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ABSTRACT

In July 2010, two vapor extraction wells were installed about 15 feet from a building at an angle of 50 degrees to the horizontal using rotosonic drilling technique (RDT). In June 2011, a crack approximately 0.5 inch wide on the wall of the building was reported. Several other small cracks were observed on the building following inspection by the authors. The owner of the building expressed concerns that the rotosonic drilling was the cause of the cracks and wanted assurance that subsequent drillings would not exacerbate the problem.

Geotechnical forensic investigation was performed to evaluate the potential cause(s) of cracking in the building and whether future drilling would impact the building and the foundation structure system. The investigations involved performing site reconnaissance surveys, site-specific field investigations, real-time vibration monitoring, crack monitoring, and geotechnical laboratory analyses.

This paper presents the results from the forensic investigations. Based on these results, potential causes for the development of cracks in the wall of the building and recommended repair measures are discussed.

INTRODUCTION

The site consists of a single-story concrete block masonry building on an approximate 0.5-acre parcel of land and is located in Tallahassee, Leon County, Florida. Dry cleaning operations at the site reportedly started in 1958 and continue to the present day. Over time, soil and groundwater at the site have been impacted with dry cleaning solvents, primarily tetrachloroethylene (PCE). As a result, the site is currently undergoing remediation under the Florida Department of Environmental Protection (FDEP) Dry Cleaning Solvent Cleanup Program. As part of the site assessment and remedial activities, soil borings and groundwater monitoring wells have been completed near and in the vicinity of the building since early 2006 by Ecology and Environmental Inc. (E&E, 2007a,b) and Geosyntec Consultants (2008, 2010). The locations of soil borings and groundwater monitoring wells completed as part of these investigations are shown on Fig. 1.

The July 2010 investigation performed by Geosyntec (2010) consisted of installing two 4-in. diameter vapor extraction wells within 8-in. diameter boreholes about 15 ft. from the southwest wall of the building. The wells were installed at an angle of 50 degrees to the horizontal using RDT and involved the advancement of an 8-in. diameter steel casing to the terminus of the borehole. Then a 4-in. diameter PVC pipe was installed and filter sand was added to the annulus between the borehole and the PVC pipe as the steel casing was removed.

Fig.1. Site layout and location of monitoring wells.

The sand filter pack was placed from the bottom to approximately 1 ft. above the slotted portion of the pipe and the remainder of the annular space was filled with cement grout. Figure 2 shows a section of the angled vapor extraction well. The angled wells went underneath the southwest wall and are estimated to be 18 ft. below the wall of the building.

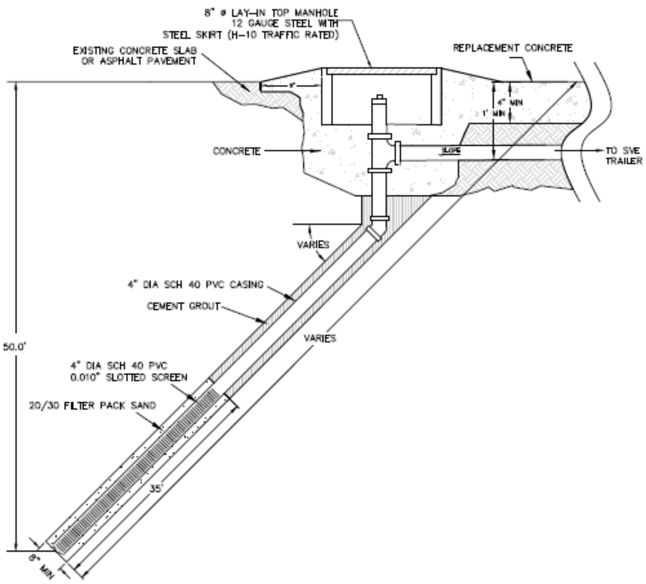

Fig. 2. Angle drilled vapor extraction well section.

In June 2011, the owner reportedly expressed concern of cracks developing on the building as a result of the July 2010 investigation and wanted assurance that subsequent drillings would not exacerbate the problem. Figures 3 and 4 show photographs of the cracks observed during an inspection of the building following the concerns by the owner.

Fig. 3. Observed Cracks on the front brick façade of the building.

Geotechnical forensic investigation was performed to evaluate the potential cause(s) of cracking in the building and whether future drilling would impact the building and the foundation structure system. The investigation involved performing site reconnaissance surveys, site-specific field investigation including real-time vibration monitoring, geotechnical laboratory analyses, and crack monitoring.

Fig. 4. A crack 0.55 in. wide observed on the southwest corner of the building.

SITE DESPRIPTION

Site Layout and Topography

The site naturally slopes from west to east and ranges from approximately elevation 120 to 135 ft. above mean sea level (as referenced to the North American Vertical Datum [NAVD] of 1988). The area immediately southwest of the site is elevated approximately 3 ft. and retained by a 4-ft. high retaining wall, which extends in a northwesterly direction from the southwest corner of the facility. The area to the southwest of the site is grassed and consists of two Sweetgum trees (*Liquidambar Styraciflua*) about 5 ft. from the southwest wall. The site vicinity is predominantly impermeable with areas to the northwest and south comprised of asphalt-paved parking lots. A parcel is located to the east, at generally the same elevation and consists of an unpaved parking lot.

Site Geology

The site-specific lithology generally consists of variably colored clayey and silty sand to sandy clay from land surface to depths ranging from approximately 70 to 81 ft. below land surface (BLS); underlain by alternating limestone and clay to depths ranging from approximately 89 to 90 ft. BLS; underlain by limestone to a depth of at least 197 ft. BLS (E&E, 2007a). The surficial aquifer system for the Tallahassee area typically consists of seasonal and perched groundwater zones. The depth to groundwater in the perched zone ranges from 37 to 60 ft. BLS (E&E, 2007b).

Site Drainage

There are no stormwater drains located on the property. There are two downdrains on the front wall and their outlets appear to be wet. No downdrains are seen on the southwest wall of the structure. However, a previously used drainage pipe and a hole, about 3 in. in diameter, can be seen on the southwest wall. No surface water bodies are located within 0.25 miles of the site. Based on field observations there are no visible weep holes in the retaining wall structure to allow for drainage from the soil retained by the wall. Field observations also indicate a depression in the ground to the southwest of the facility and it is likely that storm water may pond in the area and eventually infiltrate into the underlying soil layers.

Weather Pattern

This section presents the characterization of weather and drainage patterns at the site to evaluate their potential effects on the building foundation and subsurface conditions. In general, Tallahassee experiences hot and humid subtropical climate, with long lasting summers and short, mild winters. According to the National Oceanic and Atmospheric Administration's (NOAA) National Weather Service (NWS), July is the hottest month of the year and the entire summer (June through September) is characterized by brief intense showers and thunderstorms.

Historic temperature and precipitation data were obtained from NWS's Tallahassee weather station. Figure 5 presents the monthly average maximum temperatures for 2010, 2011, and average values from 1981 to 2010. The monthly average maximum temperature for the past 30 years ranged from 63.7 to 92.3 degrees Fahrenheit (ºF). However, in 2010 they ranged from 58.6 in February to 94.7 ºF in June.

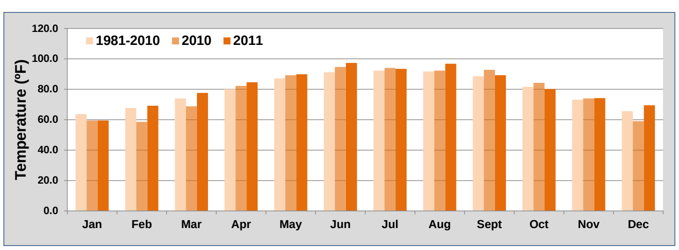

Fig. 5. Monthly average maximum temperatures (ºF) in Tallahassee, Florida.

Further, according to NOAA, June of 2011 recorded the all-time high temperature of 105 ºF in Tallahassee with a monthly average maximum temperature of 97.3 ºF. It should be noted from Fig. 5 that the 2010 and 2011 summer (June through September) temperatures were all higher (0.6 to 6.1 ºF) than the monthly average maximum temperatures for the past 30 years. Further, it was observed that the monthly average temperature for 2010 and 2011 summer were all higher (1.4 to 4.2 ºF) than the average temperatures for the past 30 years.

Precipitation data collected at the Tallahassee Regional Airport, about 7 miles from the site, is presented in Fig. 6. Monthly average precipitation is shown for the years 2010, 2011 and average values from 1981 to 2010.

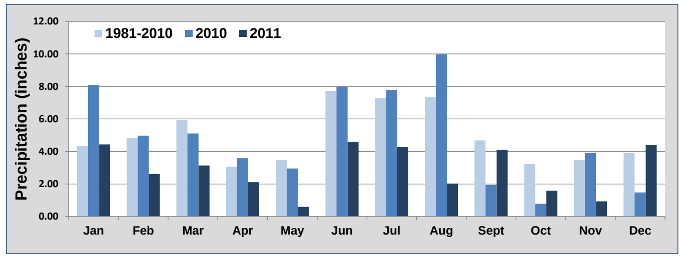

Fig. 6. Monthly average precipitation (inches) in Tallahassee, Florida.

It was observed that August 2010 experienced very high precipitation, immediately followed by two months of very low precipitation as compared to the average precipitation. On September 1, 2010 the NWS of Tallahassee reported "…rainfall at the Tallahassee Regional Airport for August measured 9.97 inches, 2.94 inches above normal. There were 20 days with measurable rainfall which was 5 above normal. The greatest amount in a 24-hour period was 2.31 inches on August 4th..." [http://www.srh.noaa.gov/tae/?n=summer2010] Further, it was observed that the precipitation from January to August 2011 was significantly low relative to the average precipitation.

GEOTECHNICAL INVESTIGATION PROGRAM

The geotechnical forensic investigation program conducted at the site consisted of the following: (i) field reconnaissance; (ii) crack monitoring; (iii) SPT soil borings and sampling; (iv) vibration monitoring during drilling activities; and (v) geotechnical laboratory analyses. The details of the geotechnical investigation program are presented below.

Site Reconnaissance

Following the reporting of the cracks in the structure in July 2011, site reconnaissance surveys were performed in August 2011. The structure was inspected from the inside and the outside for cracking. Areas where potential vertical or horizontal movement of the structure or the structural components had occurred were identified. Areas adjacent to the structure were observed for any visible irregularities.

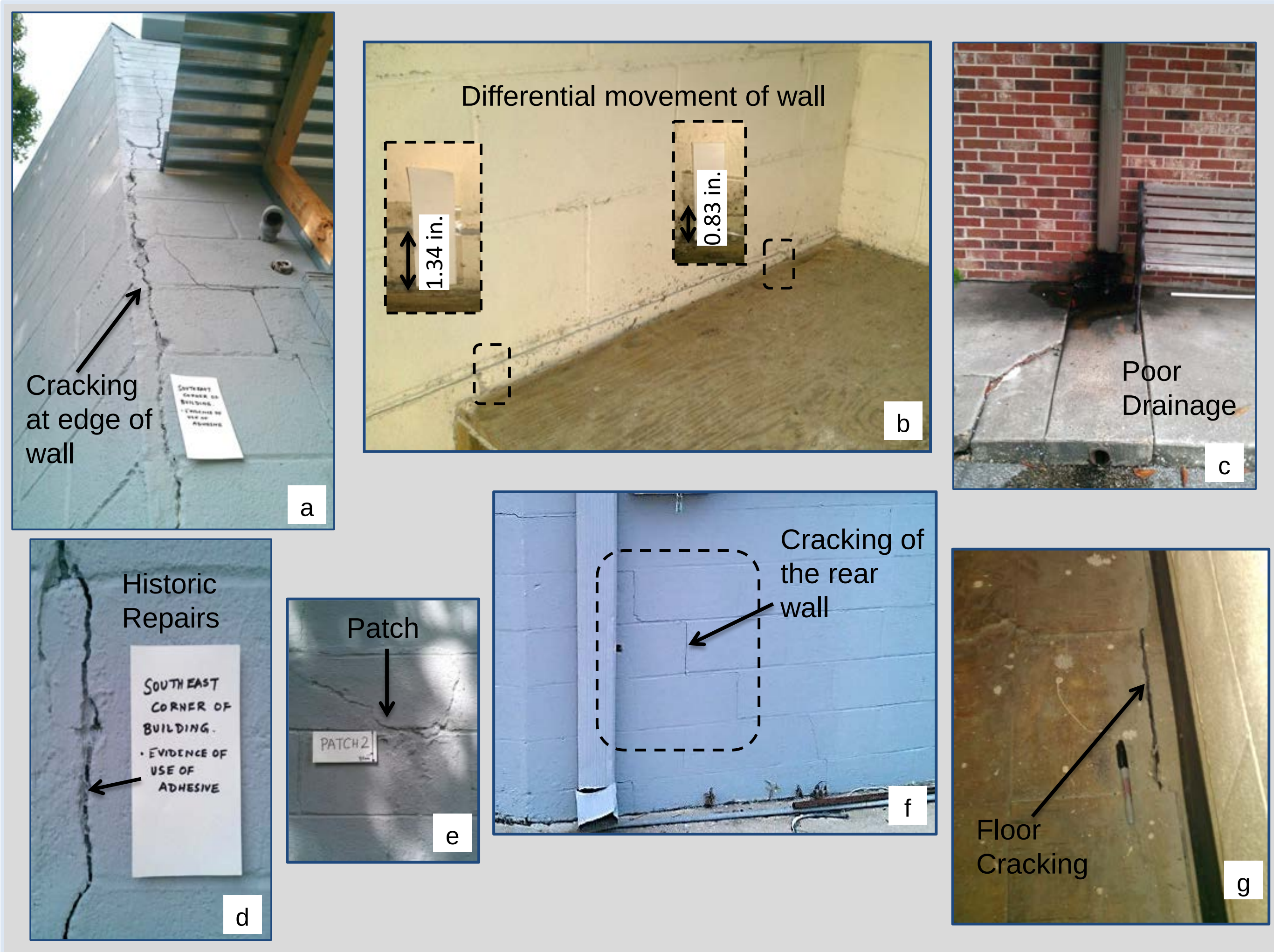

Fig. 7. Various building distresses identified during field reconnaissance survey.

Observations were performed for potential areas experiencing lack of stormwater drainage and thereby leading to stormwater ponding. Various distresses including vertical and horizontal cracks inside and outside the structure, patches, holes, and interior slab settlement were observed. Cracks were observed on the southwest, southeast and on the front wall of the structure. The observed building distresses are depicted in Fig. 7.

A small vertical crack was observed near the drainage downchute on the front façade of the structure. The vertical crack width was in the range of 0.079 to 0.28 in. The crack continued vertically on the wall and then translated to a horizontal crack. The horizontal crack width ranged between 0.18 and 0.55 in. (Fig. 3). The crack depth ranged from 1.18 to 5.12 in. The maximum crack width of 0.55 in. was observed to be at the corner where the southwest wall and the front façade meet (Fig. 4). The crack continued on the southwest wall, which is a masonry wall, of the structure.

Evidence of historical repairs was observed at the site. On the exterior of the southwest wall, three patches were observed (Fig. 7e). On the exterior corner of the southeast wall, a thick crack extending from the roof to the floor slab was observed (Fig. 7a). The crack appeared to be filled-in with an adhesive (Fig. 7d). Cracking was observed on the southeast wall as shown in Fig. 7f.

From the interior of the structure, cracking was observed in the front wall near the roof. Patches were observed on the southwest wall from the interior. Further, patches, paints and cracks were observed at the southeast corner from inside the structure. The inside floor of the structure depicted cracks (Fig. 7g) filled with cement/grout. Outward movement of the wall was observed from the interior of the structure. A gap was observed between the wooden stairs resting on the floor slab and the southwest wall. Differential vertical movement of the southwest wall was observed near the top of the stairs (Fig. 7b).

Close inspection of the cracks and floor repair depicted presence of paint in the cracks and on the grout. The fact that there was evidence of some patches to repair or close some cracks on the building wall in the past indicated that some movement or displacement of the building wall or foundation had already taken place.

Crack Monitoring

A part of the forensic investigation was to evaluate whether the cracks were still developing (i.e., propagating with depth and dimension) or were completely developed. Three crack monitoring devices were fixed on the southwest wall of the structure on August 12, 2011 (Fig. 8). The crack monitoring devices were installed to monitor potential increase or decrease in the crack widths over time. Horizontal and vertical movements of the structure would potentially show up as displacement on the crack monitors. These displacements could be easily read-off from the monitors.

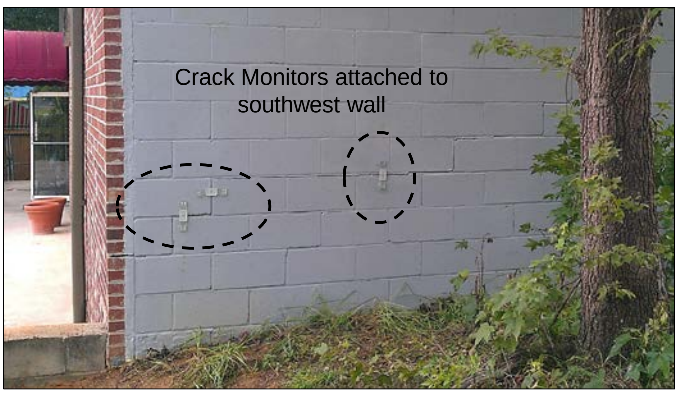

Fig. 8. Crack monitors attached to the southwest wall.

Soil Boring and Sampling

Three SPT soil borings, designated as SB-1 through SB-3, were advanced at the locations shown in Fig. 9 using hollow-stem auger (HSA) drilling techniques. As shown in Fig. 9, all three soil borings were advanced near the southwest wall of the structure which had evidence of cracking. Soil boring SB-1 was advanced to a total depth of 42 ft. whereas SB-2 and SB-3 were drilled to the depths of 32 and 25 ft., respectively. Soil sampling and SPT blow counts were performed using split-spoon sampling procedures (ASTM D 1586) during soil boring advancement. Continuous soil sampling was performed for all the borings up to 25 ft. BLS and then at 5-ft. intervals.

The split-spoon samples collected at each sampling interval were logged and described in general accordance with ASTM D 2488. Select samples were shipped to a geotechnical testing laboratory for index property testing. In addition, thin-walled Shelby-tube samples were obtained at depths of 17 to 19 ft. for SB-1, 11 to 13 ft. and 15 to 17 ft. for SB-2, and 15 to 17 ft. for SB-3. These Shelby tube samples were also shipped to the geotechnical testing laboratory for index property and compressibility tests as described in the following sections. SPT N-values were recorded and lithologic logs were prepared for each boring.

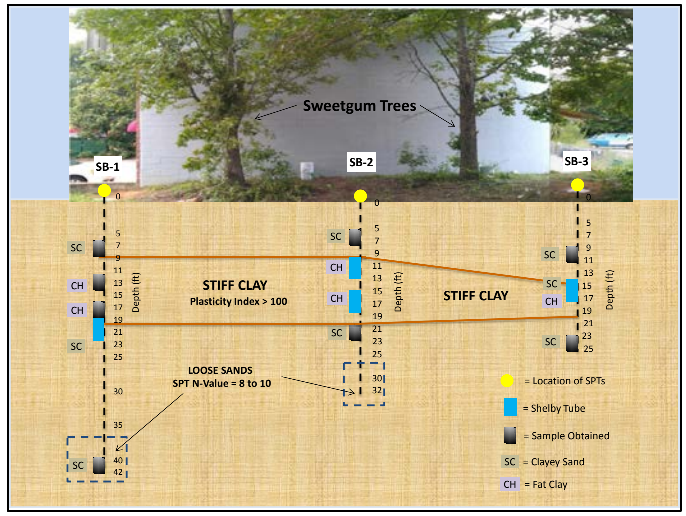

Fig. 9. SPT soil boring and sampling.

Real-time Vibration Monitoring

A real-time vibration monitoring was performed during the SPT soil borings and sampling operations at the site. The monitoring was performed using the JoyWarrior® 24FB strong motion instrument (accelerometer) connected to a personal computer with a software-based 32-bit data acquisition system. This instrument recorded acceleration time histories in three orthogonal directions (two horizontal and one vertical).

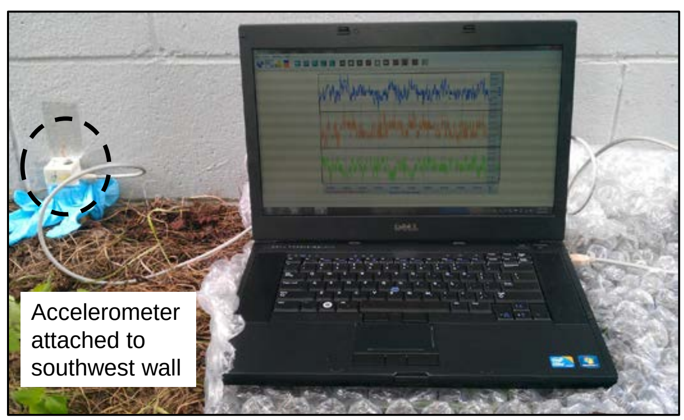

Fig. 10. Real-time vibration monitoring using an accelerometer connected to a personal computer.

The recording location is shown in Fig. 10 relative to drilling location SB-2. The recording was performed during the HSA drilling and the SPTs at boring SB-1 and SB-2. During boring

SB-1, the sensor was attached to the corner of the structure where the southwest and the front wall meet. During boring SB-2, the sensor was attached to the southwest wall of the structure.

Geotechnical Laboratory Analyses

The undisturbed Shelby-tube and select split-spoon samples were subjected to the following laboratory analyses with the applicable ASTM test standard in parenthesis (ASTM 2010):

- In-Situ Moisture Content (ASTM D 2216);
- Particle-Size Analysis (ASTM D 422);
- Atterberg Limits (ASTM D 4318);
- Engineering Classification (ASTM D 2487); and
- One-dimensional consolidation tests (ASTM D 2435)

The one dimensional consolidation tests were performed with the following modifications:

- One specimen was subjected to a seating load of 500 psf and consolidated for approximately 5 minutes, inundated and the vertical pressure was incrementally increased to prevent the specimen from swelling; and

- The other specimen was initially subjected to a similar pressure as the previous specimen at a seating pressure of 100 psf, and was then consolidated at 4000 psf and 8000 psf loads in general accordance with the ASTM D 2435 test procedure.

SUMMARY OF GEOTECHNICAL DATA

Site Stratigraphy

From 0 to approximately 10 ft. BLS, sands and clayey sands were encountered with SPT N-values ranging from 10 to 64. Loose sands were present in boring SB-1, whereas dense sands were present in SB-2 and SB-3. From 10 to 20 ft. BLS, clay with N-values ranging from 16 to 29 was encountered. The clay consistency was identified to be stiff to very stiff (AASHTO, 1988). From 20 to 40 ft. BLS, loose to medium dense sands and clayey sands with N-values ranging from 8 to 25 were present. Groundwater was not encountered during drilling.

Geotechnical Laboratory Test Results

Index Properties. The laboratory test results classified the upper sands and clayey sands as SC per the Unified Soil Classification System (USCS) with moisture contents ranging from 6.2 to 10.1%; fines content ranging from 20.7 to 30.6% and plasticity index (PI) ranging from 15 to 26%. The clay layer which varied in thickness from 11 ft. at SB-1 to 6 ft. at SB-3 was classified as CH per the USCS with moisture content ranging from 51.2 to 58.3%; fines content ranging from 81.4 to 95.2%; and PI ranging from 108 to 120%. The underlying loose to medium dense sands and clayey sands were classified as SC per USCS with moisture contents ranging from 13.9 to 31.1%; fines content ranging from 16.3 to 31.5%; and PI ranging from 12 to 34%.

Compressibility Test Results. One-dimensional consolidation tests were performed on the samples of clay obtained during the field investigation. To evaluate the swelling potential of the clay, two clay specimens were subjected to different seating loads and inundated. For the clay specimen subjected to a seating load of 500 psf, the measured swelling pressure (applied vertical pressure to keep the specimen from swelling) was 2,232 psf. The swelling pressure was measured to be 2,740 psf for the specimen subjected to a seating load of 100 psf. These results definitely confirmed that the clay layer was an expansive soil that would potentially swell when wet from precipitation and shrink when dry during drought conditions.

The compressibility parameters [i.e., modified compression index ($C_{c\varepsilon}$) and modified recompression index ($C_{r\varepsilon}$)] for the clay were measured from the one-dimensional consolidation test results. The calculated value of modified compression index was 0.11 and that of the modified recompression index was 0.003. Further, compressibility parameters of the clay were estimated using the empirical correlations between compressibility and index properties (i.e., natural water content, plasticity index, and liquid limit) (Kulhawy and Mayne, 1990; Lambe and Whitman, 1969; Mesri, 1973; and Skempton, 1944). Based on the estimate, the modified compression index and the modified recompression compression index of the clay were estimated to be 0.53 and 0.072, respectively. It is noted that the empirical correlations estimated higher values than those measured from the one-dimensional consolidation tests.

VIBRATION AND CRACK MONITORING DATA ANALYSES

Vibration Data Analysis

The vibration monitoring events are summarized in Table 1. Also included in Table 1 are the processed results of vibration measurements. The processing was performed by zeroing acceleration records (vibrations are deduced to +/- oscillation around equilibrium), correcting for a drift that may have occurred due to a poor fastening of the sensor to the wall (if any), and conversion from m/s^2 units to "g" units (1 g = 9.81 m/s^2 = acceleration of gravity). One (1.0) g was subtracted from logs of vertical vibrations to separate drilling-induced vibrations from the acceleration of gravity (1.0 g). The peak ground acceleration (PGA) value listed in Table 1 is a vector sum of three components (two horizontal and one vertical). The predominant frequency of the recordings (f) was evaluated by inspection, by stretching the time scale to be able to count records and counting zero-crossings of the time history. The lower bound (1 Hz) corresponds to incidents of blows imparted by the SPT hammer to drive the split-spoon sampler in the ground.

Table 1. Summary of Real-time Vibration Monitoring

Excitation	Distance from Recording Instrument	Vibration Log Number	Recorded PGA (g)	Predominant Frequency of Record (Hz)	Calculated PGV (in./sec)
SB-1	4 ft	000046	0.013	1 – 10	0.01 - 0.1
SB-1	4 ft	000049	0.013	1 - 10	0.01 - 0.1
SB-2	15 ft	000056	0.011	1 - 10	0.01 - 0.1

PGA = Peak Ground Acceleration; PGV = Peak Ground Velocity

To further quantify recorded vibrations, the PGA values were converted to their peak ground velocity (PGV) counterparts using the following equation from the vibrations theory (Bolt, 1999):

$$PGV = PGA / (2 * \Pi * f) \tag{1}$$

where: f = frequency of the vibrations in Hertz (Hz)

For the subject vibration records, calculated PGV ranged from 0.01 in./sec (10 Hz) to 0.1 in./sec (1 Hz). This is graphically shown in a chart developed by Siskind et al. (1980) that is commonly used in mining industry to limit blasting charges to levels that do not induce damage to plaster and/or drywall. This chart is shown in Fig. 11 along with the limits of perceptible vibrations as established by Richart et al. (1970).

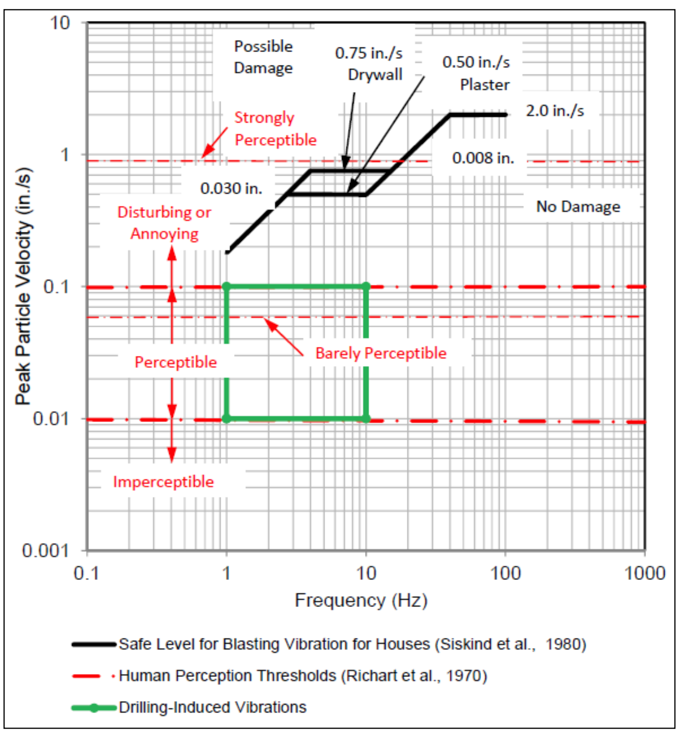

Fig. 11. Vibration data analysis chart.

As can be seen from Fig. 11, evaluated velocity/frequency pairs fall within the range of perceptible vibrations, yet vibrations that do not induce damage to plaster and/or drywall.

Therefore, it can be concluded that the vibrations from the SPT tests using the HSA drilling techniques, were significantly low to have any impact on the structure. Drilling induced vibrations attenuate with distance and as such the vibrations from a rotosonic drilling at 15 ft. from the wall are expected to be less than those from SPT tests using HSA drilling 4 ft. from the wall based on the phenomenon of radiation damping. As such, it was postulated that the rotosonic drilling in July 2010 did not cause the cracks on the building.

Crack Monitoring Data Analysis

On September 27, 2011 a condition survey was performed to observe and record additional displacements of the cracks since installation of the monitors in August 2011. The crack monitors depicted additional displacements since installation on August 12, 2011. Figure 12 depicts the measured displacements in 45 days. Crack monitor 1 showed 0.0197-in. vertical and horizontal displacements. Crack monitor 2 showed a vertical displacement of 0.0197-in. and horizontal displacement of 0.0295-in. Crack monitor 3 depicted a vertical displacement of 0.0591-in. and no horizontal displacement.

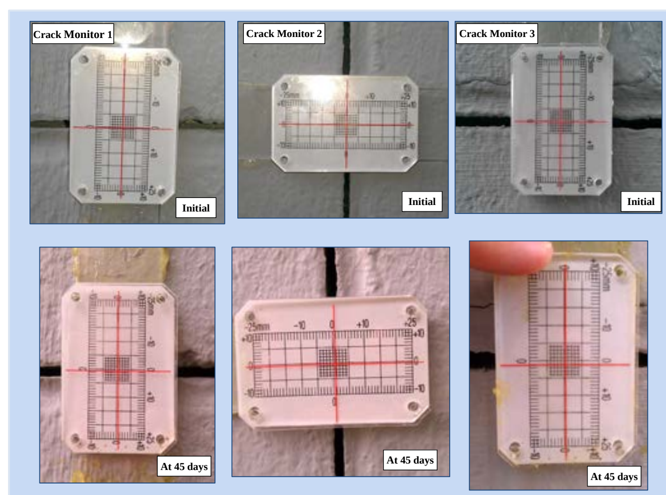

Fig. 12. Crack monitors depicting displacements.

The displacements indicate that there have been continued movements of the structure since their installation on August 12, 2011. It should be noted that these additional displacements were observed in the absence of drilling activities.

DISCUSSION OF POSSIBLE CAUSES OF CRACKS ON THE BUILDING

Overview

The analysis of the real-time vibration monitoring performed at the site during the SPT soil borings using HSA drilling technique for the geotechnical investigation conducted as part of this study concluded that the resulting vibrations were too low to have an impact on the building. It was also postulated that the vibrations from a rotosonic drilling are expected to be less than those from the HSA drilling such that it is very unlikely that the drilling performed in July 2010 was responsible for the cracks that had developed on this building. In addition, observations and photographic documentation conducted as part of the site reconnaissance indicated that the building had already experienced some crack development and been repaired and patched in the past. Some of these cracks have since been observed to be increasing in width or propagating further than initially (August 2011) noticed; thereby suggesting that additional movements or displacements were going on with the building structure and foundation.

Based on the results of the geotechnical investigations, the following possible cause(s) for the development of the cracks in the front and southwest wall of the structure were evaluated:

- Differential settlement and/or heave due to expansion and shrinkage of the fat clay layer underneath the building; and
- Poor drainage and impact of root penetration underneath the floor slab.

It is most likely that a combination of the above contributed to the cracking on the building wall. The remainder of this section provides a discussion of the above possible cause(s) for the cracking on the building wall.

Expansion and Shrinkage of Fat Clay Layer

As observed from laboratory test results presented in previous section, the area near the southwest wall showed presence of fat clays. Further, during previous monitoring well installations, similar clays were observed in several borings around the site. Fat or expansive clays characteristically exhibit volumetric expansion and shrinkage due to infiltration of moisture in wet seasons and evaporation in dry seasons and thereby resulting in settlement or heave. The temperature and precipitation patterns in Tallahassee for the past few years were discussed in previous sections. It can be inferred from the data that the site experienced very high temperatures in summer 2010. Simultaneously, initial very high precipitation in August 2010 was followed by low precipitation or dry periods toward the end of the year 2010 and early 2011. It is most probable that the higher precipitation resulted in ponding and further infiltration of stormwater in the area near the southwest wall. The infiltrated water reached down to and saturated the expansive clay approximately 10 ft. BLS. This was followed by comparatively lower precipitation and dry period leading to moisture egress from the clayey soils. The expansive clay experienced moisture fluctuations. Moisture reduction led to volumetric shrinkage of the clay. Since the thickness of the clay layer varies from 11 ft. at SB-1 to 6 ft. at SB-3, the volumetric shrinkage was differential. The differential volumetric shrinkage led to differential settlement of the clay layer. Thus, greater settlement occurred for the thicker clay layer which showed up as a crack at the corner of the structure.

Further, laboratory test results indicated swelling pressure of greater than 2,200 psf for the fat clay. If the foundation loads of the one-story structure were of a lesser magnitude than the swelling pressure of the clay, it is likely that the clay would heave when inundated. The excess swell pressure (difference between the swell pressure and the foundation pressure) would then act as uplift pressure on the foundation resulting in movements of the wall. Also, differential heave could occur if the load on the strip footing was greater than that on the floor slab.

The geotechnical literature contains numerous examples of foundation damage due to swelling and shrinkage from expansive clays. The literature also notes that process of expansion and shrinkage resulting in damage to buildings is a slow process such that it takes time for the bigger or visible problems to manifest. The development and propagation of cracks on the building meet the observations of foundation damage from expansive soils reported in the geotechnical literature. It is likely that the cyclic process of expansion and shrinkage of the clay layer at the site has been taking place since construction of the building, which provides an account for the historic repairs at the site.

Effects of Poor Drainage and Root Penetration

The presence of the Sweetgum trees and other form of vegetation near the southwest wall of the structure deserve special attention. The Sweetgum tree near the cracked corner of the building has one of its major roots advanced toward the cracked corner. The problem with trees is that their roots withdraw moisture from the soil in a local area. During dry periods, the ground surface may be dried out and moist soil may exist beneath the building where surface evapotranspiration has been prevented. Thus, the tree sends its roots beneath the structure causing localized drying and shrinkage (Tand and Vipulanandan, 2011), consequently damaging the structure. In that case, the corner of the structure would experience settlement which would then show up as a crack as currently observed at the site. Biddle (2001) describes the tree system as a living pipeline for the upward flow of water from the ground. Historically, such occurrences have been reported. For example, Driscoll (1984) collected data showing the maximum distance from the house to a tree that caused damage. Table 2 presents some of his data, with trees listed in decreasing order of damage claims. For the cracks observed at the site, the southwest wall might be experiencing vertical and/or horizontal movements as a result of the root growth; roots withdrawing moisture; roots pushing on the wall; or a combination of these.

Table 2. Distances for Damage from Trees (Driscoll, 1984)

Rank	Species	Maximum Height (ft)	Max. Dist. for no Damage (ft.)	Dist. to Damage in 75% of the Cases (ft.)
1	Oak	50-75	100	45
2	Poplar	80	100	50
3	Lime	50-80	65	25
4	Ash	75	70	35
5	Plane	80-100	50	25
6	Willow	50	130	35
7	Elm	65-80	80	40
8	Hawthorn	30	35	25
9	Maple/Sycamore	55-80	65	30
10	Cherry/plum	25	35	20
11	Beech	65	50	30
12	Birch	40-45	35	25

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

Based on the results of real-time vibration monitoring analysis, it was concluded that the rotosonic drilling performed in July 2010 was not responsible for the cracks that had developed on this building. Therefore, future drilling activities (using rotosonic drilling or equivalent techniques) were not expected to cause additional cracks or affect the integrity of the building. This conclusion was recommended to be verified during future drilling activities at the site. The results of the SPT soil borings and laboratory analyses indicated that soils underlying the building included a fat clay (CH) layer at depths of approximately 10 to 20 ft. BLS. Fat clays are expansive soils that expand and shrink due to changes in moisture content and consequently result in potential damages to building foundations. It was concluded that the possible cause(s) of the cracks on the building were: (i) volumetric changes in the expansive clays due to moisture fluctuation; (ii) differential settlement and/or heave of the southwest wall; (iii) poor drainage conditions and roots of the Sweetgum tree growing underneath and pushing/uplifting the wall; and/or (iv) combination of all three scenarios.

Recommendations

Continued Crack Monitoring. From the crack monitoring, it was concluded that the structure was experiencing continued vertical movements. Continued crack monitoring was recommended to evaluate increase and/or decrease in the crack width. The increase/decrease in the crack widths can be correlated to the wetting and drying seasons to determine if volumetric changes in the expansive clay are the cause for the cracks.

Real-time Vibration Monitoring during Rotosonic Drilling. Additional real-time vibration monitoring and analysis was recommended to be performed during future rotosonic drilling at the site to confirm the aforementioned conclusion.

Eliminate potential problems with trees and its roots. One other recommendation was to remove and/or relocate both the Sweetgum trees to a distance of 75% of the tree height from the wall. In case the trees cannot be relocated, the roots of the trees growing toward/underneath shall be identified and cut off to prevent future growth.

Other Recommendations. Other recommendations for similar problems on expansive clays reported in the literature and that could potentially be used on a case by case basis are: (i) Use of drilled piers, piles; (ii) Use of mud-jacking; (iii) Removing and replacing the soil; (iv) Chemically treating the soil; (v) Controlling surface drainage; and (vi) Wetting the soils during dry seasons.

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