Investigation of a shallow slope failure on expansive clay in Texas

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A B S T R A C T
Highway slopes constructed with highly plastic clay soils undergo shrink-swell cycles during the wet-dry season, which usually soften the soil and may cause failure a few years following construction. There are two major wet-dry cycles in Texas, where summer to early fall is categorized as the dry period, and late fall to spring is categorized as the wet period. During this study, the failure of a highway slope located in Fort Worth, Texas was investigated, and the effect of rainfall on the slope failure was evaluated. The failure of the slope was investigated using soil test borings, geophysical testing, laboratory testing, slope stability analysis, and unsaturated flow analysis. The site investigation included sample collections from different depths of borings, determination of the soil index properties, and various shear strength testing to determine soil strength parameters. The obtained test results were further utilized in slope stability and flow analyses using the finite element method (FEM) in PLAXIS. The study indicated that both a fully-softened condition and rainfall were responsible for the slope failure. The fully-softened condition provided a factor of safety of 1.46, whereas the perched water condition after rainfall provided a nearly failing factor of safety of 1.05.

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1. Introduction

Most highway fill slopes in the North Texas areas are constructed using in-situ high plasticity clay soil, which is highly expansive in nature. These fill slopes have recurring shallow failures a few years after construction, causing a significant maintenance problem for the Texas Department of Transportation (TxDOT). Shallow slope failures generally do not constitute a hazard to human life or cause major damage; however, they may constitute hazards to infrastructures by causing damage to guardrails, shoulders, road surfaces, drainage facilities, utility poles, or the slope landscaping (Day and Axten, 1989; Titi and Helwany, 2002). In some cases, shallow slope failures affect regular traffic flow if debris flows onto roadway pavements.

Clay minerals consist of two or three layers of gibbsite and silica sheets (Das, 2013). These are negatively charged aluminosilicate layers with the ability to absorb water between the layers (Hensen et al., 2002). Due to the presence of exchangeable ions on the surface of clay particles, dipolar water molecules are attracted towards clay minerals, resulting in a double layer of water. The plastic property of clay is attributed to the presence of this double layer.

The surface area of clay particles per unit mass is generally referred to as specific surface. The specific surfaces of kaolinite, illite, and montmorillonite are about 15, 90, and 800 m²/g, respectively (Das, 2013). Clay minerals that have less surface area attract less water (such as illite and kaolinite) and are characterized as low plastic clay. On the other hand, montmorillonite has a significantly high surface area, which attracts a high volume of water, and is characterized as high plasticity clay. The thickness of the double layer water is highest for montmorillonite (Das, 2013). Due to the affinity of a high volume of water, high plastic clay exhibits expansive behavior: it swells when it absorbs water and shrinks when water is dissipated. Chabrillat et al. (2002) reported an abundance of montmorillonite in clays in the mid-zone of the United States. Previous studies indicated the presence of montmorillonite minerals in the Dallas-Fort Worth (DFW)-area clay (Punthutaecha et al., 2006; Puppala et al., 2013). Absorbance of water in the wet season results in volume expansion; the opposite is true in the dry season. Rahardjo et al. (2001) presented a case study that described the effect of antecedent rainfall on slope stability in Singapore. The authors investigated several shallow slope failures on residual soil that occurred after a rainfall event of 95 mm (3.75 in.) within 12 h. The authors reported that the landslides were initiated by rainwater infiltration; that is, no changes in geometry or additional loading applied to the slopes could have initiated failure. In addition, both the daily rainfall and the antecedent rainfall were important triggering factors for the occurrence of the landslides.

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Rahimi et al. (2010) conducted a study on rainfall-induced slope failure due to antecedent rainfall for high and low conductivity residual soils of Singapore. The authors applied three antecedent rainfall patterns to soil slopes and conducted a transient seepage analysis to investigate the effect of rainfall on the stability of the slope. Results from the study indicated that antecedent rainfall affected the stability of both high-conductivity and low-conductivity soil slopes, with the stability of the low-conductivity soil slopes being more significantly affected. In addition, the stability of the slope was controlled by the amount of rainfall that infiltrated the unsaturated zone of the slope.

Highway slopes constructed on highly plastic clay soil are usually very strong immediately following construction. Skempton (1977) first reported that over time, the strength of slopes in the highly plastic London clay decreases, eventually reaching what Skempton termed a “fully-softened” strength. Skempton (1977) indicated that fully-softened strength is comparable to the shear strength of the soil in a normally consolidated state. A few years after construction, shrink-swell behavior can reduce the shear strength of the top few feet of a slope, which is susceptible to moisture variations. After the soil reaches the fully-softened shear strength, water that infiltrates the soil during intense and prolonged rainfall events may cause excess pore water pressure and compound the overall problem. As a result, the factor of safety of the slope may reduce to unity and approach failure (Wright, 2005). These types of failures occur within the upper 0.91 m–1.82 m (3 ft–6 ft) of the slope, and the failure surface is generally parallel to the slope face (Wright, 2005). The surficial failure may occur 3–7 years after construction. Surficial failure can even take decades to form, depending on the frequency of extreme weather conditions.

The shallow slope failure of a highway slope located in Fort Worth, Texas was investigated during this study. It was conducted using standard soil tests, geophysical testing, electrical resistivity tomography (ERT), extensive numerical modeling, and the finite element method (FEM). The strength parameters of the slope were determined using the direct shear (DS) and consolidated undrained (CU) triaxial tests.

Fig. 1. (a) Failure Condition of SH 183 Slope (Plan View), (B) Section 1–1, (C) Failure near Crest.
The obtained parameters were used to conduct a slope stability analysis in PLAXIS 2D. Unsaturated flow analyses were also conducted to understand the rainwater flow and its influence on the slope failure after a prolonged summer period.

2. Project background and site geology

The investigated slope was located along SH 183, east of the exit ramp from eastbound SH 183 to northbound SH 360, in the northeast corner of Tarrant County. The schematic and failure photos of the slope are presented in Fig. 1. The height of the slope was approximately 10.97 m (36 ft), with a slope geometry of 2.5(H): 1(V).

The project site was located on the Eagle Ford formation and contained shale, siltstone, and limestone. In the upper part, the formation was lime and shale, yellowish-brown, and flaggy. The lower part of the formation was composed of silt and very fine-grained sandstone, yellow to gray in color, and mostly laminated flaggy; however, some limestone is silty, medium-brown in color, and laminated (U.S. Geological Survey, 2014). The Eagle Ford formation consisted of sedimentary rock that was in the process of degrading into a soil mass. This formation also contained smectite clay minerals and sulfates. The swell potential, compressibility, and creep deformation were expected to be high in the Eagle Ford Shale due to the high percentage of smectite (Hsu and Nelson, 2002). This could lead to problems such as slope failures, foundation damage, mine failures, and shale embankment failures (Abrams and Wright, 1972).

3. Site investigation

The site investigation included a collection of samples from three test borings, designated as BH-1, BH-2, and BH-3. BH-1 and BH-2 were located near the crest of the slope, and BH-3 was located at the toe of the slope. The collected samples were tested in the laboratory to investigate the soil index properties and shear strength parameters that would be used in further analyses. Geophysical investigation was carried out using 2D electrical resistivity tomography (ERT). The layout of the soil test borings and ERT lines are presented in Fig. 2.

3.1. Standard soil tests

The soil index properties, moisture content (MC), liquid limit (LL), and the plasticity index (PI) of the soil samples obtained from site are presented in Fig. 3. The liquid limit and plasticity index tests were conducted in accordance with the ASTM D 4318 guidelines. The LL of the soil samples were within a range of 36 to 48 for the top 8 ft, near the crest of the slope from BH-1 and BH-2. The PI values ranged between 20 and 24. The LL ranged between 60 and 64, and PI ranged between 24 and 35 in the top 8 ft near BH-3. The LL and PI values ranged from 48 to 64 and 31–42, respectively, for the soil below 8 ft. The soil test results were analyzed according to the Unified Soil Classification System (USCS) and were classified as low-to-highly plastic clay (CL-CH) for boreholes BH-1 and BH-2, and highly plastic clay (CH) for BH-3. The variation of moisture content with the depth of each borehole is shown in Fig. 3, which indicates that the moisture content of borehole BH-1 was 20% to 23%, while borehole BH-2 had a moisture content of 17% to 21%.
Fig. 4. (a) Test results of the resistivity line RI-1 (at crest), (b) Test results of the resistivity line RI-2 (at the middle of the slope), and (c) Test results of the resistivity line 3 (at toe of the slope).

Fig. 5. a. Comparison of ERT at failed area during April 2014 and July 2016, b. Vertical ERT variation along Line 1, c. Vertical ERT variation along Line 2.
3.2. 2D resistivity imaging

Electrical resistivity tomography (ERT) is one of the most popular geophysical investigation methods which is used to determine the subsurface condition of the soil. ERT is used extensively in geophysical investigations of soils at shallow depths and in geo-hazard studies (Hossain et al., 2010; Hossain et al., 2012; Kibria and Hossain, 2012). The resistivity of the soil primarily depends on the type of soil, moisture conditions, and void ratio. Kibria and Hossain (2016) developed a model, that correlate electrical resistivity with the degree of saturation of highly plastic clay soil, as presented in the following.

\[
\text{Electrical Resistivity} \sim 0.75 = -0.0106 \times 0.00299 \times \text{(Degree of Saturation)}
\]

The study was conducted using dipole-dipole array which is widely used for high sensitivity to horizontal change, and better horizontal data coverage for 2D survey. In this study, three 2D ERT tests, designated as ERT-1, ERT-2, and ERT-3, were conducted near the crest, middle, and toe of the slope. Investigation of the subsurface conditions included analysis of the lateral and vertical variations in subsurface moisture content and near surface geology. The ERT tests on the slope were conducted using 8-Channel Super Sting equipment, with 56 electrodes at 1.5 m (5 ft) c/c spacing, at the end of April 2014 during the early summer period in Texas. As a result, the length of each ERT line was 84 m (275 ft). The recorded data from the ERT tests were analyzed using the Earth Imager 2D software, and the obtained resistivity profiles are presented in Fig. 4.

Based on the resistivity imaging profile at ERT-1, a comparatively high resistivity zone was observed up to the top 2.13 m (7 ft) depth (Fig. 4a), which might suggest the existence of a low moisture zone near the top of the slope. It should be noted that the ERT test was conducted during the early summer period; therefore, the low moisture zone likely indicated the active zone. Moreover, a low resistivity zone was found in ERT-1 after 2.13 m (7 ft) depth, which might indicate the presence of high moisture below the active zone.

Testing at ERT-2 was conducted over the tension crack at the middle of the slope (Fig. 1b). A high resistivity zone was observed immediately below the tension crack zone in the resistivity profile of ERT-2, where the depth of the high resistivity zone was 3.66 m (12 ft). Electricity was unable to pass through the tension crack, resulting in very high resistivity. On the other hand, a low resistivity zone was observed below the tension crack, marking the presence of a saturated zone due to rainwater intrusion through the crack. The electrical resistivity tomography line ERT-3 was located at the bottom of the slope; however, no distinct moisture or high resistivity zone was observed.

Another ERT test was conducted in July 2016 to investigate the resistivity variation along ERT-1 at the failed area near the crest of the slope and verify the depth of the moisture variation zone. The variation of the ERT at the failure location between April 2014 and July 2016 is presented in Fig. 5(a). Moreover, the variation of resistivity with depth along lines Line-1 and Line-2 is presented in Fig. 5(b) and Fig. 5(c). Due to the temperature and moisture distribution, evaporation occurring in the surficial soil caused moisture variation. For highly plastic clay soil, the depth of the moisture variation zone is known as the depth of active zone. The vertical resistivity profiles in Fig. 5(b) and Fig. 5(c) present the resistivity variation at the upper 2.13 m (7 ft) depth at different time periods. The observed resistivity variation occurred as a result of seasonal moisture variation. Therefore, the depth of the active zone was determined to be 7 ft.

3.3. Temperature and rainfall

As a part of the site investigation, the temperature and rainfall data of the slope was analyzed during the failure. The slope was located in

<table>
<thead>
<tr>
<th>Date</th>
<th>Avg. Temperature (°C)</th>
<th>Rainfall (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/1/2011</td>
<td>0.00299</td>
<td>75</td>
</tr>
<tr>
<td>1/16/2011</td>
<td>0.0106</td>
<td>20</td>
</tr>
<tr>
<td>1/31/2011</td>
<td>5/16/2011</td>
<td>30</td>
</tr>
<tr>
<td>2/15/2011</td>
<td>5/31/2011</td>
<td>40</td>
</tr>
<tr>
<td>3/17/2011</td>
<td>6/30/2011</td>
<td>60</td>
</tr>
<tr>
<td>4/1/2011</td>
<td>7/15/2011</td>
<td>70</td>
</tr>
<tr>
<td>4/16/2011</td>
<td>7/30/2011</td>
<td>80</td>
</tr>
<tr>
<td>5/1/2011</td>
<td>8/14/2011</td>
<td>0</td>
</tr>
<tr>
<td>5/16/2011</td>
<td>9/13/2011</td>
<td>10</td>
</tr>
<tr>
<td>6/15/2011</td>
<td>10/13/2011</td>
<td>20</td>
</tr>
<tr>
<td>7/15/2011</td>
<td>11/12/2011</td>
<td>30</td>
</tr>
<tr>
<td>8/14/2011</td>
<td>12/12/2011</td>
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</tr>
<tr>
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<td>12/27/2011</td>
<td>0</td>
</tr>
<tr>
<td>10/13/2011</td>
<td>4/1/2011</td>
<td>15</td>
</tr>
<tr>
<td>10/28/2011</td>
<td>4/16/2011</td>
<td>20</td>
</tr>
</tbody>
</table>

Fig. 6. Rainfall and temperature of the slope site during the year of 2011 (NOAA, 2014).

Table 1
Summary of shear strength test on intact samples.

<table>
<thead>
<tr>
<th>Borehole No.</th>
<th>Sample depth (m)</th>
<th>Specimen</th>
<th>Moisture content</th>
<th>Test type</th>
<th>Test standard</th>
<th>Cohesion (kPa)</th>
<th>Friction angle (deg.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH-1</td>
<td>1</td>
<td>Intact</td>
<td>16%</td>
<td>Direct Shear</td>
<td>ASTM D3080</td>
<td>26.33</td>
<td>21</td>
</tr>
<tr>
<td>BH-2</td>
<td>5.8</td>
<td>Intact</td>
<td>20%</td>
<td>Direct Shear</td>
<td>ASTM D3080</td>
<td>23.94</td>
<td>13</td>
</tr>
<tr>
<td>BH-3</td>
<td>1</td>
<td>Intact</td>
<td>19%</td>
<td>Direct Shear</td>
<td>ASTM D3080</td>
<td>19.29</td>
<td>20</td>
</tr>
<tr>
<td>BH-3</td>
<td>4.6</td>
<td>Intact</td>
<td>21%</td>
<td>Consolidated Undrained (CU) Triaxial</td>
<td>ASTM D4767</td>
<td>29.20</td>
<td>12</td>
</tr>
<tr>
<td>BH-3</td>
<td>5.8</td>
<td>Intact</td>
<td>20%</td>
<td>Direct Shear</td>
<td>ASTM D3080</td>
<td>45.49</td>
<td>16</td>
</tr>
<tr>
<td>BH-3</td>
<td>1</td>
<td>Remolded with Wet-Dry Cycle</td>
<td>N/A</td>
<td>Direct Shear</td>
<td>ASTM D3080</td>
<td>5.75</td>
<td>21</td>
</tr>
</tbody>
</table>
close proximity to the Dallas Fort Worth (DFW) Airport. Therefore, the temperature and rainfall data of the DFW Airport from the National Oceanic and Atmospheric Administration website (NOAA, 2014) were utilized and are presented in Fig. 6. Weather data indicated that there was a prolonged summer and few rainfall events prior to the slope failure in October 2011. The slope was constructed using highly plastic clay soil with a plasticity index value of 30 and higher. According to Holtz and Kovacs (1981), soil may have a high degree of expansion due to moisture variations. The weather data showed that there was a long, dry period from June 2011 to October 2011. Due to the prolonged drought, there were significant shrinkage cracks, which possibly acted as a vertical preferential path during rainfall. The rainwater likely remained underneath the active zone because of the low permeability of the highly plastic clay soil. Therefore, a flow analysis

Table 2
Soil parameters for safety analysis.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Friction angle</th>
<th>Cohesion</th>
<th>Unit weight</th>
<th>Elastic modulus</th>
<th>Poisson ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>- (Fully-Softened)*</td>
<td>n/a</td>
<td>kPa</td>
<td>kN/m³</td>
<td>kPa</td>
<td>n/a</td>
</tr>
<tr>
<td>1 (Peak Shear Strength)</td>
<td>20</td>
<td>5.75</td>
<td>19.66</td>
<td>4788</td>
<td>0.35</td>
</tr>
<tr>
<td>2 (Peak Shear Strength)</td>
<td>20</td>
<td>19.2</td>
<td>19.66</td>
<td>4788</td>
<td>0.30</td>
</tr>
<tr>
<td>3 (Peak Shear Strength)</td>
<td>20</td>
<td>45.5</td>
<td>20.52</td>
<td>9576</td>
<td>0.25</td>
</tr>
</tbody>
</table>

* Sample prepared according to the guidelines of US Army Corps of Engineers (Stephens and Branch, 2013).

Fig. 7. (a) Initial model, (b) Slope stability analysis with fully-softened strength at top 2.1 m (7 ft), FS = 1.46.
was required to evaluate the effect of the rainfall through the top soil of the slope.

### 3.4. Shear strength parameters

Previous studies indicated that initial failure occurs when fully-softened strength develops under environmental conditions such as shrinkage-swelling and wetting-drying in both excavated and fill slopes constructed over highly plastic clay soil (Saleh and Wright 1997). Hence, the peak and fully-softened shear strength of the soil were determined for the current study using the DS and CU triaxial test methods. A major advantage of using the direct shear test is that it allows sample preparation from slurry into a shear box to determine the fully-softened shear strength. Moreover, the test results for the remolded samples from both the direct shear test and the triaxial test are similar (Castellanos and Brandon, 2013).

Undisturbed soil samples from test borings at different depths were utilized to determine the peak shear strength using both DS and CU tests. The fully-softened strength was determined from the DS test conducted on remolded samples from BH-3 that had been subjected to a single wet-dry cycle. The sample for the fully soften shear strength was prepared according to the guidelines of US Army Corps of Engineers (Stephens and Branch, 2013). The obtained test results are summarized in Table 1.

The shear strength test results indicated that the peak shear strength of the soil was comparatively higher for the undisturbed samples, both in the top soil (at 1 m depth) and in the foundation soil (at 4.6 m and 5.8 m depth). Based on the direct shear test results, the minimum peak cohesion was 19.29 kPa and the friction angle was 20 deg. at 1 m (3 ft) depth. The peak shear strength of the foundation soil at 4.6 m (15 ft) and 5.8 m (19 ft) was stronger (minimum cohesion was 23.94 kPa and friction angle was 13 deg.). On the other hand, a significant drop of cohesion was observed at the fully-softened strength. Usually, the friction of the sample remains the same, but a significant drop of cohesion takes place during the fully-softened shear strength (Wright, 2005; Wright et al., 2007; Zornberg et al., 2007). The shear strength test results were utilized for the slope stability analysis that are further described in Section 4 and Section 6.

### 4. Slope stability analyses

The FEM program PLAXIS 2D was used to perform the slope stability analyses, utilizing an elastic, perfectly plastic Mohr Coulomb soil model. A 15-node triangular element was used, which had nine stress points and provided high quality stress results for different problems (Brinkgreve and Broere, 2010). Standard fixities were applied as a boundary condition in PLAXIS.

Slope failure is initiated when the factor of safety reaches the value of 1.0 during the analysis. The factor of safety of a soil slope is defined as the factor by which the original shear strength parameters can be reduced in order to bring the slope to the point of failure in the shear strength reduction method (Griffith and Lane, 1999). The soil parameters, as shown in Table 2, were used in the numerical analysis using PLAXIS 2D. The initial soil model is shown in Fig. 7. During the slope stability analysis, the top 2.13 m (7 ft) was considered as the active zone, based on the ERT results. The phi-c reduction analysis was conducted to obtain the factor of safety of the slope. The factor of safety of the slope was observed as 1.46, which indicated that the slope was stable even under the fully-softened condition. Therefore, the slope might have failed because of the formation of a perched water condition resulting from post-summer rainfall events. To evaluate the formation of the perched water zone, a flow analysis was conducted.

### 5. Flow analyses

#### 5.1. Flow parameters and initial model

Precipitation of different intensities was added to the soil model to evaluate the flow behavior during rainfall. The analysis was conducted at three rainfall intensities: 0.0012 mm/s (0.167 in./h), 0.0019 mm/s (0.271 in./h), and 0.0022 mm/s (0.3125 in./h). The rainfall intensities were selected based on 2, 5, and 10 year periods of Texas rainfall data collected from National Oceanic and Atmospheric Administration (NOAA, 2014). The flow through the top soil was determined for each of the intensities assuming rainfall durations lasting 3 h, 6 h, 12 h, and 24 h.

The soil model used for safety analysis (Fig. 7a) was again utilized to simulate the rainfall behavior. It should be noted that during the dry

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Fig. 8. Soil water characteristics curve for high plastic clay (Hossain, 2012).
period, the highly plastic clay soil developed desiccation cracks which might have significantly increased the permeability along the vertical direction of the top soil at the active zone. Albrecht and Benson (2001) conducted a study on the effect of desiccation on compacted clay. The study reported that the hydraulic conductivity of clay soil increased as much as 500 times during drying, where the largest increase in hydraulic conductivity occurred after first drying cycle. However, due to the desiccation crack, the permeability along the horizontal direction might have had no effect and could have remained unchanged.

Omidi et al. (1996) conducted a study on the effect of desiccation cracking on vertical permeability. The author conducted permeability tests on both undesiccated and desiccated clay samples (smectite and illite) and found considerable differences. The ratio of permeability for a desiccated sample to an undesiccated sample was 21.7 for illitic soil and 452 for smectitic soil. Consequently, the permeability along the vertical direction was found to be higher, to a value closer to that of silt. The permeability along all remaining directions was uniform and considered as clay.

![Schematic of rainfall intrusion through desiccation crack](image1)

![Boundary conditions for the flow model](image2)

![Suction profile of slope prior to rainfall](image3)

![Suction profile 16 h after rainfall](image4)

![Suction profile 2 days after rainfall](image5)

![Suction profile 7 days after rainfall](image6)

**Fig. 9.** (a) Schematic of rainfall intrusion through desiccation crack (b) Boundary conditions for the flow model (c) Suction profile of slope prior to rainfall (d) Suction profile 16 h after rainfall (e) Suction profile 2 days after rainfall (f) Suction profile 7 days after rainfall.
Rainfall with uniform intensity was applied during the flow analysis, and the Van Genuchten model was used for the flow parameters. From the electrical resistivity tomography results, the top 2.13 m (7 ft) was determined to be the possible active zone and may have possessed the desiccation crack. Therefore, a high vertical permeability value of $k_y = 1.063 \text{ m/day (1.23 \times 10^{-5} \text{ m/s})}$ was used for the top 2.13 m (7 ft) of the slope to simulate the effect of the desiccation crack. In other clay layers, the permeability for both horizontal and vertical directions was selected as $0.0475 \text{ m/day (5.5 \times 10^{-7} \text{ m/s})}$. Hence, the ratio of vertical to horizontal permeability became approximately 22.

Hossain (2012) conducted a study of highly plastic clay soil in Fort Worth, Texas and determined the soil water retention curve (SWRC) at different normal stresses. It should be noted that the LL and PI of the soil test by Hossain (2012) closely resembled the physical properties of the soil sample of the current study. Therefore, the Van Genuchten fitting parameters ($\alpha = 0.064$, $n = 1.219$ and $m = 0.1797$), as suggested by Hossain (2012), were utilized for the flow analysis. The SWRC curve for the flow analysis is presented in Fig. 8.

5.2. Flow analysis results

In the flow analysis, the left and right boundaries were selected as the closed boundaries, and the top of the slope was selected as rainfall infiltration (Fig. 9 a, b). A constant head of 6.1 m (20 ft) was selected for the bottom boundary. The water table was assumed to begin 15.24 m (50 ft) below the top of the slope, as revealed during the site investigation, to define the initial unsaturated condition.

The variations of suction at the crest of the slope for a 12-h rainfall are presented in Figs. 9(c) to 9(f). The suction immediately dropped at the top after rainfall and continued to drop for two days, representing the accumulation of water at the corresponding depth. After 7 days, the suction had almost regained its original profile.

The variations of change in suction at 0.91 m (3 ft), 1.82 m (6 ft), and 2.43 m (8 ft) depths are presented in Fig. 10. The change of suction refer to the change from the initial suction value prior to rainfall. The figure shows various rainfall durations with a 2-year return period (rainfall intensity of 0.0012 mm/s [0.167 in./h]) and a 10-year return period.

Fig. 10. Variation of change in suction near the crest of the slope (a) at 0.91 m (3 ft) depth for 2 yr return period rain (b) at 1.82 m (6 ft) depth for 2 yr return period rain, (c) at 2.43 m (8 ft) depth for 2 yr return period rain (d) Suction at 0.91 m (3 ft) depth for 10 yr return period rain, (e) Suction at 1.82 m (6 ft) depth for 10 yr return period rain (f) Suction at 2.43 m (8 ft) depth for 10 yr return period rain.
The suction was observed to drop significantly with higher intensity and longer duration of rainfall. Moreover, the change in suction was more significant at depths of 0.91 m (3 ft) and 1.82 m (6 ft) when compared to the higher depth of 2.43 m (8 ft).

The drop of suction was instantaneous at the depth of 1.82 m (6 ft) for different rainfall intensities and durations, as depicted in Fig. 10(b) and 10(e). In contrast, the change in suction took several days to weeks post-rainfall to achieve a steady value at 2.4 m (8 ft) depth. The constant value of suction at 2.43 m (8 ft) depth indicated that the percolated water could not drain out from the slope due to the very low permeability of the highly plastic clay soil. The moist zone at the deeper depth agreed with results obtained from ERT, which indicated an active zone of 2.13 m (7 ft).

The variations of change in suction at the crest, middle, and toe of the slope at 0.91 m (3 ft), 1.82 m (6 ft) and 2.43 m (8 ft) depths are presented in Fig. 11. The variations were due to similar rainfall intensity as of Fig. 10. The maximum change in suction was observed up to 1.82 m (6 ft) depth at the crest of the slope for different rainfall intensities and durations. Furthermore, the change in suction was more significant at lower depths due to the longer period of high rainfall intensity. The change in suction was not substantial at the middle or toe of the slope.

Hossain et al. (2016) instrumented a highway slope constructed over highly plastic clay soil in Dallas, Texas to investigate the effect of rainfall on the variation of moisture content and matric suction near the surficial soil using several sensors installed at 1.2 m (3.94 ft) depth. The study presented the moisture and matric suction variation for a rainfall event with a total rainfall of 54.2 mm (2.13 in.). The moisture content of the slope increased from 4% to 25% after the first hour of rainfall. Additionally, the matric suction value at 1.2 m (3.94 ft) depth of the crest dropped from 125 kPa to 15 kPa (drop in suction = 110 kPa) in the second hour of rainfall. The study also showed that the matric suction value increased to 60 kPa, 2 weeks after the rainfall event.

The FEM analysis results indicated that the matric suction drop was 140 kPa with a total rainfall volume of 50.1 mm (2 in.) at 0.91 m (3 ft)

Fig. 11. Change in suction at the crest, middle and toe of the slope, (a) at 0.91 m (3 ft) depth after 3 h rainfall, intensity 0.0011 mm/s (b) at 1.82 m (6 ft) depth after 3 h rainfall, intensity 0.0011 mm/s (c) at 2.43 m (8 ft) depth after 3 h rain, intensity 0.0011 mm/s (d) at 0.91 m (3 ft) depth after 24 h rain, intensity 0.0022 mm/s (e) at 1.82 m (6 ft) depth after 24 h rain, intensity 0.0022 mm/s (f) at 2.43 m (8 ft) depth after 24 h rain, intensity 0.0022 mm/s.
depth. In addition, the change matric suction value decreased to 85 kPa (Fig. 10[a]), after two weeks since the rainfall event which indicates an increase of suction of 55 kPa which closely resembles the field data. Hence, the flow analysis results using finite element analysis were supported by the field investigation results presented by Hossain et al. (2016).

The flow pattern through the topsoil was important to the investigation of the failure. The slope failed in October 2011, after a long drought condition followed by a rainfall event (Fig. 6). The existence of shrinkage cracks allowed for easy rainwater intrusion at the slope crest in the top few meters immediately after rainfall. The low permeability of the highly plastic clay likely prevented downward movement of the water, creating a perched water zone near the crest of the slope. Hence, the slope stability analysis of the slope was extended to consider a perched water zone near the crest of the slope.

6. Slope stability analysis with perched water condition

The slope stability analysis was conducted using the soil model presented in Fig. 7. In addition, a perched water zone was applied in the crest (top 1/3 of the slope face) to evaluate the effect of rainfall on the safety of the slope in the presence of desiccation cracks. The depth of the perched water zone was 1.82 m (6 ft) near the crest. The perched water condition was defined to apply the similar pore water pressure inside the slope, as observed in the flow analysis described in the previous section. The soil model with the perched water zone is presented in Fig. 12. The phi-c reduction analysis was conducted, and results indicated that the factor of safety of the slope reduced to 1.05 when considering the perched water condition. This reduced factor of safety was very close to failure. The failure plane of the slope is presented in Fig. 12(b).

The slope failure occurred at the crest. That is, the failure plane of the FEM model (Fig. 12) matched the failure plane in the field (Fig. 1).

Fig. 12. (a) Initial soil model, including the perched water zone, (b) Slope stability analysis with fully-softened strength and perched water zone at crest, FS = 1.05 for the perched water zone at crest.
Therefore, the slope failure took place due to the combined action of the perched water zone at the crest and the fully-softerned condition resulting from seasonal wet-dry cycles. After a prolonged summer, rainwater intrusion via desiccation cracks formed a perched water zone at the top soil. The combination of the desiccation crack and the perched water zone in the top few feet initiated the shallow slope failure.

7. Conclusions

Shallow slope failure occurred on a highway slope located along SH 183 in Fort Worth, Texas, resulting in cracks on the shoulder near the bridge abutment. A site investigation was conducted to determine the causes of the shallow slope failure. The site investigation included soil testing, borings, and geophysical testing using electrical resistivity toigraphy. According to site investigation results, the slope was constructed using low-to-highly plastic clay susceptible to shrinkage and swelling behavior. Weather data revealed that the slope failed after a prolonged summer with few rainfall events. During the dry summer, the top soil became desiccated and formed cracks which acted as a pas-sageway for water into the slope. Both the peak and fully-softerned shear strengths of the slope were determined, and a slope stability analysis was conducted. A high factor of safety (1.46) was observed when assuming a fully-softerned shear strength at the top 2.13 m (7 ft) of the slope.

A flow analysis was then conducted by considering a high permeability along the vertical direction to simulate the effect of the desiccation crack. The flow analysis indicated that the crest of the slope became saturated at a shallow depth within a few days after rainfall. An additional slope stability analysis was conducted by considering a perched water condition at the crest. As a result, the factor of safety of the slope reduced to 1.05, and the failure plane matched that of the field condition.

Residual soils are products of chemical weathering, and their characteristics are dependent upon environmental factors of climate, parent material, topography, drainage, and age (Townsend, 1985). Slopes constructed over high plastic clay reach it’s fully soften condition due to seasonal moisture change and associated swelling - shrinkage. The flow analysis results presented in the previous section indicated that rainfall water infiltrated the desiccated cracks of highly plastic clay soil and saturated the top 2 m of the slope. As a result, the factor of the safety of the slope was reduced and caused the shallow slope failure. Findings from the current study were supported by studies conducted by Rahardjo et al. (2001) and Rahimi et al. (2010) and demonstrated that rainfall triggered shallow slope failure after the highly plastic clay soil reached its fully-softerned shear strength.

Shallow slope failure is typical and recurring in most of the highway slopes in North Texas due to the existence of the high PI clay soil. The failures usually take place after the first few prolonged showers of the summer, when desiccation cracks are still present. The current study revealed that existing desiccation cracks act as a preferential flow path for rainwater intrusion following the dry season. A perched water zone near the crest of the slope develops within a day of the rainfall because low permeability of the soil prevents the downward movement of the intruded rains. Consequently, the combination of fully-softerned shear strength and excess pore water pressure caused by the formation of a perched water condition in the upper portion of a slope results in shallow slope failure a few years after its construction.

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