LESSONS LEARNED FROM DISTRESS OF FOUNDATIONS ON EXPANSIVE CLAYS IN THE ACTIVE ZONE

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ABSTRACT

The high initial cost for a deep foundation system with a structurally suspended floor slab for light commercial and retail buildings on expansive clays would make many, if not most, projects economically prohibitive. Reality is that such structures are commonly supported on a hybrid foundation system where exterior walls and interior columns are supported on a relatively deep foundation system, and the floor slab bears on a pad of fill on expansive clays in the active zone. Most foundation systems of this design experience tolerable movements, but some experience significant distress resulting in damaging the building. This paper presents case studies on common design and construction defects (some would appear minor) that can contribute significantly to distress, and offers suggestions to address these issues.

INTRODUCTION

Four case studies are presented of building distress due to the foundations situated in the active zone. Two are located in Houston, Texas (Beaumont Formation) and two in Irving, Texas, a suburb of Dallas (Eagle Ford Formation). Both metropolitan areas are underlain by expansive soils to varying degrees. Descriptions of the geotechnical properties of each formation are provided in the Case Study sections.

Houston, Texas receives an average annual rainfall of 48 inches (122cm), with an approximate Thornthwaite Moisture Index (1948) of +18. The average annual rainfall for the Dallas, Texas area is 33 inches (84cm), with an average Thornthwaite Moisture Index of 0.

THE ACTIVE ZONE

Geotechnical professionals offer various definitions of the active zone within the literature. Chen (1975) discusses that the “depth of seasonal moisture content fluctuation”, now commonly referred to as the “depth of seasonal moisture change”, occurs in an uncovered field due to wet/dry moisture cycles. O’Neill (1980) reported that the depth of the seasonal active zone was 5 to 10 feet (1.5 to 3m) in Houston, and 7 to 15 feet (2.1 to 4.6m) in Dallas.

However, Chen discusses that this depth of moisture will be deeper when the area is covered with structures, paving and etc., due to environmental changes. He refers to this as the “depth of desiccation”, and states “ This depth represents the total thickness of material that can expand because of water deficiency.”

Reed (1985) expanded upon this definition dividing the active zone into two sub-zones; a seasonally active portion, and a zone of deep-seated” movement. The latter was defined as the portion of the active zone where the potential swell pressures exceed the overburden stress.

Fredlund and Rahardjo (1993) define the “active zone” as the “zone of soil undergoing volume change on an annual basis”. Inherent in this definition is the concept of a seasonally active soil. Many practicing geotechnical engineers use this definition of the active zone when calculating the potential for heave, but it fails to consider the potential for deep-seated moisture changes that can occur when a barren field is covered with a structure or paving. Also, leaking utility lines or a rise in the water table can alter the depth of moisture change.

More recently, Nelson, Overton and Durkee (2001) proposed that the active zone be defined as “that zone of soil that is contributing to heave due to soil expansion at any particular time.” The depth of the active zone is further defined as “the depth to which the overburden vertical stress equals or exceeds the swelling pressure of the soil”. It is this definition that will be used in the following case studies.

CASE STUDIES IN BEAUMONT FORMATION

Geotechnical Data – The Beaumont Formation is a Pleistocene age alluvial deposit consisting predominately of highly plastic clay, with interbedded silt and sand lenses and layers. Site topography within the Beaumont is generally flat with gentle slopes of one to two percent. Ground water is typically present at depths of 10 to 20 feet (1.5 to 3m).

The presence of ground water somewhat limits drying of the Beaumont Formation. Typical seasonal drying varies from 5 to 10 feet (1.5 to 3m); however, greater depths of drying can occur in the vicinity of mature trees.

Case Study 1 – This site is located in Sugar Land, Texas. The building is a 2-story tilt-wall structure surrounded by concrete driveways and parking lots. Landscaping included small planter beds on the north, east and west sides of the office area.

The building was constructed in the early 2000's, and foundation movements were noted within the first year. Three years later, heave of the clay subsoils caused 1 to 2 inches (2.5 to 5cm) of differential movement of the floor slab resulting in cracks in dry walls, sticking doors, and dislocation of the acoustical ceiling tile system.

The pre-construction geotechnical investigation report was prepared in May 2000. Only two soil borings had been drilled to depths of 20 feet (6.1m) under the building where there should have been several more due to the size of the building. The upper ±6 feet (1.8m) of soil was stiff to very stiff sandy clay/clay. Only two Atterberg Limit tests had been performed on samples from this stratum which was a very small sampling for a large mass of soil. The average Liquid Limit (LL) was 50 and the Plasticity Index (PI) was 33. The clay was underlain by a ±4-foot (1.2m) layer of very stiff to hard sandy clay with an average LL of 26, and PI of 11. A stratum of medium dense silty sand was found at a depth of 10 feet (3m) in both borings. The ground water table was found at a depth of 16 feet.

The geotechnical engineer-of-record computed that the potential vertical rise (PVR) was 2¼ to 3 inches (5.7 to 7.6cm) using the McDowell Method (1956), now known as the Texas Department of Transportation Method Tex 124-E. They reported that the "depth of seasonal moisture change" was 6 feet. A foundation system of underreamed piers bearing at a depth of 8 feet below existing grade was recommended. The engineer recommended that the floor slab be situated on 3 feet (.9m) of select fill with a PI of 10 to 20 to minimize the potential for heaving of the floor slab.

The post-construction forensic study found that the floor slab was situated on 2½ to 3 feet of select sandy clay fill as recommended by the geotechnical engineer. The average LL was 33, and the average PI was 19. However, the LL of the upper clay layer was found to be 58 to 73, and the PI was found to be 42 to 53. The LL of the underlying sandy clay strata was 36, and the PI was 24. The small number of borings, and minimal laboratory testing performed had failed to properly identify the expansive nature of the clays.

Two post-construction borings were drilled in areas where little heave occurred. The liquidity index of the upper clays ranged from -.02 to -.04 indicating a moderate to high swell potential. The liquidity index of the underlying sandy clays ranged from -.09 to -.12. Site construction had apparently occurred during a dry period of the year.

Based on results of the post-construction swell tests, the swell potential below the floor slab was computed to be ±4 inches at the in situ moisture content of the clays. The geotechnical engineer-of-record did not address measures to reduce the swell potential to a reasonable magnitude. A PVR of 1 inch (2.5cm) is a commonly accepted value in the greater Houston area. The depth of the active zone is commonly accepted as 5 to 10 feet in the Houston area (O'Neill, 1980). Sugar Land is west of Houston in a somewhat more arid zone, and the depth of the active zone was judged to be 10 feet (3m) due to a sand layer at this depth. Placement of the floor slab on expansive clays with a 4-inch (10cm) swell potential was an error, and significantly contributed to distress of the building.

Expansive clays need a source of water for swelling to occur. The landscape architect placed small planter beds next to the office area for aesthetic purposes. He did not seal the beds in a concrete box, or other impermeable materials. To aggravate matters, he placed an irrigation system in the planter beds.

The areas where the largest amount of heave occurred were over the underground sewer line and close to the planter beds. However, camera surveys and leak detection tests did not find leaking sewer lines. The MEP engineer who designed the underground utilities specified that the underground sanitary sewer lines be bedded and encased in sand. The 'clean-out' for the lines was located at the west wall in the landscape area. As shown on Photo 1, a test pit in this area found water-bearing sand backfill at a depth of 1½ feet (0.5m). Since the clean-out pipe slopes down to the main line, water was flowing by gravity down the sand-filled trench to the main trunk line, where it went south and exited the building. Another test pit in the area where the main trunk line exited the building found wet sand backfill around the pipe.

Photo 1: Water Bearing Trench Backfill

The distress occurred due to a combination of errors. The geotechnical engineer-of-record failed to adequately address the high swell potential that existed at this site, and he under-estimated the depth of the active zone. Furthermore, the number of borings and the quantity of laboratory testing was inadequate to determine the expansive nature of the clay subsoils. The landscape architect did not understand the impact that water has on expansive clays, and the MEP engineer designed a pipeline for direct movement of water under the building.

The lesson learned is to only hire qualified professionals for the geotechnical study, and to prepare the plans and specifications. A "dollar saved" by the owner by bidding professional service can result in hundreds of thousands of dollars in damages, and much larger costs if litigation occurs.

Case Study 2 - This site is located in southwest Houston near the Westwood Mall. The building is a 6–story steel frame structure with exterior concrete panels and window wall. Most of the area around the building is maintained grass, with large oak trees at several locations.

The building was constructed in the early 1980's. The geotechnical engineer-of-record is unknown. The foundation system was spread footings bearing at a depth of 8 feet (2.4m) below natural grade. The floor slab had been raised with 4 feet (1.2m) of clay fill, and thus the footings bear at a depth of 12 feet (3.7m) below top of slab. Post-construction borings found that the LL of the fill ranged from 49 to 52, and that the PI ranged from 30 to 37. The clay fill would be described as having a high shrink/swell potential, and would not need the currently accepted criteria of a PI of 15 ± 5.

The LL of the underlying stiff to very stiff clays ranged from 69 to 92, and the PI ranged from 43 to 61. The natural clays would be classified as having a high to very high shrink/swell potential.

A water level survey found that 1 to 2½ inches (2.5 to 6.4cm) of settlement of the ground-supported slab occurred at the southwest corner of the building. Differential movement caused cracks in dry walls, sticking doors, gaps along the baseboards, and separations at the ceiling tiles. A water level survey was also performed on the 3rd floor, and 2½ inches (6.4cm) of settlement of the footing at the southwest corner was found, and the footing east of this location settled ±1½ inches (3.8cm).

As shown on Photo 2, there were 5 oak trees ranging from 20 to 30 inches (50 to 76 cm) in diameter close to the building where the settlement occurred. The building was elevated on a berm about 2 feet (0.6m) above the parking lot and driveways, and drainage was very good. The grass on the south side was sparse due to the lack of water, and what grass that remained was severely distressed.

Photo 2: Oak Trees at Perimeter of Building

Two borings were drilled to depths of 20 feet (6.1m) inside the building in the area where settlement occurred. A ¾-inch (1.9cm) void was found under the floor slab at both locations. Also, two borings were drilled in locations where the floor slab was relatively level for comparison purposes. Laboratory testing included Atterberg limit, moisture content, swell tests in an oedometer cell, and soil suction tests.

Figure 1 presents a plot of moisture content versus depth, and it indicates that the depth of the active zone is ±18 feet (5.5m). The soil suction tests indicate constant suction occurs at a depth of ±20 feet (6.1m). For comparison, the presently accepted depth of "seasonal moisture change" is 5 to 10 feet (1.5 to 3m) in Houston (O'Neill, 1980).

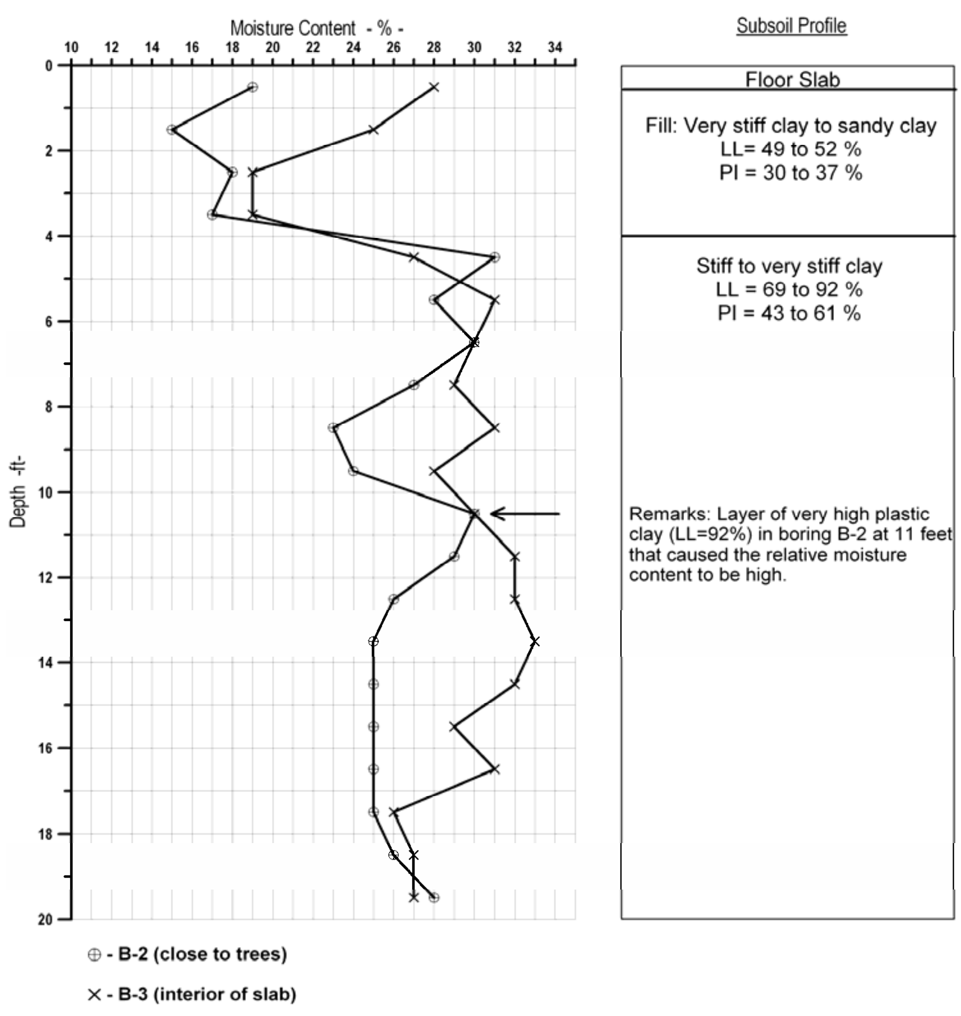

Figure 1: Profile of Moisture Content vs. Depth

The trees were removed by the owner to stop any additional settlement. An 8-foot (2.4m) deep vertical moisture barrier was placed next to the building in the area where the trees were removed in an attempt to slow down and equalize the moisture content of the clays as they suck moisture to reach an equilibrium condition. The building is currently being monitored for potential rebound as the moisture content stabilizes over time.

The landscape architect clearly did not understand the fact that moisture loss due to trees would cause settlement of the foundation. The depth of the active zone due to moisture demand of trees can be much greater than the presently accepted "depth of seasonal moisture change" of 5 to 10 feet (1.5 to 3m) in Houston.

The lesson learned is not to plant trees close to buildings on expansive clays unless a deep foundation system is used, and a structural slab is designed and constructed so that it does not deflect when the soil shrinks away. Also, trees can lower the moisture content to deep depths greatly magnifying the potential for heave if a building is constructed over the area where a tree or trees have been removed.

CASE STUDIES IN EAGLE FORD FORMATION

Geotechnical Data – The Eagle Ford Formation is a Cretaceous Age, deep ocean marine deposit of highly plastic silty clay with occasional seams of volcanic ash deposits. Depositional pressures have transformed the clay into soft to very soft, dark gray shale with seams of bentonite associated with volcanic deposits. Weathering of the shale produces a yellowish-brown to light gray, highly plastic clay. Weathering extends to depths of 10 to 50 feet, dependent upon the specific physiographic setting. The weathered profile is highly jointed and fractured.

Typical pre-development slopes within the Eagle Ford are 5 to 12 percent. Due to the site slopes and rainfall pattern, ground water is generally not present.

Seasonal moisture variation is limited; however, because of extensive growth of mature mesquite trees, extremely dry soil conditions extend to depths in excess of 30 feet. Natural moisture is typically 5 to 8 percentage points below plastic limit.

Case Study 3 - The case study consists of observation of the magnitude of heave over a 28-year period. Lessons to be learned include appreciation of the depth of the potential active zone and limiting access of water to expansive soils.

The structure consists of a tilt-wall office/warehouse constructed within residual soils of the Eagle Ford Group. The severely weathered shale extended to depths in excess of 40 feet. At the time of the original investigation in 1977, the depth of activity (i.e., the active zone) was generally accepted to extend to a depth of 12 to 15 feet below grade.

Foundation support for the structure was provided by underreamed piers founded at a depth of 15 feet below finished floor. The floor is a ground-supported "floating" slab over three feet of clayey sand (PI of 10 to 15). The select fill was placed to reduce the potential for movement. The depth of "select" fill resulted in the interface of the select fill/native soil being approximately 2½ feet below exterior grade along the north and west sides of the building.

The ground surface along the north side of the structure was landscaped with grass and irrigated. The ground surface graded to a low swale located approximately eight feet from the north wall of the building. The surface along the west side of the building was paved for a lateral distance of approximately 100 feet.

The pre-construction geotechnical investigation was preformed in 1977. The most recent investigation was performed in 2005.

A relative differential elevation survey of the floor was performed in 2005. A copy of the elevation survey is provided in Figure 2.

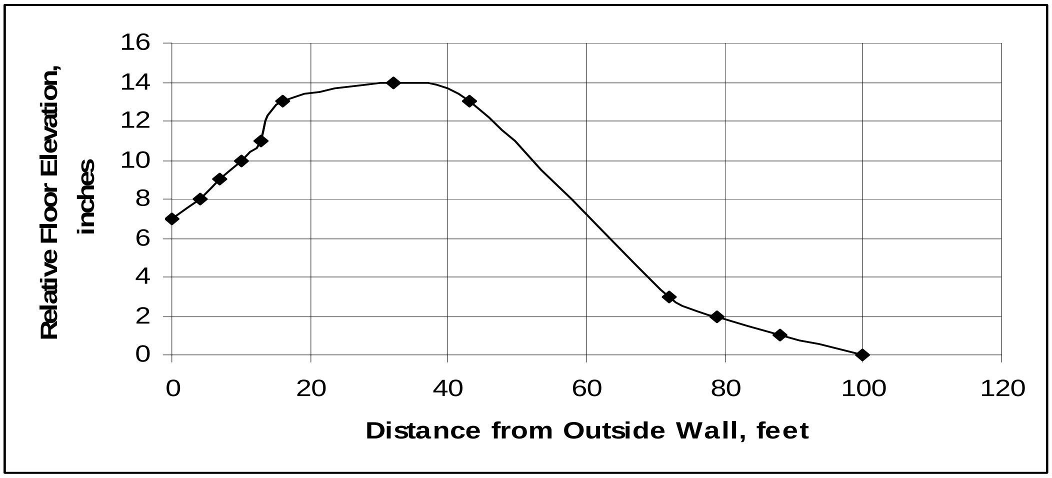

Figure 2. Relative differential elevation survey of site in Irving, Texas.

Based on the survey, the maximum differential elevation from the high to the low point of the floor slab was on the order of 16 inches (41cm), measured from the center of the warehouse slab to the high point at the west end of the building. The pattern of movement is consistent with the wetting front moving from the building exterior to the interior. The pattern of movement is not consistent with the presence of natural ground water.

The pattern indicated by the elevation survey is also consistent with some differential movement of the piers supporting the tilt-wall panels. Based on the observed pattern of movement, it is estimated that the underreamed piers have moved two to seven inches differentially since construction, dependent upon location. It appears the area of greatest pier movement is at the west end of the building. Movement is attributed to heave of the severely weathered shale below the founding depth of the piers.

As the severely weathered shale heaves, there would be a corresponding increase in the soil moisture. A comparison of the moisture contents of the severely weathered shale, prior to construction of the building in 1977 and in 2005 at the northwest corner of the building, is presented in Figure 3.

The comparison shown in Figure 3 indicates a significant increase in moisture within the severely weathered shale during the life of the structure.

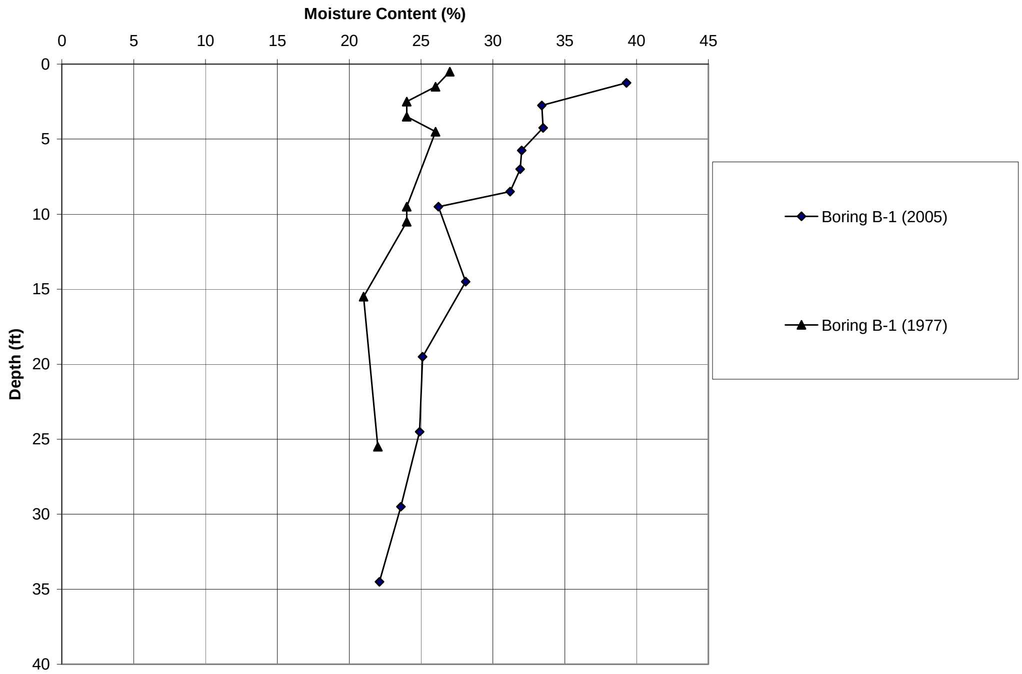

Figure 3. Moisture Content Profile – Northwest Corner of Building. (1 ft. = 30.48 cm)

A comparison of the two moisture content profiles in Figure 3 indicates a significant increase in moisture since project construction. Local experience in the Eagle Ford Group indicates an approximate one percent vertical swell for each one percent gain in moisture. Using this correlation and the two moisture profiles shown in Figure 3, discounting the upper 1½ feet (.5m), an estimate of heave of 17¾ inches (45cm) is obtained. This correlation would indicate that approximately 10½ inches (27cm) of heave would be anticipated within the upper 15 feet (4.6m), with a balance of approximately 7¼ inches (18cm) of heave occurring below a depth of 15 feet (4.6m). Considering the relative accuracy of the available information, the correlation between moisture gain and observed differential elevation of both the floor and tilt-wall panels is reasonable.

This study clearly indicates a lack of appreciation for heave-related movement below what was at the time considered the active zone. Placement of a clayey sand "select" fill below grade also created a "bird bath" condition where the interface of the select fill/native soils was approximately 2½ feet (.8m) below exterior grade. Water made available along the perimeter of the structure was able to pond within the select fill and saturate the weathered shale from the top down.

Case Study 4 – This case study is a senior nursing facility within the Eagle Ford Formation. The structure consists of a post-tensioned flat slab suspended on drilled, cast-in-place piles (piers) founded within unweathered shale. Design incorporated use of cardboard forms to construct a required 24-inch (61cm) void between the bottom of the slab and underlying soil. The plumbing was ground-supported, rising vertically through penetrations in the suspended flat slab.

Differential movement of the structure was noted shortly after the building was occupied. Inspection of the constructed void space below the slab noted broken and leaking sewer lines. It was also noted that limited degradation of the cardboard forms was occurring and that a void was developing between the top of the piers and overlying flat slab. Review of the structural plans found that the design structural connection between the slab and pier consisted of vertical extension of the pier steel into the slab.

The maximum differential movement through 2008 between any two locations was documented to be 17 inches (43cm). The recorded movement is attributed to inadequate structural reinforcement of the slab to the piers to prevent uplift. The uplift pressure from the soil on the cardboard forms exceeded the available uplift capacity at the pier/flat slab connection. Limited degradation of the cardboard boxes was also occurring.

Although water was being made available from broken utilities, it is likely that, over an extended period, irrigation would have supplied sufficient water resulting in movement of a similar magnitude.

CONCLUSIONS

One of the critical components to evaluation of the potential for movement of expansive soils is estimation of the depth of the “active” zone. Common practice of estimating the depth based on “past experience” or to a depth of constant moisture may severely underestimate the depth and hence the magnitude of potential movement. Environmental factors associated with development, such as changing site grades, removal or addition of trees, and addition of irrigation can significantly alter pre-construction moisture regimes.

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