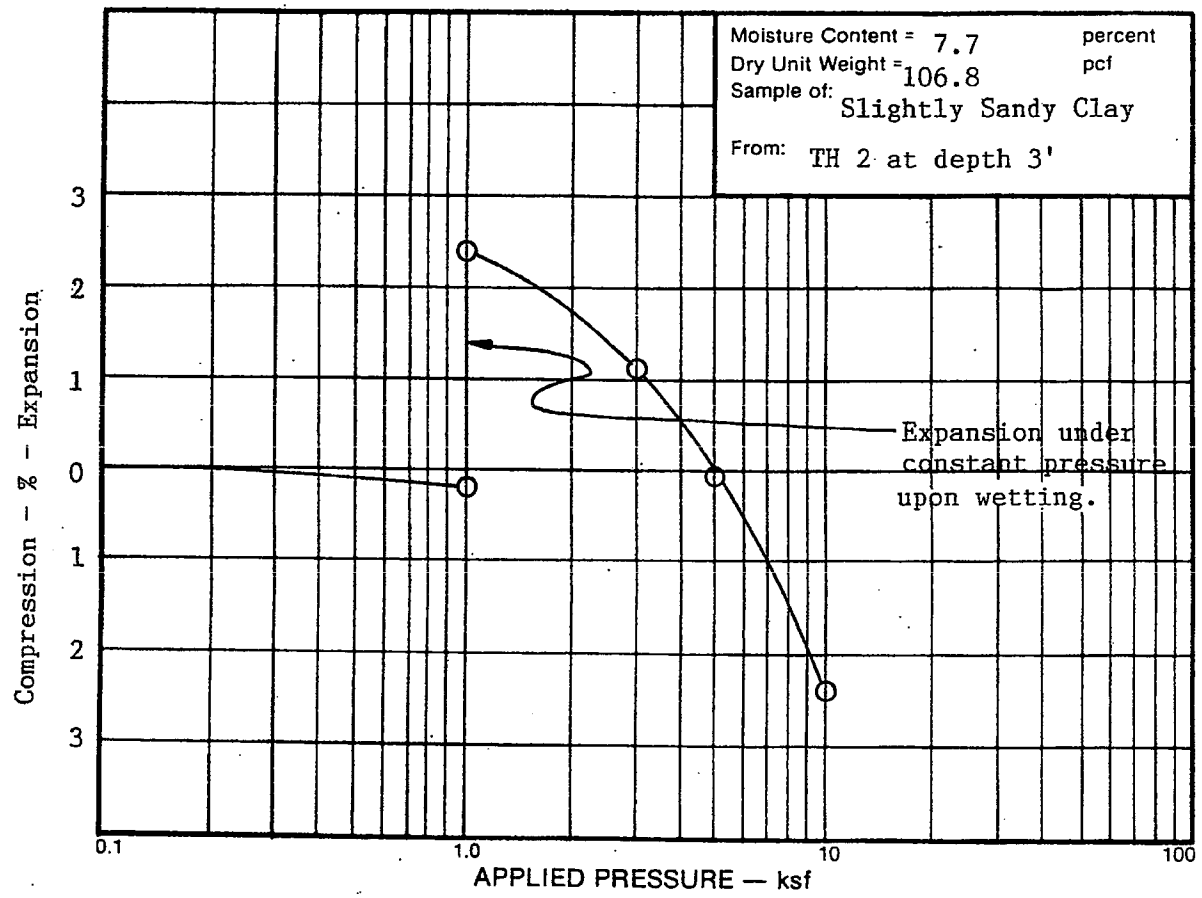


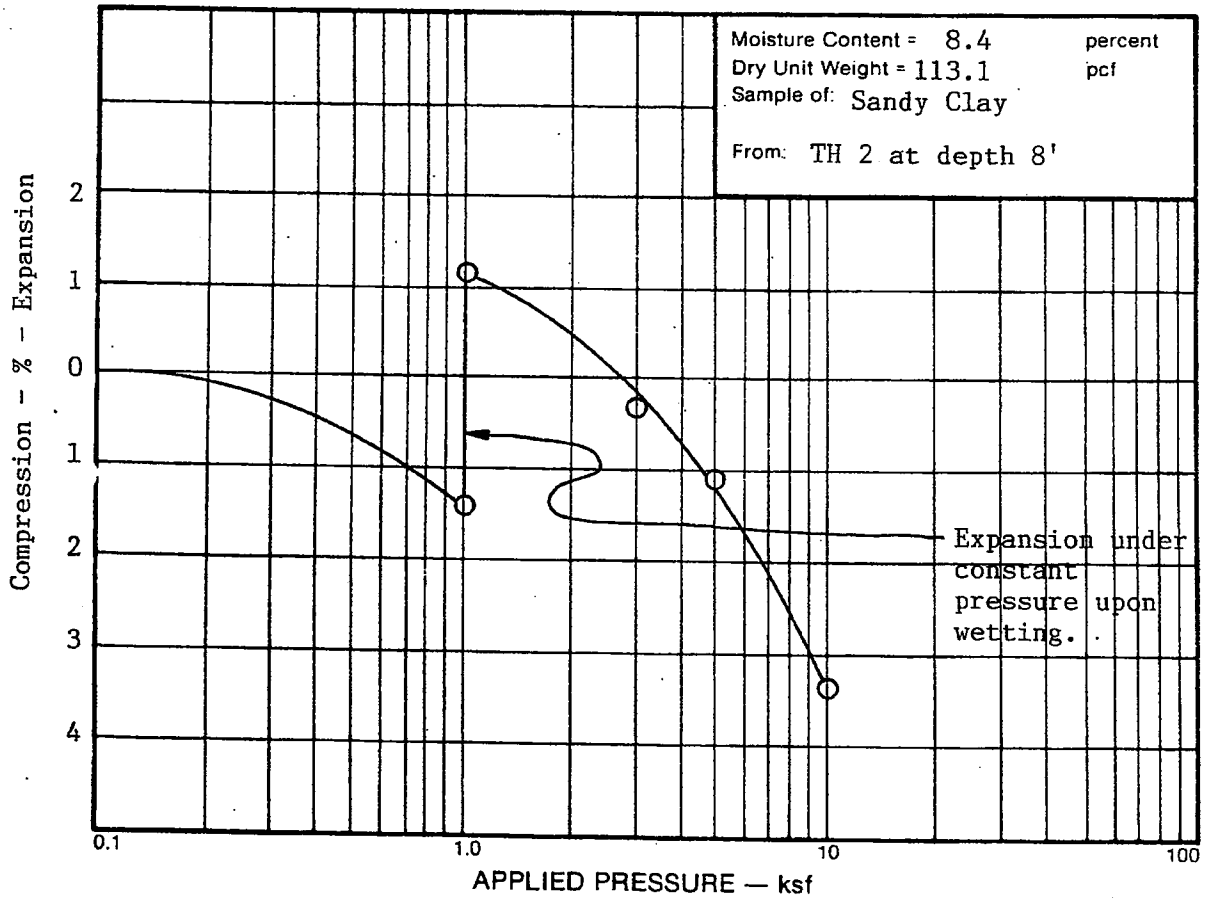


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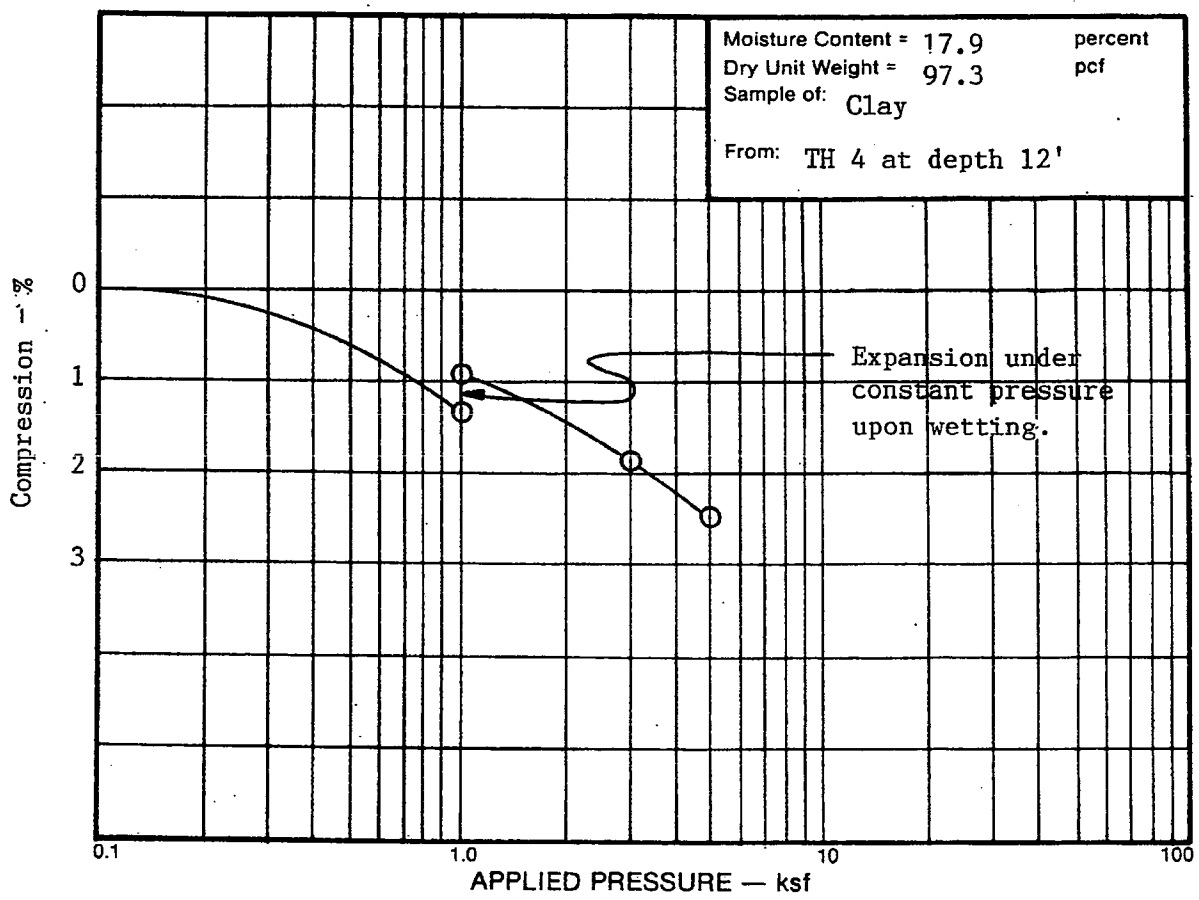
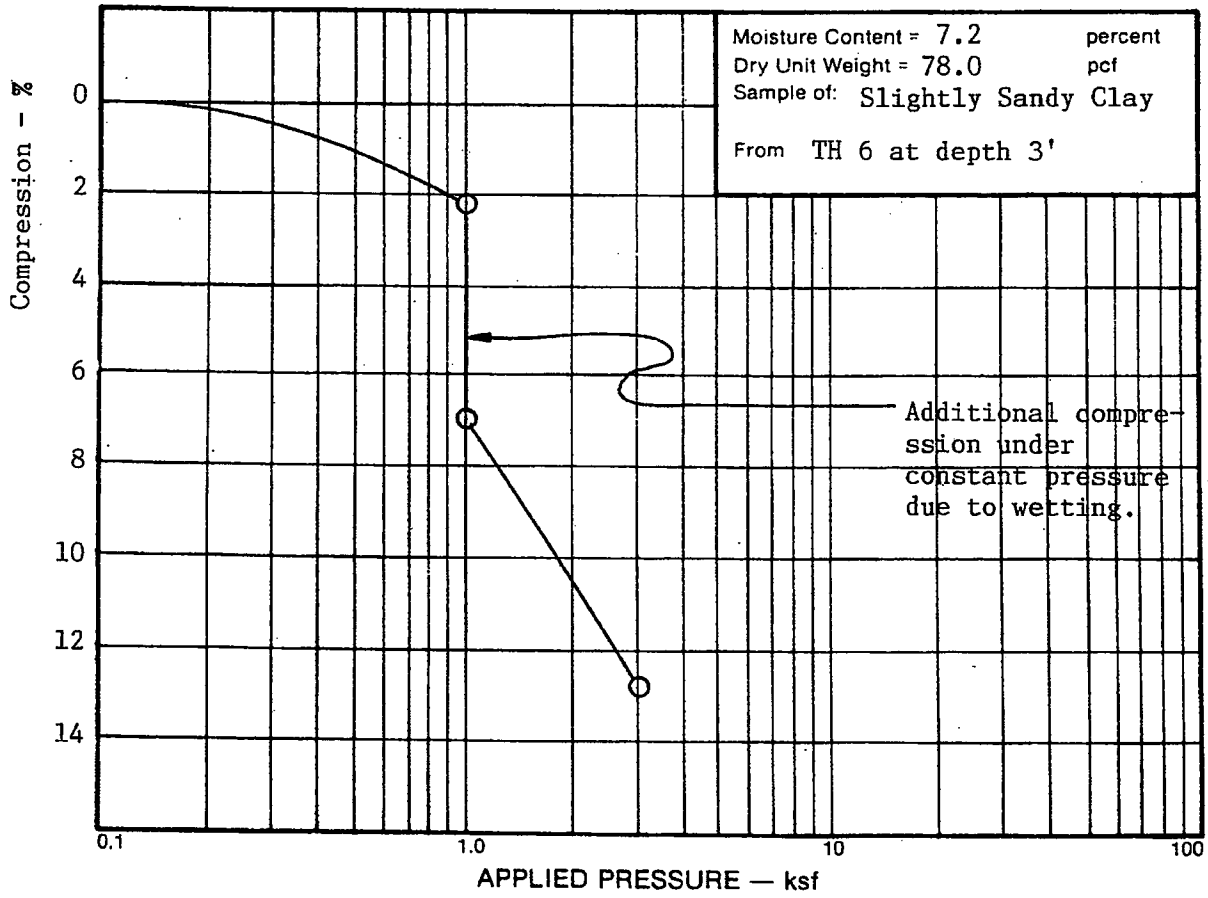
SWELL-CONSOLIDATION TEST RESULTS

Fig. 4

# chen and associates, inc.

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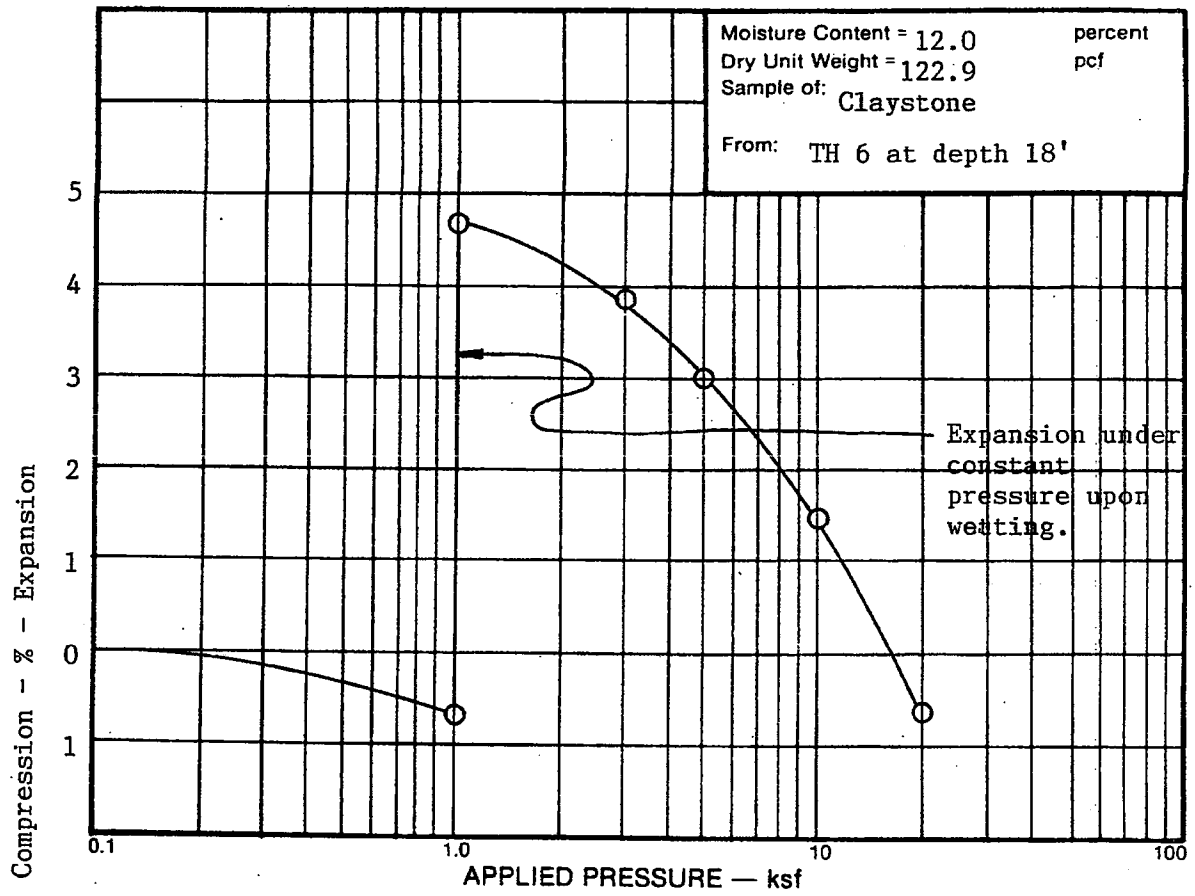
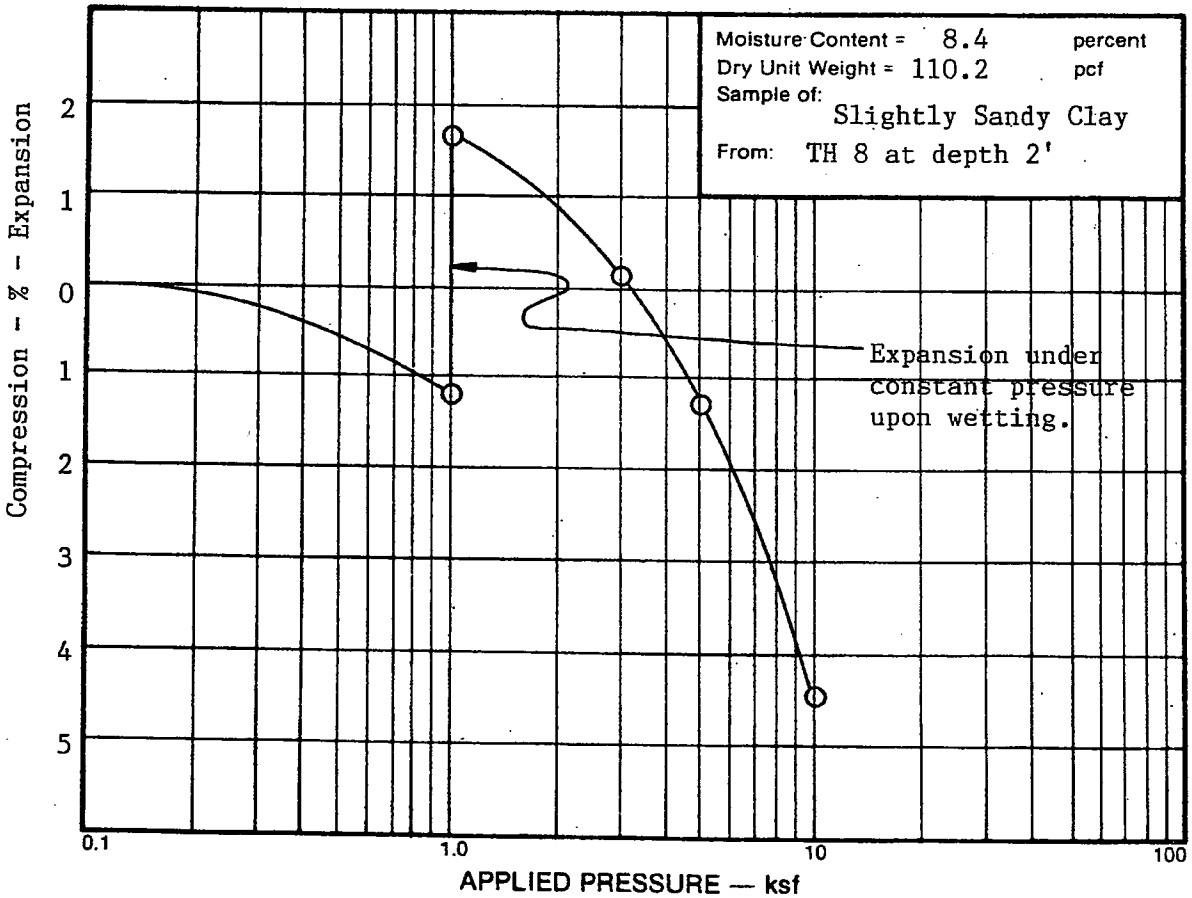
SWELL-CONSOLIDATION TEST RESULTS

Fig. 5

# chen and associates, inc.

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SWELL-CONSOLIDATION TEST RESULTS

Fig. 6

CHEN AND ASSOCIATES

# TABLE I

## SUMMARY OF LABORATORY TEST RESULTS

| SAMPLE LOCATION | | NATURAL MOISTURE CONTENT (%) | NATURAL DRY DENSITY (PCF) | GRADATION | | PERCENT PASSING NO. 200 SIEVE | ATTERBERG LIMITS | | UNCONFINED COMPRESSIVE STRENGTH (PSF) | | SOIL OR BEDROCK TYPE |
|---|---|---|---|---|---|---|---|---|---|---|---|
| HOLE | DEPTH (FEET) | | | GRAVEL (%) | SAND (%) | | LIQUID LIMIT (%) | PLASTICITY INDEX (%) | | | |
| 1 | 12 | 9.8 | 101.0 | | | 73 | 29 | 14 | | | Sandy Clay |
| 2 | 3 | 7.7 | 106.8 | | | | | | | | Slightly Sandy Clay |
| | 8 | 8.4 | 113.1 | | | | | | | | Sandy Clay |
| 3 | 17 | 24.6 | 99.9 | | | 94 | 31 | 18 | | | Slightly Sandy Clay |
| 4 | 12 | 17.9 | 97.3 | | | 96 | 33 | 20 | | | Clay |
| 6 | 3 | 7.2 | 78.0 | | | | | | | | Slightly Sandy Clay |
| | 18 | 12.0 | 122.9 | | | | | | | | Claystone |
| 8 | 2 | 8.4 | 110.2 | | | 90 | 33 | 20 | | | Slightly Sandy Clay |
| 9 | 9 | 8.3 | 99.8 | | | 71 | 27 | 12 | | | Sandy Clay |

# ATTACHMENT NO. 8

## SUPPLEMENTAL GEOTECHNICAL INFORMATION

## Letter Report Dated April 28, 2007

## Prepared by Hepworth-Pawlak Geotechnical, Inc.

HP Geotech

Hepworth-Pawlak Geotechnical, Inc.
P.O. Box 388
Monument, Colorado 80132-0388
Phone: 719-633-5562

Fax: 719-633-5582
email: hpgeo3@hpgeotech.com

April 28, 2007

Roger Sams
GMS, Inc.
611 North Weber, Suite 300
Colorado Springs, Colorado 80903

Subject: Wastewater Treatment Facility
Lower Fountain Metropolitan Sewage Disposal District (LFMSDD)
Birdsall Road East of Old Pueblo Road
El Paso County, Colorado

Job Number: 3070112

Dear Mr. Sams:

As requested, we have performed a field reconnaissance at the site of the proposed Wastewater Treatment Facility for the LFMSDD. We also reviewed the Preliminary Geotechnical Investigation prepared for the site by Chen & Associates, Inc. (job number 2-411-86, report dated November 5, 1986).

Proposed Construction: The proposed wastewater treatment facility will be located on the southern portion of the site. The facility will consist of cast-in-place concrete tankage and an operations building. No lagoons are planned.

Site Conditions: The site is currently vacant and has been used as a cattle pasture. Vegetation consists of native grass, weeds, cactus and a few deciduous trees. The ground surface is generally flat with a slight slope down to the south and southeast. The maximum elevation difference across the site is approximately 60 feet. A large steeply sided channel crosses the southern portion of the site from the northwest down to the southeast. The channel leads to a shallow drainage with several small earthen embankments located south and east of the site. The channel slopes are near vertical in many areas and show signs of erosion. The channel varies from about 30 to 100 feet wide and is up to about 25 feet deep. Several embankments and channels having a height or depth of a few feet extend out from the large channel. No water was observed in either the large channel or the smaller channels. Except for the channels and embankments, it does not appear that any significant grading has occurred on the site. The site appears to relatively unchanged since the Preliminary Geotechnical Investigation was performed. Properties adjacent to the site are currently vacant and have also been used as cattle pastures.

Geotechnical Investigation: During the Preliminary Geotechnical Investigation performed by Chen & Associates, 10 exploratory borings were drilled on the site. Borings TH 7 through TH 10 were drilled on the southern portion of the site in the area of the proposed treatment plant. The soils encountered in TH 7 through TH 10 consisted of slightly sandy to sandy clay which was encountered to the maximum depth drilled of 25 feet. The results of swell-consolidation tests indicate the clay possesses a low to moderate swell potential.

One of the clay samples tested from TH 6 exhibited a collapse potential upon loading and wetting. In borings TH 3 and TH 6, claystone bedrock was encountered below the clay at depths of 35 and 16½ feet, respectively. The results of a swell-consolidation test indicate the claystone possesses a high swell potential.

Spread footings were recommended in the report for support of structures. To reduce foundation movements due to swelling and/or collapsible soils, bearing the footings on a layer of nonexpansive structural fill was recommended. In areas where bedrock was shallow, founding the structures on drilled piers was discussed as a foundation alternative.

Conclusions: Based on our site reconnaissance, it appears that the Preliminary Geotechnical Investigation performed by Chen & Associates is still applicable for the site. Based on the site conditions observed and the results reported in the Preliminary Geotechnical Investigation, the site will support the proposed facility subject to following the preliminary recommendations in the report. As discussed in the report, measures to mitigate the effects of expansive/collapsible soils will be required. The effects of the large channel on the proposed construction will also need to be addressed during final design.

The Chen report is preliminary in nature and intended for general design and planning. Additional investigations will be required to provide specific design criteria for foundations, floor slabs, pavements and other soil related activities.

If there are any questions, please feel free to contact us.

Sincerely,

HEPWORTH-PAWLAK GEOTECHNICAL, Inc.

COLORADO REGISTERED DAVID A. HERMAN 16613 PROFESSIONAL ENGINEER

David A. Herman, P.E.

3070112.doc