Example Soils Report

December 11, 1996

Archdiocese of Colorado
200 Vine Street
Denver, Colorado 80206

Subject: Additions to Saint Joseph Seminary
South Main Street and East Tennessee Avenue
Denver, Colorado
Job No. 96-274

This letter presents the results of our Soils and Foundation Investigation for the proposed four-story, stair tower addition and the masonry portal addition to Saint Joseph Seminary, in Denver, Colorado. The purpose of this investigation was to evaluate subsurface conditions at the site, assist the Structural Engineer in foundation design and provide foundation, slab-on-grade and subsurface drainage recommendations for the proposed construction. Our recommendations were developed based on data from our field and laboratory investigations, experience with similar projects, and our understanding of the proposed construction.

Site Conditions

Saint Joseph Seminary is located between East Tennessee Avenue and East Texas Avenue and between South Main Street and South Rose Street in Denver, Colorado (Fig.1). The Seminary consists of several buildings. The additions will be constructed to the Catholic Pastoral Center building which is located in the south portion of the site and was built around 1956. We understand the existing building is founded on footings reportedly penetrating 6 inches into the underlying bedrock. Interior building columns are founded on shallow piers. The building has a full basement with slab-on-grade construction. Areas surrounding the building are irrigated grass with some concrete sidewalks and asphalt pavement. The site slopes gradually away from the building on all sides. An approximately 7 feet deep areaway runs along the south side, outside of the existing building, and has an underground service tunnel beneath it. The existing building is a concrete block structure with brick veneer. We observed no cracks in the walls of the existing building and basement floor slabs and slab bearing partition walls showed no signs of distress. Representatives from the Archdiocese of Colorado reported no history of water problems in the basement area. There is no knowledge of a soil report for the existing building.
Proposed Construction

The existing entryway and stairway at the north side of the existing building will be demolished and a new four-story stair tower constructed in its place. Information provided by the Structural Engineer indicates the stair tower addition will be supported by the existing footing foundations and four new helix piers underpinning the existing grade beams. We were informed that the existing entryway and stairway are founded on footing pads with 2’6” x 6” dimensions. The Structural Engineer estimated there is currently about 15,000 psf footing pressure on each footing pad and the footing pressure would be increased to about 17,000 psf after construction of the new stair tower. The Structural Engineer plans to use helix piers to carry the additional loads and control the new loads on the existing footings lower than the existing loads.

A masonry portal addition will be built on the south side of the existing building. The portal will be separate from the existing building and connected only with horizontal cross members. The Structural Engineer plans to integrate the portal with the existing areaway below-grade wall to increase lateral resistance of the wall. It is planned that the portal be founded on drilled piers penetrating the bedrock.

We anticipate that the new additions will be constructed using concrete masonry units with brick veneer. Some site grading and extensive repaving will be performed surrounding the additions. A representative from the Archdiocese of Colorado has informed us that they do not require recommendations regarding the repaving work.

Subsurface Conditions

Subsurface conditions were investigated by drilling one exploratory boring in the area of each proposed addition to depths of 25 and 30 feet. Locations of the exploratory borings are shown on Fig. 1. Graphic logs of the soils and bedrock found in our borings and results of field penetration resistance tests are shown in Figs. 2 and 3. Soil and bedrock samples were returned to our laboratory where they were visually classified and tested. Tests included natural moisture content and dry density, swell consolidation and unconfined compression tests. Results of the laboratory testing are presented on Figs. 4 through 5 and summarized on Table I.

Soils encountered in our borings included 5 feet of fill near the masonry portal addition and no fill in the area of the stair tower addition. Native soils in both borings consisted of stiff, sandy clays overlying sandstone, claystone and interbedded sandstone and claystone bedrock. Clay samples tested showed low swell to slight compression. Claystone samples tested in laboratory showed low and moderate swell. Ground water was encountered at depths of 14 feet and 17 feet during drilling. Bedrock depth is likely to vary between borings.
Additions Foundations

We understand the proposed stair tower addition is planned to be supported on the existing footings and new helix piers and the portal addition to be supported on new drill piers bottomed in bedrock. Our investigations indicate the clays and claystone bedrock under the site are low to moderately expansive. Drilled piers and helix piers can be used to support the additions. Piers should penetrate below the zone of probable moisture variation and be designed and constructed to resist the swelling pressure. Footings founded on the expansive soils and bedrock need to maintain high deadload pressure to resist the swelling pressures. Normally, footings have higher risk of heave than the piers. Considering the existing building footing foundations have performed reasonably well, we believe the new addition can also be supported on these footings provided similar footing deadload and total load pressures are maintained after construction. Differential movements between the additions and existing building will occur due to either settlements of new foundations or rebound and reloading of the existing footings under the addition. A slip joint should be provided where possible at the connection to reduce the potential damages associated with the differential movements. Foundation design and construction criteria are as follows:

Helix Piers

1. The end bearing pressure is dependent on the properties of the helix size selected and the torque applied to install the piers. The bedrock under the site can provide an ultimate end bearing pressure of 90,000 psf. This value does not include a factor of safety. Helix piers should penetrate a minimum of 4 feet into relatively unweathered bedrock. Piers should be advanced to a torque required to accommodate the end bearing pressure specified by the structural engineer. Manufacturers recommendations regarding the torque and bearing relationship should be followed. A minimum factor of safety of 2.0 is required. The pier should be placed as close to vertical as possible.

2. The connection between the pier and grade beam should be designed to resist lateral earth pressure (if any) against the grade beams. The connection should be designed by a structural engineer. The helix piers should be attached to the existing grade beam using brackets that allow the pier to be pre-loaded during construction. The helix piers should be loaded so that the deadload on the existing footing foundation remains approximately the same before and after construction.

3. Twisting of the shaft can occur during the installation process. We recommend the structural engineer evaluate the effect twisting of the shaft has on the capacity of the helix pier.
4. Installation of helix piers should be observed by a representative of our firm to confirm the depth and installation torque of the helix piers are adequate.

Drilled Piers Bottomed In Bedrock

1. Piers should be designed for a maximum allowable end bearing pressure of 30,000 psf and an allowable skin friction value of 2,500 psf for the portion of the pier in bedrock. Skin friction should be neglected for the portion of the pier within 3 feet of the bottom of the foundation walls and grade beams.

2. A skin friction value of 2,000 psf can be used for uplift calculations. Additional bedrock penetration may be necessary to resist uplift.

3. Piers should be designed to maintain deadload pressure as high as possible.

4. Piers should penetrate at least 6 feet into relatively unweathered bedrock and have a minimum length of 18 feet. Depending on loading, the minimum penetration may need to be increased.

5. Piers should be reinforced their full length with Grade 60 reinforcing bars having a cross-sectional area equal to or greater than 0.005 times the end area of the pier. For 12 inch piers, minimum reinforcing with at least 2, No. 6 bars is recommended. Reinforcement should extend into grade beams and foundation walls.

6. Piers should be carefully cleaned prior to placement of concrete. Ground water was encountered at the time of this investigation. We believe casing may be necessary for proper cleaning and dewatering of the pier holes.

7. Concrete used in cased piers should have sufficient slump so it will fill the pier holes and will not hang on the sides of the casing during the extraction. We recommend a slump in the range of 5 to 7 inches if casing is used. To facilitate temporary casing installation and dewatering, piers should be designed with the diameter of 12 inches.

8. Piers should have center to center spacing of at least 3 pier diameters or they should be designed as a group. When checking this spacing, the largest diameter pier in the group should be used. If it is necessary to have the piers close together, we can provide criteria for group design on individual cases.
9. Installation of drilled piers should be observed by a representative of our firm to identify the proper bearing strata.

Lateral Loads

We anticipate that no new basement construction will be part of this renovation. It may be necessary to re-analyze the below-grade, retaining wall at the areaway on the south side of the existing building for lateral earth pressures. We anticipate that the retaining wall will be restrained so that deflections are minimal. An at rest equivalent fluid pressure of 50 pcf may be used in design calculations providing wall backfill consists of on-site soils. Loads due to surcharges and hydrostatic pressure should be added to the above distribution. Hydrostatic pressure may be alleviated by providing weep holes at the base of the retaining wall. If the entire retaining wall is reconstructed or a portion of it, we recommend a gravel drain behind the wall to reduce the hydrostatic pressure. The drain should consist of at least 12 inches of reasonably well graded clean sand and gravel backfill to within 2 feet of the ground surface. The top 2 feet should be compacted on-site soils. A manufactured drain such a Mira Drain may be substituted for the drain gravel. Manufactured drains should be installed following the manufacturers recommendations. Weep holes should be at least 4 inches in diameter and spaced 10 feet on center with no less than 3 weep holes provided for each wall. The back of the weep holes should be protected from clogging. Wall backfill should be placed in 8-inch maximum loose lifts, moisture conditioned to within 2 percent of optimum and compacted to at least 95 percent of standard Proctor maximum dry density.

We anticipate foundation piers will be designed to resist lateral loads applied to the structure through wind, seismic, and lateral earth pressures. Several methods are available to analyze laterally loaded piers. For helix piers and drilled piers with a length to diameter ratio of 7 or greater, we believe the method of analysis developed by Matlock and Reese is most appropriate. The method is an iterative procedure using applied lateral load movement, vertical load, and pier diameter to develop deflection and movement versus depth curves. Software developed by Reese can be used to calculate deflection for various pier diameters and loading conditions anticipated by the structural engineer. Movement versus depth curves developed from these analyses to aid the structural engineer and optimizing the location of reinforcement. Other procedures require input of horizontal modules subgrade reaction $K_h$. For purposes of design, we recommend the following horizontal modulus of subgrade reaction:
(For Bedrock)
\[ K_b = \frac{400}{D} \text{ (tons/ft}^3\text{)}, \]

(For Soils)
\[ K_b = \frac{50}{D} \text{ (tons/ft}^3\text{ where } D = \text{ shaft diameter in feet)} \]

These designs values do not include a factor of safety.

Exterior Slabs

We do not anticipate any new slab-on-grade floor construction. Any new exterior flatwork and sidewalks should be separated from the structures. Movement of slabs-on-grade should not be transmitted to the foundations of the structures. Frequent control joints should be provided according to ACI or PCA criteria to relieve problems associated with shrinkage or cracking.

Concrete

A soluble sulfate concentrate of 0.003 percent was measured in a sample of the on-site soils. Our experience with other sites in this area indicated a low risk of sulfate attack as does this measurement. We believe a Type I of Type II cement can be used for concrete in contact with the soils.

Surface Drainage

Performance of foundation is influenced by subgrade moisture conditions. The risk of wetting the subsoils can be reduced by carefully planned and maintained surface drainage. We recommend the following precautions be observed during construction and maintained at all times after the construction is completed.

1. Wetting or drying of the open foundation excavation should be avoided.

2. The ground surface surrounding the exterior of the building should be sloped to drain away from the buildings in all directions. We recommend a minimum slope of at least 6 inches in the first 10 feet if possible.

3. Roof downspouts and drains should discharge well beyond the limits of all wall backfill. Splash blocks and downspout extenders should be provided. Roof drainage should not be directed below the building.
4. Plastic membranes should not be used to cover the ground surface immediately surrounding buildings. These membranes tend to trap moisture and prevent normal evaporation from occurring. Geotextile fabrics can be used to control weed growth and allow for evaporation.

Limitations

Our borings were used to obtain a reasonably accurate picture of the subsurface conditions. The borings are representative of conditions only at the exact boring locations. Our analysis and recommendations apply to the proposed construction and soil conditions outlined in this letter. Should construction details change or differing soil conditions be encountered, we should be contacted to evaluate our recommendations. We believe this report was prepared using methods and procedures consistent with other professional practicing geotechnical engineering in this area at this time. No other warranty, express or implied, is made.

If we can be of further service in discussing the contents of this report or in our analysis of the influence of the subsoil conditions on the design of the structures, please call.

Sincerely,

Howard A. Perko
Geotechnical Project Engineer
Example Geotechnical Report
96-274

Fig. 1

Boring Locations

Tennessee Avenue

Main Street

Texas Avenue

Rose Street
### LOG OF EXPLORATORY BORING TH-1

**CLIENT**
Archdiocese of Colorado

**ARCHITECT/ENGINEER**
H. Perko

**SITE**
Saint Joseph Seminary

**PROJECT**
Proposed Additions

#### GRAPHIC LOG

<table>
<thead>
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<th>DEPTH (FT)</th>
<th>USCS SYMBOL</th>
<th>SAMPLES</th>
<th>TESTS</th>
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**DESCRIPTION**

- **FILL, CLAY AND SAND, SILTY**
  Medium stiff to stiff, moist to wet, brown/tan/red, occasional debris

- **CLAY, SANDY**
  Stiff, slightly moist to moist, brown

- **SANDSTONE AND CLAYSTONE**
  Dark Brown/Rust, slightly moist, hard

**WATER LEVEL OBSERVATIONS**

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<th>WL</th>
<th>14’ W.D.</th>
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**Example Geotechnical Report**

**BORING STARTED**
10/5/1996

**BORING COMPLETED**
10/5/1996

**RIG**
CME-55

**FOREMAN**
DS

**APPROVED**
HP

**JOB NO.**
96-274
### Graphical Log

**Log of Exploratory Boring TH-2**

**Archdiocese of Colorado**

**Prospective Additions Saint Joseph Seminary**

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<th>Depth (ft)</th>
<th>USCS Symbol</th>
<th>SPT</th>
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**Water Level Observations**

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**Stratification Lines**

The stratification lines represent the approximate boundary lines between soil and rock types. In situ, the transition may be gradual.

**Boring Started**

10/5/1996

**Boring Completed**

10/5/1996

**Example Geotechnical Report**
Sample of CLAY, SANDY (CL)
From TH-1 AT 9 FEET
NATURAL DRY UNIT WEIGHT = 110 PCF
NATURAL MOISTURE CONTENT = 11.6 %

Sample of CLAYSTONE
From TH-1 AT 14 FEET
NATURAL DRY UNIT WEIGHT = 112 PCF
NATURAL MOISTURE CONTENT = 14.5 %

Swell Consolidation Test Results FIG. 4
Sample of CLAYSTONE
From TH- 2 AT 9 FEET
NATURAL DRY UNIT WEIGHT = 110 PCF
NATURAL MOISTURE CONTENT = 12.2 %

EXPANSION UNDER CONSTANT PRESSURE DUE TO WETTING

Sample of CLAYSTONE
From TH- 2 AT 19 FEET
NATURAL DRY UNIT WEIGHT = 107 PCF
NATURAL MOISTURE CONTENT = 12.6 %

EXPANSION UNDER CONSTANT PRESSURE DUE TO WETTING

Swell Consolidation Test Results FIG. 5