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# A case history of structures constructed on expansive soils

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**ABSTRACT**

A case history is presented that documents distress, and lessons learned from the failure of foundations constructed on a site with highly expansive soils. Eighteen (18) multi-story buildings were constructed on post-tensioned concrete slabs-on-grade founded on engineered fill. The pre-construction soils report recommended that the in-situ expansive foundation soils be over-excavated, moisture conditioned, and recompacted. A few years after the structures were constructed the buildings began to show signs of distress in the form of drywall cracking, foundation heave, and racked doors. Differential movement as large as approximately 100 mm (4 in) was observed in the slabs. Calculated values of future heave ranged from 160 to 225 mm (6.4 to 8.9 in) with one value of 530 mm (20.8 in). The site is located in a semi-arid environment and was irrigated after development. A well irrigated golf course was located on an adjacent property. Several of the buildings were located parallel to the golf course property boundary. Those buildings were observed to be tilting away from the golf course, due to the differential heave caused by the lateral migration of groundwater from the extensive irrigation on the golf course.

This paper presents the results of the forensic investigation and identifies the causes of the distress. The original pre-construction site investigation was found to be inadequate. Of the soil samples that were tested for the design, only a small number of samples were actually taken from depths below the depth of over-excavation. Thus, the foundations were designed based on very limited soil sampling and analyses. The post-tensioned slab foundations were inadequate, and incorrect assumptions were used in their design.

Lessons learned from this project are that the geotechnical site investigation must extend down to a depth to which the soil properties will influence foundation performance. If expansive soils are present, that depth may extend for several meters. The geotechnical investigation must also consider environmental conditions, not only at the site, but at properties located adjacent to and at some distance from the site. Grading and drainage of the site is of paramount importance. When a failure of this nature occurs, the cost of the remediation may exceed the cost of the original construction.

**Keywords:** Expansive Soil, Site Investigation, Heave Prediction, Post-Tensioned Slabs, Forensic Investigation, Foundation Design, Building Distress.

## 1 INTRODUCTION

Expansive soils occur widely around the world, and are most problematic in arid or semi-arid environments. Figure 1 shows the global distribution of expansive soil sites. In Asia, they are found in areas throughout India, Northern Thailand, China, and Japan. It is also seen from Figure 1 that they are frequently encountered throughout North America. The case history presented in this paper documents a number of foundation failures that occurred in Colorado, USA.

The site is located in an area along the Front Range of the Rocky Mountains in Colorado. A number of multi-story apartment buildings were constructed on slab-on-grade post-tensioned foundations. A few years after construction, distress was observed in the form of large cracks in the walls and ceilings, racked doors, and foundation heave.

A forensic investigation was undertaken that documented the nature and cause of the distress. It also identified serious errors in the geotechnical engineering site investigation and the engineering design. This paper describes the site, the errors in the site investigation, and in the design. It also discusses the lessons learned from this case.

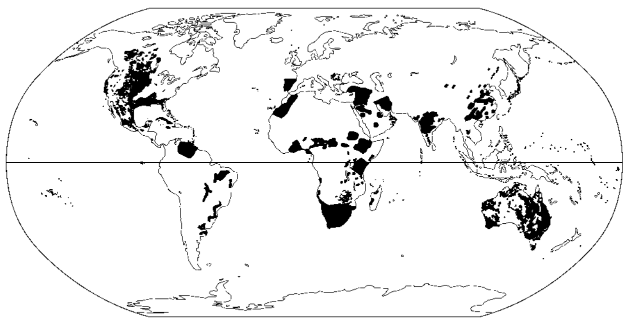

Figure 1. Global Distribution of Reported Expansive Soil Sites (Nelson et al., 2015)

## 2 DESCRIPTION OF SITE

The site is located in an area with a semi-arid climate near the foothills of the Rocky Mountains in Colorado, USA. During the pre-construction site investigation, the bedrock was encountered at depths of 1.1 to 3.6 m. (3.7 to 12 feet) below the ground surface. Borings drilled at the site showed that the bedrock at this location consists of claystone and silty claystone with interbedded siltstone and minor lignite seams, which provide a conduit for easy transmission of groundwater. Consolidation-swell tests (ASTM D4546) were performed on samples of the claystone in both the pre-construction and forensic investigations. The test results indicated that the claystone is highly expansive.

After the buildings were constructed, evapotranspiration from the soil surface no longer occurred, and the water content in the subsoil increased. Landscape irrigation and other factors introduced additional water into the subsoil beneath the slabs and foundations. Development of adjacent sites, particularly a well-landscaped golf course that bordered the site, introduced water into the claystone through cracks and fissures in the subsoils and the lignite seams.

## 3 NATURE OF DISTRESS

The project consisted of 18 multi-story residential structures that were constructed on post-tensioned slab-on-grade foundations. The structures were constructed over a period of several years beginning in about 2003. Shortly after construction, distress began to occur. Distress was caused by the differential heave of the expansive claystone. It took the form of mild to very severe cracking in the interior walls and differential movement of the slabs. An example of the cracking is shown in Figure 2.

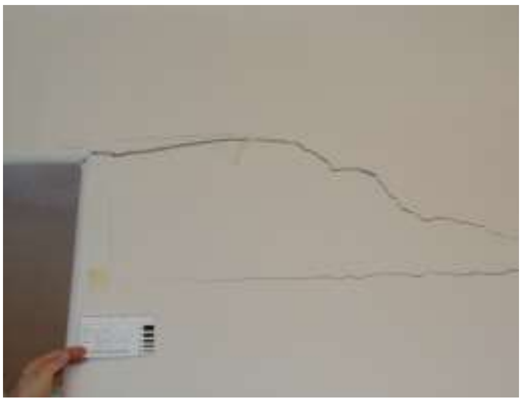

Figure 2. Example of Severe Cracking of an Interior Wall

A typical building footprint is shown in Figure 3. The breezeways between the units were convenient locations in which to conduct elevation surveys to monitor the shape of the slabs. The results of one such survey are shown in Figure 4. In that figure, it can be seen that the heave continued to occur over time with heave in amounts up to 190 mm (7.5 in) over a period of about 9 years. The heave shown in Figure 4 is that of the post-tensioned slabs and is representative of the heave encountered inside the units.



Figure 3. Typical Building Footprint With Survey Locations of Survey Points in Breezeways

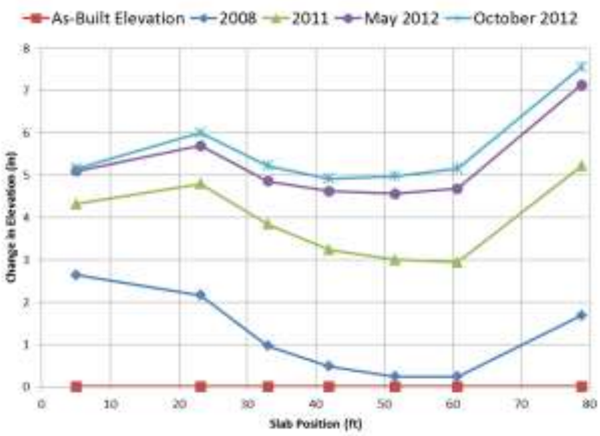



Figure 4. Breezeway Survey Results for Building 9

## 4 SITE INVESTIGATION AND FOUNDATION DESIGN

The pre-construction site investigation was performed in late 2002 prior to construction. A total of 21 borings were drilled across the site to a depth of 6.1 m (20 ft). The materials encountered in the borings were described as lean clay overlying sandstone, siltstone, and claystone. Groundwater was not encountered at the time of drilling. Of the total number of soil and bedrock samples that were taken, only 21 were tested in consolidation-swell tests. The samples exhibited values of percent swell ranging up to 9.3 percent at an inundation pressure of 24 kPa (500 psf). Swell pressures up to 320 kPa (6,700 psf) were measured.

In the first pre-construction soils report, consolidation-swell tests were reported for samples taken from depths ranging from 0.6 to 2.7 m (2 to 9 ft) below the ground surface. A second geotechnical investigation was conducted in 2007 after some of the buildings had been constructed. Consolidation-swell tests were performed on samples taken from depths up to a maximum depth of 7.3 m (24 ft). Some samples taken from depths greater than 2.7 m (9 ft) exhibited very high expansion potential with values of percent swell up to 10.9% and values of swelling pressure up to 527 kPa (11,000 psf).

It was recommended that the buildings be supported on post-tensioned slab-on-grade foundations and that the soil beneath the planned foundations be over-excavated and replaced with newly placed engineered fill. The depth of over-excavation for the fill was specified to be 0, 0.9, 1.7, or 2.1 m (0, 3, 5.5 or 7 ft) depending on the location of the building within the site.

After major distress had been observed, the authors' company (Engineering Analytics, Inc. – EA) conducted a forensic investigation. EA drilled nine deep borings. Five of these borings were drilled in 2008 and four were drilled in 2011. One of these was drilled to a depth of 24 m (79 ft) for the purpose of installing a deep benchmark below the depth of potential heave. The other eight were drilled to depths of 12 to 18 m (39 to 58 ft). Consolidation-swell tests exhibited values of percent swell ranging up to 11.2 percent at an inundation stress of 48 kPa (1,000 psf). Swell pressures for these samples ranged up to 945 kPa (19,750 psf).

## 5 COMMENTS ON FOUNDATION DESIGN

### 5.1 Heave Prediction

The pre-construction soils report did not include any computations of predicted heave for any of the buildings. Thus, the foundations were designed with no computations of expected heave values. In the forensic investigation, EA used the results of the original soils report to compute predicted heave. The computed values ranged from 70 to 230 mm (2.8 to 9.0 in).

Those computations used limited data from the shallow (6 m; 20 ft) borings. In the forensic investigation, data from the deeper borings performed by EA were used to compute the remaining heave. Values computed using the data from the 2008 borings ranged from 66 to 114 mm (2.6 to 4.5 in). The data from the 2011 borings yielded values ranging from 160 to 225 mm (6.4 to 8.9 in) with one very high value of 530 mm (20.8 in). Those values represent future heave that would occur in addition to that which had already occurred. As shown in Figure 4, heave in amounts up to about 190 mm (7.5 in) had already occurred by 2011.

The values of predicted heave represent the values that should have been used in the design. It will be discussed below that the values used for the design of the foundation slabs were actually much lower than those computed in the forensic investigation.

### 5.2 Foundation Design

The foundations were designed using incorrect design procedures. The design of the post-tensioned slabs was based on the 2$^{nd}$ edition of the Post-Tensioning Institute design manual (PTI, 1996). The original soils report listed the input design values of differential soil heave, $y_m$, as 100 and 38 mm (4 and 1.5 in) for the center lift and edge lift conditions, respectively. No calculations for these input parameters were provided in the soils report. In the forensic investigation that was undertaken after the distress, the predicted free-field heave at the time of construction was calculated to be as high as 230 mm (9.0 in). These calculations were conducted using the procedures that have been presented in Nelson et al. (2015). Therefore, the design value of $y_m$ should have been 230 mm (9.0 in). However, the PTI design manual states that it is valid only for values up to 100 mm (4.0 in). The design value used for edge moisture variation distance, $e_m$, was 1.7 m (5.5 ft) for the center lift conditions, and 0.76 m (2.5 ft) for the edge lift conditions. No calculations for the recommendations were provided in the report. Our experience indicates that the edge moisture variation distance, $e_m$, can extend to the center of the slab (Nelson, Chao, and Overton; 2006). The values used for the design were much lower than those computed above in Section 5.1.

### 5.3 Over-excavation

Over-excavation removed almost all of the soil on which testing had been performed during the original soil investigation. The process of over-excavation is described in detail in Nelson et al. (2015). It consists of removing the existing foundation soil to a specified depth and replacing it with a low or non-expansive soil. In the pre-construction soils report, consolidation-swell tests were performed on samples taken from depths of 2.7 m (9 ft) or less. The maximum depth of over-excavation was specified to be 2.1 m (7 ft). When the sample depths were mapped below the foundations, it was observed that approximately 90% of the samples that were tested were from soil that was removed during over-excavation. Thus, even if heave predictions had been performed for use in design, almost all of the samples that would have been used for design data were actually removed. Most of the foundations were designed without any actual data.

### 5.4 Type of Foundation

The slab-on-ground foundation was the wrong type of foundation to use at this site. In addition, the 2$^{nd}$ edition PTI design manual (PTI, 1996) was not appropriate for the following reasons.

5.4.1 The PTI method is highly empirical in nature, and is based solely on climatic conditions, and for heave values much lower than those at this site.

5.4.2 The 2nd edition of the PTI manual limits the depth of wetting to 2.1 m (7 ft) It was known that the depth of wetting at the site would be much deeper than that.

5.4.3 The 2nd edition of the PTI manual states that the procedures estimate the amount of differential soil movement due to climate alone. They do not consider factors such as uneven irrigation around the foundation, leakage from utility lines or detention ponds, or migration of water from off-site development.

A second reason not to use a slab-on-ground foundation at this site is the flexibility of the slab and the high expansion potential of the foundation soils. The slab-on-ground foundations that were used had a slab of uniform thickness, rather than one with deep stiffening beams (i.e., a ribbed foundation). A uniform thickness slab is not capable of resisting the differential heave of the highly expansive soil. While the slab is designed to be flexible, the structures resting on post-tensioned slabs are not. As a result, large differential movements were transmitted to the upper structure resulting in the distress described above, and negatively affecting the structural integrity of the buildings.

### 5.5 Groundwater Analysis

Although groundwater was not observed in the original site investigation, the analysis of the site did not account for future changes in the groundwater regime. Piezometers were installed in the forensic investigation. They showed a general migration of groundwater from the golf course across the site. The measured heave of the foundations bordering on the golf course was greatest on the side next to the golf course. Thus, the buildings were tilting in the direction away from the golf course. Infiltration from on-site irrigation due to poor drainage and over-watering of the landscaping also contributed to wetting of the sub-soils.

### 5.6 Remedial Measures

A remediation plan using micro-piles was designed. That system required that the slab-on-ground be stiffened using stiffening beams. The cost of the remediation plan was greater than the value of the structure, and therefore it was not implemented. It is common that the repairs for foundation failures on expansive soils exceed the value of the structure.

## 6. CONCLUSIONS

The value of case histories lies in the lessons learned from the failures. In this case, the following lessons were learned.

6.1 Attention to the geology of a site is of paramount importance. It was known that the site had highly expansive soils and extraordinary care should have been exercised in the investigation and design.

6.2 Consideration was not given to expected changes in the groundwater regime. Although groundwater was not observed in the pre-construction soils report, attention should have been given to expected future changes, especially with the existence of a highly irrigated golf course adjacent to the site.

6.3 The depth of borings and testing of samples was not adequate. The drilling of borings and testing should extend to the depth to which the soils will have an influence on the behavior of the foundation. When expansive soils are present, the depth of potential heave can be very deep. The investigation must extend to that depth. At this site, the recommended depth of over-excavation removed almost all of the soil on which tests had been performed to gather design data.

6.4 The design methodology was faulty. There were no actual predictions of heave. Also, the PTI design manual was not applicable for this site.

6.5 The type of foundation was not appropriate for this site. The uniform thickness slab without stiffening beams was too flexible to resist the large deformations and swelling pressures produced by the highly expansive soils.

6.6 Heave will continue for long periods of time and takes a long time to fully manifest itself. This was evident from the elevation survey results.

6.7 Because the cost of remediation is usually very high, it is of utmost importance that extraordinary attention be given to the appropriate design of the foundation system. It is important for the engineer to convince the owner of the need for the extra cost.

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