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Apparatus for Constant Rate-of-Strain Consolidation of Slurry Mixed Soils

ABSTRACT: A constant rate-of-strain (CRS) consolidation apparatus was developed to allow testing of slurry mixed soils prepared within 102-mm (4-in) inner diameter acrylic columns with varying lengths using a laboratory mixing apparatus. The laboratory mixing apparatus was previously developed to mimic the mixing process associated with the use of slurries containing bentonite and granular zero-valent iron (ZVI) for in situ remediation at sites contaminated with chlorinated solvents. Example consolidation results are provided for specimens of dry sand mixed with bentonite-ZVI slurry with lengths of 305 mm (12 in), 457 mm (18 in), or 610 mm (24 in). After mixing, all of the specimens except for the 610-mm specimen were consolidated directly within the same columns to minimize sampling disturbance. The 610-mm specimen was sectioned into thirds before consolidation testing to evaluate effects of vertical heterogeneity. As expected, all specimens were highly compressible, with maximum (large) strains ranging from 10.7% to 48.0%. For specimens prepared and tested within the same columns, the effects of differences in column length and strain rate were minor. The CRS testing apparatus offered a convenient, rapid (< 3 day), and economical approach for evaluating the consolidation behavior of the bentonite-ZVI slurry mixed sand.

KEYWORDS: bentonite, consolidation, slurry, soil mixing, zero-valent iron

Introduction

The zero-valent iron (ZVI)-clay technology refers to in situ remediation of subsurface source zones contaminated with chlorinated solvents (e.g., tetrachloroethylene, trichloroethylene, carbon tetrachloride) by injecting and mixing granular ZVI suspended in water-based clay slurry directly into the contaminated soil. The clay, typically either kaolin or bentonite, serves to suspend the granular ZVI in the slurry, and the ZVI serves as the reactive media that reductively dechlorinates (i.e., treats) the chlorinated solvents (Gillham and O’Hannesin 1994, Wadley et al. 2005).

Injection of the water-based clay slurry results in surface heaving and an increase in soil compressibility and decrease in soil shear strength. As a result, redevelopment of sites treated using this technology may be hindered by the post-mixing condition of the treated zone. In this regard, knowledge of the consolidation behavior of the mixed soil is crucial to understanding the potential for future development of the treated site. However, the unique composition and properties of such soil-ZVI mixtures present several issues that typically are not encountered in routine testing of more uniform foundation soils. For example, the injection of ZVI-clay slurry during mixing converts what typically is essentially an undisturbed, coarse-grained parent soil of minimal compressibility (e.g., contaminated sand and/or gravel) into a remolded, highly compressible mixture. In addition, the composition and properties of the mixed materials vary significantly with depth due to changes in overburden stress and the high level of disturbance caused by the mixing auger.

A laboratory mixing apparatus was previously developed to mimic the mixing process associated with injection of ZVI-clay slurry into a coarse-grained parent soil. The mixing apparatus is described in detail in Castelbaum et al. (2011). As a result of the non-uniform property distributions of the mixtures, relatively large consolidation specimens were preferred in order to encompass the behavior of as much of the mixed soil column as possible. As a result, an apparatus was developed to perform constant rate-of-strain (CRS) consolidation testing of entire soil mixed specimens, which could be transferred directly from the mixing machine to the load frame without removing the specimens from the mixing columns, thereby minimizing disturbance. The CRS method also allowed for shorter test durations than standard, incremental load tests, as well as easier, automated data collection (e.g., Wissa et al. 1971, Armour and Drnevich 1986, Olson 1986, Sheahan and Watters 1997).

A standard has been developed for CRS testing (ASTM D4186-06), which is a method that has been used for consolidation testing and described in the literature since 1959 (e.g., Hamilton and Crawford 1959, Crawford 1964, Smith and Wahls 1969, Wissa et al. 1971, Deen et al. 1978, Gorman et al. 1978, Armour and Drnevich 1986). Generally, specimens are contained within a consolidation cell and subjected to a constant rate of displacement. The rate of displacement is selected to limit excess pore pressures generated during the test. Appropriate criteria for selecting a displacement rate (or strain rate for a particular specimen) have been discussed extensively in the literature (e.g., Smith and Wahls 1969, Wissa et al. 1971, Armour and Drnevich 1986).

Standard triaxial load frames often are used to apply a constant rate of displacement. Therefore, specimen size is usually limited...
Materials and Experimental Methods

Materials

The two constituent soils used in the study were fine, Ottawa sand (Grade F-58 foundry sand, U.S. Silica Company, Ottawa, IL), and powdered, polymer modified, air-float bentonite (Hydrogel, Wyoming), respectively. The sand classified as poorly graded sand (SP) and the bentonite classified as high plasticity clay (CH) based on the Unified Soil Classification System (ASTM D2487). The granular ZVI used in this study was obtained from Peerless Metal Powders & Abrasive Co., Inc., Detroit, MI. The measured specific gravity of solids for the granular ZVI filings was 6.67, and the particle sizes ranged from 0.15 to 0.30 mm.

Deionized water (DIW), with an electrical conductivity (EC) less than 1.0 mS/m at 25°C was used during specimen preparation. The DIW was de-aired for saturation of the pore-water pressure line and preparation of saturated sand specimens.

Preparation of Injection Slurry

The injection slurry (i.e., bentonite plus ZVI suspended in water) was prepared by adding the appropriate mass of granular ZVI to a 5% bentonite-water slurry. The bentonite-water slurry was prepared by adding 5 g of bentonite (dry mass basis) per 95 g of water. The slurry was mixed with high-speed colloidal mixers and allowed to hydrate for a minimum of 16 h before adding the ZVI. The viscosity of the bentonite-water slurry measured using a Marsh Funnel ranged from 40 to 45 s, which is typical of bentonite slurries used in soil-bentonite vertical cutoff walls (Evans 1993). Further details on preparation of the injection slurry are provided by Castelbaum (2007), Sample (2007), and Castelbaum et al. (2011).

The dry mass of sand in the column was used as the basis for determining the minimum required amount of the bentonite-ZVI slurry to be injected into the column. The required mass of hydrated bentonite slurry was determined from the percentage of bentonite in the slurry and the target bentonite content of the post-mixed column. The required mass of ZVI was determined similarly by the target iron content of the post-mixed column.

Specimen Preparation in Test Columns

Before mixing, specimens comprised of fine sand were prepared in clear, acrylic columns. The bottoms of the columns were 6.35-mm (0.25-in)-thick, 114.3-mm (4.5-in) inner-diameter rings that inserted into an acrylic base. A small reservoir (25.4-mm diameter, 6.35-mm height) and pore-water pressure line were located under the center of the column (within the acrylic base). An inlet for a 6.35-mm (0.25-in)-thick O-ring under the walls of the column allows for a watertight seal at the connection of the base and the column. The top ends of the columns were cut to provide a 6.35-mm (0.25-in)-thick, 102-mm (4-in) inner-diameter ring that inserts into an acrylic top plate to secure the column to four 650-mm (26-in) threaded rods, which also attach to the base plate.

Prior to placement of the sand, the pore-water pressure line in the base of the column was saturated with de-aired water, and a 6.35-mm (0.25-in)-thick, 102-mm (4-in)-diameter porous stone was placed on the base and covered with two sheets of saturated No. 42 Whatman filter paper (Whatman Inc., Florham Park, NJ). The sand was placed using air pluviation (i.e., raining of sand through air), as described by Zlatovic and Ishihara (1997) and Okamoto and Fityus (2006), via a funnel and tamped in lifts to provide a uniform pre-mix density of approximately 1590 kg/m³ (99 lb/ft³). The pre-mix specimen thicknesses (i.e., heights) for the 305-mm (12-in), 457-mm (18-in), and 610-mm (24-in)-tall columns were 200 mm (7.9 in), 250 mm (9.8 in), and 310 mm (12 in), respectively.

Mixing and Consolidation Equipment

Specimens of bentonite-ZVI slurry mixed sand were prepared using the laboratory mixing apparatus. Sand-filled columns were attached to the base plate of the mixing machine, as shown in Fig. 1. The laboratory mixing machine mimics field applications, using a hollow-stem auger to inject slurry as the auger rotates and moves vertically through the soil (Shackelford et al. 2005). The bentonite-ZVI slurry was stored in a stainless steel reservoir and delivered via piston displacement. All specimens were mixed with
two soil-mixing passes. After the auger was extracted from the mixed specimen upon completion of the second mixing pass, the column was removed from the mixing machine and the final thickness and weight of the mixed specimen were recorded. The post-mix specimen thicknesses typically were 240 mm (9.4 in), 310 mm (12 in), and 395 mm (16 in) for columns with heights of 305 mm (12 in), 457 mm (18 in), and 610 mm (24 in), respectively. After mixing, the specimens were covered, sealed with plastic wrap (Glad Products Company, Oakland, CA), and allowed to undergo self-weight consolidation for 1 day before consolidation testing, as illustrated in Figs. 2 and 3. The pore-water pressure line at the base of the column was connected to the pore-pressure transducer (ELE International model 27-1625, 1000 kPa capacity, Loveland, CO). Three No. 42 Whatman filter papers were placed carefully on top of the mixed specimen, followed by a saturated porous stone (trimmed to a diameter slightly less than 102 mm), three more No. 42 filter papers, and the top platen (Fig. 2(b)). The purpose of the stone on top of the specimen was to evenly distribute drainage over the bottom face of the platen. Several 3.85-mm (0.15-in)-diameter holes were drilled in the platen to provide drainage (Fig. 2(c)). Two O-rings fit into insets around the perimeter of the platen to prevent sidewall drainage, and the O-rings were generously coated with vacuum grease to reduce wall friction. A small spherical groove was machined into the top of the platen to accommodate the tip of an S-type load cell (ELE International model 27-1583, Loveland, CO), which was secured to the cross arm of the load frame (Geotest model S5760, Evanston, IL). The capacities of the load cell and load frame were 8.9 kN (2000 lb) and 10 kN (2248 lb), respectively, and the maximum pedestal travel was 76 mm (3 in). A linear position transducer, or LPT (Geotac, Houston, TX), with a measurement range of 150 mm (6 in) was attached to the top plate and used to measure the relative movement of the top platen to verify the deformation rate during loading.

Testing Conditions

Selection of Strain Rate—The maximum value for the strain rate, SR, to be used in CRS testing according to ASTM D4186-89 is the value of SR that limits the ratio of the excess pore-water pressure, \( u_h \), to the applied vertical total stress, \( \sigma_v \), to values from less than 3% to 30% (note that the testing reported in this study was conducted prior to promulgation of ASTM D4186-06, which revised the upper limit on \( u_h \) to only 15% of \( \sigma_v \)). Thus, expected values of \( u_h/\sigma_v \), also known as the pore-water pressure ratio, PPR, generally are determined based on the typical coefficient of consolidation, \( c_v \), for the type of soil to be tested. However, because \( c_v \) values of bentonite-ZVI slurry mixed sand were...
unknown, SRs for the testing program initially were selected based on those from the literature, which according to Head (1992) typically range from \( 1.67 \times 10^{-6} \) s\(^{-1}\) (i.e., 0.0001 \% min\(^{-1}\)) to \( 6.67 \times 10^{-6} \) s\(^{-1}\) (i.e., 0.04 \% min\(^{-1}\)).

On the basis of the reported ranges in \( SR \) and recommended values for \( PPR \), SR values from \( 6.33 \times 10^{-7} \) s\(^{-1}\) to \( 5.00 \times 10^{-6} \) s\(^{-1}\) were chosen for the specimens of bentonite-ZVI slurry mixed sand tested in this study. In addition, the deformation rate limits (minimum \( 1.67 \times 10^{-5} \) mm s\(^{-1}\)) of the load frame prevented the use of SRs lower than \( 2.08 \times 10^{-7} \) s\(^{-1}\), \( 6.94 \times 10^{-8} \) s\(^{-1}\), and \( 5.38 \times 10^{-8} \) s\(^{-1}\) for specimens contained within testing columns with lengths of 152 mm (6 in), 305 mm (12 in), and 457 mm (18 in), respectively. Strain rates faster than \( 2.50 \times 10^{-6} \) s\(^{-1}\) for specimens contained within longer, 305 mm (12 in) and 457 mm (18 in) test columns resulted in \( PPR \) values > 30 \% and, therefore, were not used.

**Evaluation of Sidewall Friction**

Two possible sources of sidewall friction were considered, viz., (1) friction between the O-rings in the top platen and the inner wall of the column, and (2) friction between the mixed specimens and the inner column wall. With respect to (1), several steps were taken to account for friction between the watertight top platen and the inner column wall, including (a) greasing the sides of the top platen generously with vacuum grease, (b) performing several friction tests at the beginning to determine which set of conditions (e.g., strain rate, O-rings) would provide the lowest friction, and (c) performing separate friction calibration tests conducted without specimens to generate friction correction curves for each acrylic column to allow for correcting the data for the friction component and ensure that the friction was a very small to negligible component of the measured forces. In the case of (c), the friction between the top platen O-rings and the inner column wall increased with increasing strain rate (Sample 2007).

In terms of the second source of friction, the friction between test specimens and the inner walls of test cells tends to increase as the height-to-diameter ratio of the specimens increases. Thus, given that the heights of the specimens tested in this study were greater than the diameter of the specimens, there was some concern about the influence of the friction between the slurry mixed specimens and the inner column wall during testing. In an attempt to reduce this concern, the columns were constructed from new, smooth, clean acrylic tubing (e.g., Gniel and Bouazza 2008), although the inside of the column was not coated or greased like the top platen, primarily because the grease likely would not have withstood the mixing of the specimens within the columns. Also, the friction between the top platen O-rings and the inner

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**FIG. 2**—(a) Cross-sectional schematic of CRS testing apparatus, and pictorial views showing (b) filter paper and porous stone setup for the test specimens and (c) load cell placement and drainage through the top platen.
column wall likely was substantially greater than that between the specimens and the inner column wall. Nonetheless, this issue is difficult to quantify or avoid easily (as with most one-dimensional consolidation tests). The composition and properties of the mixed soil specimens already vary significantly with depth, and additional variations to the soil structure may be introduced by consolidating the relatively long samples within the acrylic tubing, further complicating the evaluation of friction. Therefore, the results of the study were evaluated with an understanding that such sidewall friction may have influenced the data (e.g., Wong et al. 2008).

Example Results

Example results for the slurry-mixed sand are presented in what follows. The test conditions for the mixing stage and the CRS consolidation stage are summarized in Table 1. These example tests allowed for an evaluation of (1) the ability to prepare bentonite-ZVI slurry mixed sand specimens with the mixing machine and subsequently measure representative consolidation behavior of the mixed material (as an entire column or in sections), and (2) the effects of mixed specimen thickness and the applied strain rate on measured consolidation behavior. As indicated in Table 1, specimens Nos. 1, 2, and 3 represented the top, middle, and bottom thirds, respectively, of a single mixed column of bentonite-ZVI slurry mixed sand. Specimens Nos. 4 and 5 were mixed and consolidated at the same SR within columns with lengths of 305 mm (12 in) and 457 (18 in) to facilitate an evaluation of the effect of specimen thickness (or column height) on the measured consolidation behavior. Specimen No. 6 was tested within a 457-mm (18-in)-long column, but at a lower SR than specimen No. 5 to facilitate an evaluation of the effect of SR. The mixing stage for each test was completed within a couple of hours, while the consolidation stages were completed within periods ranging from 1 to 3 days.

The example test data were analyzed in accordance with the linear and nonlinear small-strain theories published by Wissa et al. (1971) and the large-strain theory published by Lee (1981). Results from the linear and nonlinear theories were essentially identical for all the specimens, indicating that the linear theory was adequate for describing the consolidation behavior of the specimens based on small strains. Therefore, the example results presented herein are based on linear small-strain theory and large-strain theory. The potential significance of self-weight consolidation also was evaluated based on the large-strain theory presented by Umehara and Zen (1980). Self-weight effects were found to be negligible for the conditions of the tests conducted in this study. Further details on the analyses including all relevant equations can be found in the aforementioned references and in Sample (2007).

Pore-water pressure, load-cell, and linear-position transducer readings were recorded at 1-min intervals with an automated data acquisition system. Error in the load-cell readings was approximately ±1 kPa (± 0.15 psi). In addition, significant scatter in the data was observed for all specimens for vertical
effective stress, $\sigma_0'$, less than about 30 kPa (4.4 psi) to 60 kPa (8.7 psi), depending on the specimen. This scatter likely was due to very low initial pore-water pressures, instrument accuracy, and/or potential frictional effects. As a result of these considerations, only the data corresponding to $\sigma_0' > 100$ kPa (14.5 psi) were used subsequently for determining consolidation parameters, e.g., to limit the maximum error of the load cell to 1%.

### Consolidation Properties

Stress-strain curves for the six specimens described in Table 1 are shown in Fig. 4. The differences between the stress-strain curves calculated using the small- versus large-strain theories are relatively minor for all specimens except for specimen No. 1, representing the top third of a mixed column (Fig. 4(a)). Overall, the

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**TABLE 1—Example test program for mixing and consolidation evaluation of specimens of sand mixed with bentonite and zero-valent iron slurries.**

<table>
<thead>
<tr>
<th>No.</th>
<th>Mixing Column Characteristics</th>
<th>Specimen Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>No.</td>
<td>Mixing Column Length [mm (in)]</td>
<td>Bentonite Content* (%)</td>
</tr>
<tr>
<td>-----</td>
<td>-------------------------------</td>
<td>-------------------------</td>
</tr>
<tr>
<td>1</td>
<td>610 (24)</td>
<td>2.27</td>
</tr>
<tr>
<td>2</td>
<td>305 (12)</td>
<td>2.58</td>
</tr>
<tr>
<td>3</td>
<td>457 (18)</td>
<td>2.33</td>
</tr>
<tr>
<td>4</td>
<td>457 (18)</td>
<td>2.33</td>
</tr>
<tr>
<td>5</td>
<td>457 (18)</td>
<td>2.33</td>
</tr>
<tr>
<td>6</td>
<td>457 (18)</td>
<td>2.33</td>
</tr>
</tbody>
</table>

*Calculated on the basis of the amount of slurry injected into the sand column; target bentonite content in all cases was 2.30%.

FIG. 4—Stress-strain curves for CRS consolidation testing of bentonite-ZVI slurry mixed sand specimens for different testing column lengths and strain rates: (a) 152 mm, $5.00 \times 10^{-6}$ s^-1, top third; (b) 152 mm, $5.00 \times 10^{-4}$ s^-1, middle third; (c) 152 mm, $5.00 \times 10^{-6}$ s^-1, bottom third; (d) 305 mm, $2.50 \times 10^{-6}$ s^-1; (e) 457 mm, $2.50 \times 10^{-6}$ s^-1; (f) 457 mm, $6.33 \times 10^{-7}$ s^-1. [Note: SS = small-strain theory; LS = large-strain theory]
specimens were initially highly compressible, as expected, with ultimate vertical strains, \( e_{v,max} \), reaching as high as 48.0% (based on large-strain theory). The observed compressibility is consistent with the loose conditions of the specimens resulting from injection of the bentonite-ZVI slurry (see Castelbaum et al. 2011).

All of the values of the pore-pressure ratio, \( PPR = \frac{p_{w}}{\sigma_v} \) (where \( p_{w} \) is pore-water pressure, versus vertical total stress, \( \sigma_v \)), were < 30% for \( \sigma_v \geq 100 \text{kPa} \) (14.5 psi) with the exception of the two specimens representing entire columns of bentonite-ZVI slurry mixed sands tested at the highest allowable strain rate of \( 2.50 \times 10^{-4} \text{s}^{-1} \) (i.e., specimen Nos. 4 and 5). For these two specimens, \( PPR \) was > 30% until \( \sigma_v \) increased to 243 kPa (35.2 psi) and 700 kPa (102 psi), respectively. Gorman et al. (1978) recommended that pore-water pressure values be greater than 7 kPa (1 psi), to avoid excessively high, inaccurate values of the coefficient of consolidation, \( c_v \). The pore-water pressure typically was maintained above 3 kPa (0.44 psi). The high sand contents of the middle and bottom thirds of the mixed columns (specimen Nos. 2 and 3) made generation of pore-water pressures consistently > 7 kPa (1 psi) impractical. For any given specimen, the value of \( PPR \) decreased with increasing \( \sigma_v \), indicating that the SR was sufficiently slow to allow some dissipation of \( u_h \) (\( u_h \) increased at a slower rate than \( \sigma_v \)).

Values of \( c_v \) were calculated based on large-strain (Lee 1981) and small-strain theories using time intervals corresponding to 100-kPa (14.5-psi) increments in \( \sigma_v \) (e.g., 100 kPa (14.5 psi), 200 kPa (29 psi), etc.). The results for each specimen are provided in Fig. 5. To check the validity of the pore-water pressure distributions assumed by the large-strain theory, \( c_v \) values were calculated at both the drained and undrained boundaries for each specimen. The similarity in the \( c_v \) values calculated at each boundary suggests that the strain rates were sufficiently slow, such that the assumed pore-water pressure distributions in the large-strain theory were valid (e.g., see Lee 1981). For all test specimens, \( c_v \) increased with increasing effective stress regardless of the theory used to calculate \( c_v \).

As expected, the calculated \( c_v \) values for the bentonite-ZVI slurry mixed sand, which ranged from \( 1.9 \times 10^{-6} \text{m}^2/\text{s} \) (Fig. 5(a), large-strain theory) to \( 5.0 \times 10^{-6} \text{m}^2/\text{s} \) (Fig. 5(b), small-strain, linear theory), are higher than those typically reported for pure clay specimens, but lower than those typically reported for pure sand.
coefficients of volume compressibility, \( m_v \) (= \( \Delta v_v/\Delta \varepsilon_v \)), versus \( \varepsilon_v \) are shown for each specimen in Fig. 6. Unlike \( c_v \), only one value of \( m_v \) was calculated based on the large-strain theory, because the value of \( \Delta v_v \) used to determine \( m_v \) is based on the difference in Eulerian (large) strain between drained and undrained boundaries (Lee 1981). For all test specimens, \( m_v \) decreased with increasing \( \varepsilon_v \), indicating that the bentonite-ZVI slurry mixed sand becomes less compressible with increasing effective stress. The measured values of \( m_v \) for the bentonite-ZVI slurry mixed sand ranged from 0.013 m²/MN (Figs. 6(e) and 6(f), small-strain theory) to 0.241 m²/MN (Fig. 6(a), large-strain theory). On the basis of the classification by Tomlinson (1986), the bentonite-ZVI slurry mixed sand specimens ranged from very low compressibility (\( m_v \leq 0.30 \) m²/MN) to only medium compressibility (\( 0.10 \) m²/MN \( \leq m_v \leq 0.30 \) m²/MN), which is in contrast to the significant strains experienced by some of the specimens (see Fig. 4). The lack of higher values of \( m_v \) can be attributed to the fact that the majority of the measured strain in the specimens occurred at \( \varepsilon_v \leq 100 \) kPa (14.5 psi), whereas the data used to determine \( m_v \) were restricted to \( \varepsilon_v > 100 \) kPa (14.5 psi) due to the aforementioned problem associated with the difficulty of determining accurate values of \( \varepsilon_v \) less than about 30 kPa (4.4 psi) to 60 kPa (8.7 psi). Thus, in this case, the calculated \( m_v \) values do not truly reflect the overall compressibility of the specimens.

Values for the hydraulic conductivity, \( k \), calculated as the product of the measured \( c_v \) and \( m_v \) values and the unit weight of water, \( \gamma_w \), or \( k = c_v m_v \gamma_w \), are plotted versus \( \varepsilon_v \) for each specimen of bentonite-ZVI slurry mixed sand in Fig. 7. As shown, differences between the calculated \( k \) values resulting from the different theories or location of the specimen boundary are virtually imperceptible. This closeness in the calculated \( k \) values is due to the compensating effect of the increase in \( c_v \) with increasing \( \varepsilon_v \) (Fig. 5) versus the decrease in \( m_v \) with increasing \( \varepsilon_v \) (Fig. 6). The calculated \( k \) values for all six specimens ranged from 3.4 \( \times 10^{-8} \) m/s (Fig. 7(a)) to 1.9 \( \times 10^{-8} \) m/s (Fig. 7(b)). As expected, this range in \( k \) values is higher than those typically reported for clay specimens, but lower than those typically reported for sand specimens. Castelbaum and
Shackelford (2009) measured \( k \) of bentonite slurry mixed sand specimens using the same mixing apparatus as used in the present study to prepare the specimens, and found that \( k \) ranged from 2.4 \( \times \) 10^{-6} m/s to 6.8 \( \times \) 10^{-6} m/s, depending primarily on the amount of bentonite in the specimen and the void ratio of the bentonite in the specimen. Thus, the range in \( k \) values calculated for the example tests presented here is at the lower end of the range of values measured by Castelbaum and Shackelford (2009).

**Effect of Testing Column Length**

The testing column length essentially reflects the initial height or thickness of the specimen, \( h_0 \), prior to consolidation (Table 1). In this regard, two testing columns, Nos. 2 and 3, contained intact specimens (Nos. 4 and 5) that were prepared in an identical manner and consolidated at the same strain rate (i.e., 2.50 \( \times \) 10^{-6} s^{-1}), such that the primary difference between the specimens was the length of the testing columns (i.e., 305 mm (12 in) for No. 4 and 457 mm (18 in) for No. 5) or specimen thickness (i.e., 240 mm (9.4 in) for No. 4 and 310 mm (12 in) for No. 5). Thus, a comparison of the results from test Nos. 4 and 5 allows an evaluation of the effect of testing column length or specimen thickness on the results, although the slight difference in the bentonite contents in these two specimens (2.33 % versus 2.58 %; see Table 1) also should be considered.

The similarity in the results suggests that the stress-behaviors of the two specimens were essentially independent of the specimen thickness. As expected, the values of \( u_h \) and PPR for the specimens increased as the specimen thickness increased, due to the increase in drainage distance. The values of \( c_v \) for the thicker specimen (No. 5) tended to be lower than those for the thinner specimen (No. 4) at any given value of \( \sigma_0^\prime \). This difference can be attributed directly to the difference in the drainage distances for the two specimens. For values of \( m_r \), the test specimen in the longer column was less than a factor of two more compressible than the test specimen in the shorter column at low values of \( \sigma_0^\prime \), and...
this difference in compressibility decreased with increasing $\sigma'$. Thus, based on both the similarity in stress-strain behaviors and the trends in and magnitudes of the $m$ values for specimen Nos. 4 and 5, the difference in specimen thicknesses had a relatively minor effect on the compressibility of the two specimens.

Effect of Applied Strain Rate

Two testing columns, Nos. 3 and 4, of identical length (457 mm (18 in)) contained intact specimens (Nos. 5 and 6) that were prepared in an identical manner, but specimen No. 6 was consolidated at a slower strain rate ($6.33 \times 10^{-7}$ s$^{-1}$) relative to specimen No. 5 ($2.50 \times 10^{-6}$ s$^{-1}$). In addition, both of these specimens contained the same amount of bentonite (Table 1). Therefore, a comparison of the results from these two tests allows for an evaluation of the effect of the applied strain rate during consolidation testing on the results. The consolidation behavior of a soil specimen is controlled by a unique vertical effective stress-vertical strain-rate relationship (Leroueil et al. 1985, Leroueil 1988). For specimen Nos. 5 and 6, at any given value of $\varepsilon_v$, the value of $\sigma'_v$ increased as $SR$ increased (Fig. 4). This trend of increasing $\sigma'_v$ with increasing $SR$ has been reported extensively (e.g., Kabbaj et al. 1986, Leroueil 1988, Silvestri et al. 1986). In terms of the maximum vertical strain, $SR$ had little effect on the results of specimen Nos. 5 and 6 (maximum difference < 0.3 %, regardless of definition of strain). Consequently, $SR$ had virtually no effect on the $m$ values (compressibility) of the two specimens.

The excess pore-water pressures generated at the undrained base of a specimen, $u_b$, and the calculated PPR values increased as $SR$ increased. These trends previously have been reported for CRS testing (e.g., Smith and Wahls 1969, Znidarcic et al. 1986, Leroueil et al. 1985, Armour and Drnevich 1986, Silvestri et al. 1986, Wissa et al. 1971). The difference between the $c_v$ values for the two specimens based on strain rate was minor, with a tendency toward slightly decreasing $c_v$ with increasing $SR$.

Summary and Conclusions

A CRS consolidation apparatus was developed to allow testing of slurry mixed soils prepared with a hollow-stem auger mixing machine. Specimens of bentonite-ZVI slurry mixed sand were prepared with the laboratory mixing apparatus and transferred directly to a consolidation load frame within the original mixing columns, thereby minimizing specimen disturbance after preparation. Data were recorded continuously throughout the consolidation tests via an automatic data acquisition system. The required test durations for the mixing stage and consolidation stage were < 1 day and < 3 days, respectively.

Example results are provided for six specimens of bentonite-ZVI slurry mixed sand. As expected on the basis of the loose nature of the prepared specimens, all the specimens were highly compressible, with maximum vertical strains, $\varepsilon_v,max$, ranging from 9.63 % to 48.0 %. The unique consolidation behavior of the bentonite-ZVI slurry mixed sand resulted in two difficulties associated with the CRS testing. First, the use of slow strain rates and the high sand content of the specimens resulted in the generation of low values of excess pore-water pressures ($< 3$ kPa (0.4 psi)). Second, significant scatter in the stress-strain data occurred for $\sigma'_v$ less than about 30 kPa (4.4 psi) to 60 kPa (8.7 psi), depending on the specimen, and this scatter did not dissipate completely in all specimens until $\sigma'_v$ was greater than about 100 kPa (14.5 psi). As a result of these difficulties, only the stress-strain data corresponding to $\sigma'_v \geq 100$ kPa (14.5 psi) could be used to determine the relevant consolidation parameters (i.e., $c_v$, $m$, $k$). Thus, compressibility of the specimens at $\sigma'_v < 100$ kPa (14.5 psi) may not be reflected by the determined consolidation parameter values.

Comparison of the results based on two specimens strained at the same strain rate ($2.50 \times 10^{-6}$ s$^{-1}$), but with different column lengths (e.g., specimen thicknesses) indicated that the effect of column length was insignificant. Similarly, comparison of the results based on two specimens of the same thickness but strained at different rates ($6.33 \times 10^{-7}$ s$^{-1}$ and $2.50 \times 10^{-6}$ s$^{-1}$) indicated that the effect of the applied strain rate was minor.

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References


