Factors Influencing Crack-Induced Tensile Strength of Compacted Soil
Tae-Hyung Kim¹; Tae-Hoon Kim²; Gi-Chun Kang³; and Louis Ge, M.ASCE⁴

Abstract: Mode I (tensile) fracture is the most commonly observed failure in geosstructures resulting from cracks in soil. Direct or indirect tensile tests have been used to evaluate the tensile strength of geomaterials. In this paper, the unconfined penetration device and experimental procedure were modified to reduce measurement errors. It was then used to determine the tensile strength of compacted soil. Factors influencing the tensile strength of the compacted soil, including the plasticity index, rate of loading, and size of specimen were discussed in detail, as well as its reliability. Experimental results indicated that the modified, unconfined penetration technique is sufficiently reliable and operator-friendly for determining the tensile strength of compacted soil. DOI: 10.1061/(ASCE)MT.1943-5533.0000380. © 2012 American Society of Civil Engineers.

CE Database subject headings: Cracking; Tensile strength; Compacted soils; Penetration tests; Geomaterials.

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Introduction

Compared to the compressive or shear strength of soil, its tensile strength is generally assumed to be zero, or insignificant, in geotechnical engineering practice because of its relatively small value and lack of a satisfying laboratory technique. The tensile strength of soil is, however, an important parameter in the design of geosystems, where tensile cracks contribute to progressive erosion or landslides in excavation, slopes, dams, highway embankments, river banks, hydraulic barriers, and other earth structures. In particular, the tensile strength in natural or manmade geosstructures containing a certain amount of fine-grained soil is far more important because fine-grained soil is very sensitive to environmental changes such as moisture, temperature, and imparted compaction energy (Fang 1997). While most geotechnical engineers are familiar with classical shear strength concepts, they are not familiar with fracture mechanics analysis, including evaluation of the Mode I (tensile), II (shearing), and III (tearing) fracture behaviors that are the underlying mechanisms of most apparent tensile failure phenomena or processes (e.g., Lacazette 2000; Anderson 2004). Because many failures in geotechnical engineering are clearly dominated by tensile fracture, most engineers tend to address them using strength theories, which often arrive at different results and overall failure mechanisms.

In this study, the unconfined penetration device (Fang and Fernandez 1981) was modified to reduce measurement errors by maintaining good alignment between loading disks and the soil specimen. It was then used to determine the tensile strengths of compacted granite-bentonite and sand-bentonite mixtures. Discussions were focused on the factors influencing the tensile strength of the compacted soil, including plasticity index, rate of loading, and size of specimen. The reliability of the modified, unconfined penetration test was also presented by comparing the results from the split-tensile test on the same tested materials.

Background

In laboratory testing on the tensile strength of soil, Leonards and Narain (1963) developed a technique to measure the tensile strength of cohesive soil using a simple flexural test involving a clay-beam to predict the cracking behavior of earth dams. Conlon (1966) conducted unconfined tension experiments on a soft clayey silt (an estuarine sediment) that is similar to that used in the conventional triaxial apparatus, except that its central portion was necked down to create failure in this zone and reduce the necking effect of the specimen. Spencer (1968) and Suklje (1969) assessed the effect of tensile strength in slope stability analysis, particularly in cohesive slopes, where creep and critical stress states with tensile stress regions appear in the upper parts of the slopes. Bishop and Garga (1969) used confining pressure to produce tensile stresses instead of pulling the ends of the specimen at the platens. Bofinger (1970) employed another technique using a prismatic specimen. The ends of the specimen were bonded to steel plates with quick setting polyester resin, and the tensile force was applied through a cap, which incorporated a spherical seating to reduce the effect of end rotation. George (1970) applied the brittle fracture theory to evaluate crack growth and its effects on stabilized soil-cement mixtures. Fang and Fernandez (1981) showed the importance of cracking behavior on the stability of slopes, earth dams, highways, and airfield pavements.
Perkins (1991) developed a direct tension apparatus to measure the tensile strength of a granular material. This apparatus was an improvement over the typical disadvantages of direct tensile apparatus: stress concentration, misalignment, and eccentricity. Mikulitsch and Gudehus (1995) designed and constructed a direct tension device similar to that developed by Perkins (1991). Pairs of beveled walls kept the sample in place. One pair was fixed, whereas the other rested on a ball-bearing and was pulled apart. The forces were increased by slowly filling water into a bucket hanging by a thread. The horizontal surface and two lateral slits were covered by plates to prevent evaporation. However, most of them have the disadvantage that the displacement or strain measurements are not reliable, and the uniformity of the stress distribution within the specimen is also questionable (Thompson 1965; Breen and Stephens 1966; Kennedy and Hudson 1968). For most direct tensile tests, the specimen is maintained intact or confined by split rings clamped around the ends of the specimen and loading head, which may cause stress concentration and friction at the ends of the specimen.

### Unconfined Penetration Test

Fang and Fernandez (1981) developed the unconfined penetration test by modifying the double punch test proposed by Chen (1970). It measures the tensile strength of compacted soil using specimens prepared in a California Bearing Ratio (CBR) mold and related equipment. Fig. 1 shows an ideal failure mechanism developed within a cylindrical specimen from an unconfined penetration test. It consists of many simple tension cracks oriented along the radial direction and two cone-shaped rupture surfaces directly beneath the disks. The conical surfaces move toward each other as rigid bodies and displace the surrounding material radially. The relative velocity vector, $\delta v$, at each point along the cone surface is inclined at an angle, $\phi$, to the surface. The compatible velocity relationship is also shown in Fig. 1. It is relatively straightforward to calculate the surface of discontinuity. The rate of dissipation of energy is found by multiplying the area of each discontinuity surface by $\sigma_t$ times the separation velocity, $2\Delta v$, across the surface for a simple “tensile” crack or $q_u(1-\sin \phi)/2$ times the relative velocity, $\delta v$, across the cone-shaped rupture surface for simple “shearing.” Equating the external rate of work to the total rate of internal dissipation yields the value of the upper bound (limit plasticity) for the applied load, $P_{\max}$

$$P_{\max} = \frac{1 - \sin \phi}{\sin \alpha \cos(\alpha + \phi)} \frac{q_u}{2} + \tan(\alpha + \phi) \left( \frac{bH}{a^2} - \cot \alpha \right) \sigma_t$$

where $\alpha$ = unknown angle of the cone; $\alpha$ = radius of the disk; $q_u$ = unconfined compressive strength; $\phi$ = undrained friction angle of soil; $b$ and $H$ = specimen radius and height, respectively (Fig. 1).

The upper bound solution has a minimum value, when $\alpha$ satisfies the condition $\partial P/\partial \alpha = 0$, Eq. (1) can be reduced to

$$\sigma_t = \frac{P_{\max}}{\pi(MbH - a^2)}$$

where $M = \tan(2\alpha + \phi)$.

The value of $M$ depends not only on the angle of friction, but also on the compression-tensile strength ratio, and the angle of the cone, depending on the size of the loading disk (punch). Fang and Fernandez (1981) considered three specimen sizes obtained from Proctor and CBR molds. The recommended values of $M$ for practical use are listed in Table 1.

Although the unconfined penetration test has effectively reduced boundary effects by using a smaller size of loading disk, it is still difficult to maintain a good alignment between the loading disks and soil specimen. The two disks should be centered on both the top and bottom parallel surfaces of the cylindrical soil specimen. Therefore, a modification to the device was made in the current study. Both disks are located in the same axis, as shown in Fig. 2, because they are installed along the axis of the loading arm, which is perfectly aligned vertically. In addition, the base plate has been furnished with engraved concentric circles of 0.5-cm spacing, which facilitates the placement of the specimen at the center. The engraved base plate also reduces slippage between specimen and end-plates by enhancing interface friction.

The modified, unconfined penetration test is also designed to induce failure on the weakest plane of the specimen, in contrast to the split tensile test and other tensile strength test methods in which the tensile strength is determined across a predetermined failure plane.

### Testing Program

#### Materials

Two types of soil samples were prepared. The first was granite-bentonite mixed soil. The granitic soil, collected from Kyungido Pochaeon in Korea, comprises weathered materials that pass through the #10 sieve. The granitic soil is typically classified as SM or SC based on the unified soil classification system and exhibits mechanical behavior between sandy soils and clay soils (Nishida 1970). The bentonite has a loose bulk density of 0.75–0.90 g/cm³.

<table>
<thead>
<tr>
<th>Size of specimen</th>
<th>$M$ value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Harvard miniature compaction mold (3.3 × 7.2 cm)</td>
<td>~1.05</td>
</tr>
<tr>
<td>Proctor mold (10.2 × 11.3 cm)</td>
<td>1.0</td>
</tr>
<tr>
<td>CBR mold (15.2 × 17.8 cm)</td>
<td>0.8</td>
</tr>
</tbody>
</table>

Note: Conditions: • Disk-specimen ratio: 0.2–0.3; • Diameter-to-height of specimen ratio: 0.46–1.0; • Rate of loading: ASTM recommendation for axial strain at a rate of 0.5–2 percent of height per minute.

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Fig. 1. Unconfined penetration test: (a) cross-section; (b) velocity relation (reprinted with permission from Fang and Fernandez 1981, © ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428)
and swelling properties of 24–30 g 24 hr. The bentonite has a montmorillonite content of 400 mg/500 mg (i.e., the percentage of montmorillonite in the bentomite is 80%), and a viscosity of 35 sec/500 cc. The properties of the prepared soil samples are shown in Table 2. The gradation curves of the three granite-bentonite soil samples (GB-1, GB-2, and GB-3) are shown in Fig. 3(a) as nonplastic, or with a plasticity index of 15 or 30, respectively.

The second type of soil samples (SB-1, SB-2, and SB-3) were prepared by mixing Jumonjin sand with bentonite. The Jumonjin sand is a fine-grained quartz sand of uniform gradation with a mean particle size of 0.445 mm and a specific gravity of 2.65. The same bentonite used for preparing the granite-bentonite samples was used. The characteristics of the prepared sand-bentonite soil samples are shown in Table 2. The SB-3 sample contains more bentonite than the SB-2 and SB-1 samples, as shown in their gradation curves in Fig. 3(b) as nonplastic, or with a plasticity index of 13.6 or 24.4, respectively.

Each soil specimen was prepared with three lifts in a Proctor compaction mold (H = 127 mm and b = 50 mm). A 5.5-lb hammer and 12-in drop at the optimum moisture content, and 25 blows per layer were used (the method A described in ASTM D 698). The optimum moisture content of each soil sample was obtained and its results are shown in Table 2.

**Procedures**

After each specimen was prepared, its weight was measured to calculate the unit weight. Two disks were installed on the top and bottom in the loading axis. The specimen was then placed on the base plate and was lined up with the disks (Fig. 2). This alignment between two disks and the soil specimen is very important, as mentioned in the previous section. Then, the bottom disk was raised by moving the axis of the loading arm until the bottom surface of the specimen was in contact with the bottom disk. A load was steadily applied at the selected rate (2%/min, 1%/min, 0.5%/min, or 0.1%/min). The force was applied until failure occurred in the specimen. The force and displacement during testing were recorded through the data logger. The recorded peak load was used for calculating the tensile strength of the mixed soil specimen by Eq. (2). After the test, the fractured specimen was collected quickly and weighed both before and after oven drying to determine its water content.

**Results and Discussions**

To examine the effect of loading disk size on the tensile strength of mixed soil, four different diameters (12.7, 25.4, 38.1, and 50.8 mm) were used. In addition, to assess the effect of loading rate on the tensile strength, four different loading rates (2%/min, 1%/min, 0.5%/min, and 0.1%/min) were applied.

**Effect of the Size of Loading Disk**

Figs. 4 and 5 show variations of tensile strength, calculated from Eq. (2), to different disk sizes. The tensile strength was found to increase with an increase in loading disk size. This indicates that the

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Samples</th>
<th>( \gamma'_{\text{max}} ) (kN/m(^3))</th>
<th>OMC (%)</th>
<th>Compaction</th>
<th>Atterberg limit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Liquid limit</td>
<td>Plastic limit</td>
</tr>
<tr>
<td>Granite-bentonite mixed soils</td>
<td>GB-1</td>
<td>18.0</td>
<td>13.3</td>
<td>N.P</td>
<td>40.7</td>
</tr>
<tr>
<td></td>
<td>GB-2</td>
<td>17.6</td>
<td>14.5</td>
<td>36.3</td>
<td>21.3</td>
</tr>
<tr>
<td></td>
<td>GB-3</td>
<td>17.5</td>
<td>14.8</td>
<td>49.2</td>
<td>19.2</td>
</tr>
<tr>
<td>Sand-bentonite mixed soils</td>
<td>SB-1</td>
<td>15.6</td>
<td>17.1</td>
<td>N.P</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>SB-2</td>
<td>15.8</td>
<td>17.6</td>
<td>30.5</td>
<td>16.9</td>
</tr>
<tr>
<td></td>
<td>SB-3</td>
<td>15.9</td>
<td>18.0</td>
<td>47.3</td>
<td>22.9</td>
</tr>
</tbody>
</table>
disk size is also an important factor to consider when measuring a reliable tensile strength of soil in the test. As shown in Figs. 6 and 7, the peak value from the test results using the 12.7-mm loading disk was difficult to determine. On the other hand, the failure pattern from the tests using a loading disk size of 50.8 mm was not as clearly developed as the one indicated in the original theoretical framework (Fang and Chen 1972). Although the loading disk size of 38.1 mm used in the study yields a larger disk-to-specimen diameter ratio than the range of 0.2–0.3 proposed by Fang and Chen (1972), the specimen’s failure pattern did follow the theoretical observations made by Fang and Chen (1972), which suggests that the modified unconfined penetration test for compacted soils can be conducted at the prescribed ASTM loading rate for the unconfined compression test.

**Fig. 3.** Grain size distribution curves: (a) granite-bentonite mixtures; (b) sand-bentonite mixtures

**Fig. 4.** Variation of tensile strength to a disk diameter of granite-bentonite mixtures

**Fig. 5.** Variation of tensile strength to a disk diameter of sand-bentonite mixtures

one. From the above observation and discussion, both loading disk sizes of 25.4 and 38.1 mm were acceptable to use.

**Axial Force-Displacement Relation**

Fig. 6 shows the axial force on the compacted granite-bentonite mixture under the loading rate of 1.0%/min. Generally, the peak of each force-displacement curve shows that the larger the loading disk used, the lower the corresponding displacement level. For disk diameters of 25.4, 38.1, and 50.8 mm, the observed peak force appeared between 3- and 7-mm displacement. However, for a disk diameter of 12.7 mm, the axial peak force could not be clearly defined. This difference is related to the shearing mechanism shown in Fig. 1. In the case of a large-size diameter disk, more energy is required to induce specimen failure because of the relative sizes of the disk and the specimen. Fig. 7 shows the test results for compacted specimens of sand-bentonite mixture subjected to a loading rate of 1.0%/min. The slope of each force-displacement is higher, and peak values are reached faster than in Fig. 6. However, similar force-displacement behaviors for relationships with loading disk size were observed, as in Fig. 6.

**Effect of Plasticity Index**

As shown in Figs. 6 and 7, the peak axial force for the same size disk increases with an increase in the plasticity index (PI) for both granite-bentonite and sand-bentonite mixtures. The peak axial force (P) for a disk diameter of 50.8 mm increased from 0.13 to 0.44 kN and from 0.15 to 0.73 kN when the plasticity index increased from nonplastic to 30, and from nonplastic to 24.4, respectively. The added bentonite induces an additional cohesion between particles, and the more added bentonite, the higher the plasticity index of the soil-bentonite mixture. Thus, the bentonite leads to an increase in axial force, and the tensile strength increases when the plasticity index (PI) increases (Figs. 4 and 5).

**Effect of Loading Rate**

In general, the unconfined compressive strength of compacted soil tends to increase when the loading rate is increased. In this study, the tensile strength increases slightly when the loading rate increases and no definite trends in tensile strength variation can be observed (Figs. 8 and 9). This result is quite consistent with observations made by Fang and Chen (1972), which suggests that the modified unconfined penetration test for compacted soils can be conducted at the prescribed ASTM loading rate for the unconfined compression test.
Fig. 6. Axial force-displacement curves of granite-bentonite mixtures at 1.0%/min loading rate: (a) GB-1 (non-plastic); (b) GB-2 (PI = 15); (c) GB-3 (PI = 30)

Fig. 7. Axial force-displacement curves of sand-bentonite mixtures at 1.0%/min loading rate: (a) SB-1 (non-plastic); (b) SB-2 (PI = 13.6); (c) SB-3 (PI = 24.4)

Fig. 8. Loading rate effect on tensile strength of granite-bentonite mixtures: (a) GB-1; (b) GB-2; (c) GB-3

Fig. 9. Loading rate effect on tensile strength of sand-bentonite mixtures: (a) SB-1; (b) SB-2; (c) SB-3
Reliability of the Modified Unconfined Penetration Test

To examine the validity of the modified unconfined penetration tests, split tensile tests were also conducted on the same specimens. The reason for selecting the split tensile tests for comparison is that the split tensile tests are widely used to determine the tensile strength of materials such as concrete, cemented soil, and compacted soil.

Fig. 10 shows the tensile strengths of the granite-bentonite and sand-bentonite soil obtained in this study in comparison with a study reported by Fang and Chen (1972). As shown in Fig. 10, the tensile strength obtained from the modified unconfined penetration tests shows little difference compared with the split tensile test. Because the test was designed to induce failure on the weakest plane of the specimen, the tensile strength obtained from the modified, unconfined penetration tests gives a slightly lower value than that from the split tensile test, which measures the tensile strength across a predetermined failure plane. In general, good agreement between two values is observed.

Conclusions

A modification was made to the unconfined penetration test to measure the tensile strength of compacted soil mixtures. This study demonstrated the use of a simple test device and established procedures for measuring the tensile strength of compacted soil. In addition, the test results can be applied to geotechnical engineering practice for understanding and quantifying cracking behavior of compacted soil, and for the design, construction, and maintenance of geosstructures.

The modified unconfined penetration device effectively reduced the misalignment between the loading disks and the soil specimen. The bottom and top disks were designed to align perfectly with the vertical axis. The base plate has been furnished with engraved concentric circles of 0.5-cm spacing, which facilitates the placement of the specimen at the center.

The modified unconfined penetration test was also designed to induce failure on the weakest plane within the specimen, measuring the true tensile strength. Both loading disks with diameters of 25.4 and 38.1 mm are suitable to evaluate tensile strength in this study. The tensile strength increased with the increase in the disk size, as well as with the plasticity index (PI). Lastly, the tensile strength also showed a tendency to increase with the increase in loading rate.

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