Geotechnical Properties of FGD Scrubber Material

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BACKGROUND

Depending on the process used, flue gas desulfurization (FGD) product results when sulfur dioxide (SO₂) is removed from coal combustion flue gas. SO₂ may be reacted with an alkaline material, forming calcium sulfite or calcium sulfate. Calcium sulfite is formed in inhibited oxidation FGD systems. This calcium sulfite residual, referred to here as FGD scrubber material, may be partially dewatered and mixed with stabilization additives prior to sluicing as a slurry to an impoundment for storage.

When an impoundment is closed, a soil cover is generally needed to limit infiltration and support vegetation. Construction over the soft FGD materials is challenging. Geotechnical characterization of the FGD scrubber material is necessary to support design and construction planning for cover and closure of an impoundment. This paper presents results from standard geotechnical and hydraulic tests, performed on samples of an FGD material from a coal burning power plant. We also consider the application of these parameters in planning construction of a soil cover over a deposit of the tested FGD material.

FLUE GAS DESULFURIZATION TEST SAMPLES

The power plant collected FGD scrubber material from the environmental control equipment, providing samples in 5-gallon buckets. The samples were identified as calcium sulfite slurry, with a solids content from 19% to 21%, and a pH of 5.8.

The solids settled to the bottom of the sample containers and the clear liquid was decanted. Saturated hydraulic conductivity and one-dimensional consolidation testing require the addition of water during the tests. The decanted liquid was retained and used for this purpose, to maintain the chemical equilibrium of the solutes.
FGD material from a power plant may differ considerably within the span of a day (Krizek 2004), making it difficult to generalize properties. This paper provides data for one FGD scrubber material, which may or may not be typical for the impounded FGD materials at other power plants.

**TEST RESULTS**

Many of the completed tests require determining the gravimetric moisture content of the material, to estimate the sample dry density and/or volumetric moisture content. Gravimetric moisture content was determined by drying the FGD material in a 60°C oven, in accordance with ASTM D2216. This test standard recommends a drying temperature of 60°C for materials containing gypsum (calcium sulfate dihydrate) or other compounds having significant amounts of hydrated water.

**Index Properties**

The FGD material had a specific gravity of 2.52, when tested per ASTM D854. Air was removed from the slurry sample using a vacuum, instead of boiling with a hot plate, because of the heat sensitivity of the material.

The FGD material was tested for particle size gradation using mechanical sieving and the hydrometer method of sedimentation, in accordance with ASTM D422. The material had 100% passing the 2 mm sieve (ASTM Sieve No. 10) and 81.2% passing the 0.075 mm sieve (ASTM Sieve No. 200). The material had 58.1% silt-sized and 23.1% clay-sized particles.

The FGD material had a measured liquid limit of 51 and a plastic limit of 28, resulting in a plasticity index of 23, when tested in accordance with ASTM D4318. The sample classified as a fat clay with sand (CH), based on the Unified Soil Classification System (USCS) described by ASTM D2487. Figure 1 illustrates the FGD material and similar soils based on the Atterberg Limits. The FGD material is near the intersection of classification as a high or low plasticity clay or silt. Another FGD sludge is shown on the plot as soil number 11. That FGD material has a greater liquid limit and smaller plasticity index, and would classify as a silt (ML) per the USCS.
**One-Dimensional Consolidation**

Multiple, one-dimensional consolidation tests were performed on the FGD material, in accordance with ASTM D2435. A computer-controlled loading frame was used for the incremental load testing. Figure 2 presents the test results, with vertical strains at varying vertical loading and unloading stress values. Load increments ranged from 40 to 14,640 pounds per square foot (psf), with a maximum volumetric compression of about 66%.

Initial void ratios in the FGD consolidation test specimens ranged from 6.2 to 7.4 (the samples were thus 86% to 88% water by volume). Table 1 presents the compression index ($C_c$), the recompression index ($C_r$), and average coefficient of consolidation ($C_v$) determined in each of the four tests. For comparison, the coefficient of compression for typical silts and low plasticity clays range from about 0.2 to 0.6, and highly plastic clays and organic soils may have values exceeding 10. The recompression index often ranges from 0.1 to 0.2 of the $C_c$. 
Figure 2. Consolidation testing with vertical effective stress.

Table 1. Consolidation testing results on FGD.

<table>
<thead>
<tr>
<th>Test</th>
<th>$C_v$ (ft$^2$/day)</th>
<th>$C_v$ (m$^2$/s)</th>
<th>$C_c$</th>
<th>$C_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.3</td>
<td>$2.4 \times 10^{-6}$</td>
<td>3.1</td>
<td>0.05</td>
</tr>
<tr>
<td>2</td>
<td>0.8</td>
<td>$9.0 \times 10^{-7}$</td>
<td>3.3</td>
<td>0.06</td>
</tr>
<tr>
<td>3</td>
<td>1.6</td>
<td>$1.7 \times 10^{-6}$</td>
<td>2.2</td>
<td>0.04</td>
</tr>
<tr>
<td>4</td>
<td>1.0</td>
<td>$1.1 \times 10^{-6}$</td>
<td>2.7</td>
<td>0.04</td>
</tr>
</tbody>
</table>

Figure 3 presents the void ratio versus vertical effective stress for some fine-grained, mineral waste materials. The values listed for other FGD materials have been highlighted with a blue line. The red dots represent the trend measured for the material tested in this study (data in Figure 2).
Vane Shear Strength Testing

Vane shear strength testing was performed on five consolidated samples, in accordance with ASTM D4648. The samples were compressed in a batch consolidometer to five different vertical pressures, ranging from 100 to 3,000 psf. After consolidation of each sample, multiple vane shear tests were performed to measure an average undrained shear strength. The shear strength tests were performed using a laboratory mini vane device, with either a 0.5-inch or 1-inch diameter vane. Each vane shear test was performed in a different location within a consolidated test specimen.

The consolidation pressures, water contents, and average measured shear strengths are presented in Table 2. Figure 4 and Figure 5 plot the measured shear strengths versus the consolidation pressure and water content, respectively.

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>Consolidation Pressure (psf)</th>
<th>Water Content (%)</th>
<th>Vane Dimensions</th>
<th>Undrained Shear Strength (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>99</td>
<td>240.8</td>
<td>1&quot; dia. x 1&quot;</td>
<td>48</td>
</tr>
<tr>
<td>2</td>
<td>504</td>
<td>227.0</td>
<td>1&quot; dia. x 1&quot;</td>
<td>73</td>
</tr>
<tr>
<td>3</td>
<td>1008</td>
<td>205.2</td>
<td>0.5&quot; dia. x 0.5&quot;</td>
<td>244</td>
</tr>
<tr>
<td>4</td>
<td>1512</td>
<td>182.8</td>
<td>0.5&quot; dia. x 0.5&quot;</td>
<td>545</td>
</tr>
<tr>
<td>5</td>
<td>3024</td>
<td>149.4</td>
<td>0.5&quot; dia. x 0.5&quot;</td>
<td>1259</td>
</tr>
</tbody>
</table>
Figure 4. Undrained strength of consolidated FGD material measured with vane shear tests.

Figure 5. Increase of undrained shear strength with decreasing moisture content.

SATURATED HYDRAULIC CONDUCTIVITY

Three samples of FGD material were tested to measure the coefficient of saturated hydraulic conductivity ($K_{sat}$) in accordance with ASTM D2434. Table 3 presents the
initial moisture contents \( (w_0) \), dry densities, initial testing void ratios, and results of the saturated hydraulic conductivity testing, which were about \( 10^{-4} \) cm/sec.

<table>
<thead>
<tr>
<th>Water Content (%)</th>
<th>Dry Density (lbf/ft(^3))</th>
<th>Void Ratio</th>
<th>( K_{\text{sat}} ) (cm/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>368%</td>
<td>14.8</td>
<td>9.6</td>
<td>( 2.9 \times 10^{-4} )</td>
</tr>
<tr>
<td>265%</td>
<td>20.1</td>
<td>6.8</td>
<td>( 1.6 \times 10^{-4} )</td>
</tr>
<tr>
<td>205%</td>
<td>25.5</td>
<td>5.2</td>
<td>( 1.2 \times 10^{-4} )</td>
</tr>
</tbody>
</table>

As the consolidation pressure increased, the sample dry density increased, and the gravimetric moisture content, void ratio, and saturated hydraulic conductivity \( (K_{\text{sat}}) \) decreased. Figure 6 presents values of permeability (saturated hydraulic conductivity) typical of some fine-grained soils. Values for other FGD materials are highlighted in blue, and data for the tested samples are shown as red circles.

![Figure 6. Permeability of fine-grained materials (after Carrier et al. 1983). The red circles represent tested FGD material, the blue line represents other FGD sludges.](image)

**SOIL WATER CHARACTERISTIC CURVE**

The same three samples tested for saturated hydraulic conductivity were tested to develop soil water characteristic curves (SWCC) during desorption, in accordance with ASTM D6836. Desorption tests included the hanging column, pressure plate, and a relative humidity box, each method capable of applying greater negative pressures to the samples. The measured negative pressures and volumetric moisture content values were fit to a sigmoidal curve function, as described by van Genuchten (1980). The resulting SWCCs are presented in Figure 7. Plots of SWCC for a typical clay, silt, and sand are presented for comparison.
The moisture characteristic curve data were used with measured values of saturated hydraulic conductivity to estimate the unsaturated hydraulic conductivity, using the methods described by van Genuchten (1980). Figure 8 presents the unsaturated hydraulic conductivity functions versus pressure head for the tested samples. Plots of SWCC for a typical clay, silt, and sand are presented for comparison.
OBSERVED FGD MATERIAL BEHAVIOR

Particle-Water Interactions

Slurry suspensions behave as fluids. As the slurry material de-waters, it transitions from a viscous fluid to a soil–like material (Krizek 2004). Not until the particles contact one another, and initiate the transfer of effective stress, do the materials have a meaningful value of void ratio. This value, at the beginning of the transfer of effective stress, is referred to as the “zero effective stress void ratio”. A slurry of FGD material behaves in this manner.

Figure 4 illustrates the relationship between consolidation stress and undrained shear strength for the FGD material. For consolidation stresses less than about 500 psf, the FGD material exhibits little shear strength, corresponding to the viscosity of the slurry. This suggests that particle contact and effective stress develops at a consolidation pressure of about 500 psf. Consolidation testing data suggest a void ratio between 5 and 6.5 (gravimetric water content between 200% and 260%) at a consolidation stress of 500 psf. This value is about one-half of that reported by Krizek (2004), who references zero effective stress void ratios of 10 to 12 for FGD sludge.

Carrier et al. (1983) suggest estimating the zero effective stress void ratio for the condition where the water content is seven times the liquid limit. This would be a
gravimetric water content of 360% (i.e. $7 \cdot 51\% \approx 360\%$) for the FGD material. Based on large strain consolidation of kaolin slurries, Monte and Krizek (1976) suggest estimating the zero effective stress void ratio as corresponding to a water content five times the liquid limit, which would be a water content of 260% (i.e. $5 \cdot 51\% = 260\%$) for the FGD material. The results for this FGD material are consistent within these reported values.

**Undrained Strength**

The data in Figure 4 indicates very little strength for consolidation pressures less than 500 psf. Assuming an average, saturated unit weight of about 75 lbf/ft$^3$, the effective stress in a saturated sediment of FGD slurry is 500 psf at a depth of approximately 40 feet. Hence, negligible strength might be expected in the upper 40 feet of an FGD impoundment.

Figure 4 also shows that the FGD material exhibits a linear increase in strength above a consolidation pressure of 500 psf. The increase in strength is approximately one-half the increment of consolidation stress (i.e., an additional 1,000 psf of consolidation stress increases the undrained shear strength by 500 psf). Figure 5 suggests that each 10% decrease in moisture content, below about 225%, increases the undrained shear strength by 150 psf.

**Drainage and Strength**

According to Krizek (2004), FGD sludges have a strength behavior similar to silts. Drained strength envelopes have small to negligible cohesion intercepts and significant internal friction angles. Consolidated-undrained friction angles are expected to be slightly greater than one-half of the drained values.

De-watering the FGD material will increase the strength. In an unsaturated state, the water between solid particles will apply a tensile force (matric suction) due to surface tension. Fredlund et al. (2012) presents a relationship for soil shear strength, which extends the Mohr-Coulomb failure criterion to include the contribution of the matric suction:

$$
\tau_f = c' + (\sigma_f + u_a) \tan \phi' + (u_a - u_w) \tan \phi^b
$$

Where:

- $\tau_f$ = shear stress on the failure plane at failure
- $c'$ = effective stress cohesion
- $\sigma_f + u_a$ = effective stress on the failure plane at failure (where $u_a$ is the effective pore air pressure)
- $\phi'$ = effective stress angle of internal friction
- $u_a - u_w$ = matric suction of the soil [where $u_w$ is the effective soil pore water pressure less the negative pressure (y-axis) from Figure 7]
- $\phi^b$ = the angle representing the rate of increase in shear strength with respect to a change in matric suction
Drained and unsaturated strength parameters \((c', \phi', \phi^b)\) were not measured on the FGD material. Additional strength testing of the FGD product, at different values of matric suction, would provide \(\phi^b\). For perspective, Fredlund et al. (2012) report a \(\phi^b\) value of 16 degrees for Tappen-Notch Hill silt.

**APPLICATION FOR FIELD CONDITIONS**

To demonstrate the possible application of the measured FGD properties, a common field problem was considered. Figure 9 illustrates construction of a hypothetical soil road on a deposit of FGD material, de-watered to some depth below the surface.

![Figure 9. Hypothetical spreading of a soil cover on FGD material.](image)

**Bearing Capacity**

Assuming \(\phi = 0\) and an undrained shear strength of \(S_u\), the theoretical, ultimate bearing capacity \(q_{ult}\) for the soil road (strip loading) can be computed using:

\[
q_{ult} = 5.14 S_u
\]

We assumed a low-ground-pressure CAT D6 bulldozer is used to spread a layer of cover soil, 3-feet thick and 12-feet wide, over a saturated FGD deposit. The bearing pressure on the FGD surface would be about 1,000 psf. The strength \((S_u)\) of the shallow
FGD would have to be greater than 200 psf to develop the required bearing capacity, and $S_u$ would have to be about 400 psf to develop a minimal safety factor of two. This is much more than the strengths measured in the FGD at consolidation pressures below 500 psf (Figure 4), representing the conditions in the upper 20 feet of a saturated FGD deposit.

Undrained shear strengths were estimated for the tested FGD material, assuming the water table was lowered by pumping or drainage. The relationship described by Fredlund et al. (2012) was used to approximate the strength in the de-watered zone. We used the measured SWCC (Figure 7) and assumed effective stress strength parameters (drained condition) of 25 degrees for friction angle and 0.0 psf for cohesion. The undrained shear strengths measured with the laboratory vane shear (Figure 4) were applied to the FGD material below the water table. We assumed weighted averages of drained and undrained strength above and below the phreatic surface, over a depth of 18 feet (1.5 times the width of the road).

From this analysis, we estimated that the FGD stratum would have to be de-watered approximately 21 feet, to achieve a factor of safety against bearing capacity failure of 2.0. Note that the stability of this hypothetical condition is minimal and may not be safe. Higher safety factors and greater dewatering may be prudent for the safety of construction operations. Geotextiles and/or plastic geogrid products could also be used to improve stability of the embankment.

The theoretical results presented here should not be used as design guidance, nor interpreted as predictions of likely performance at a given site. The results have not been verified with field experience. The conditions in a real FGD impoundment could be substantially worse than assumed in this example. In all cases, construction on these potentially unsafe conditions must be planned and executed with extreme care.

**Consolidation Settlements**

Figure 2 presents the results from consolidation testing on four samples of FGD material. Figure 2 uses vertical strain, so that the shapes of the functions from the different tests may be compared. The consolidation curve shapes are very similar, implying a consistent consolidation behavior.

The $C_c$ may be used to estimate the primary consolidation settlements due to increases in applied vertical stress. The average $C_c$ for the four samples was 2.75; for each one order of magnitude increase in vertical stress, the void ratio will decrease by 2.75. The initial void ratios of the samples ranged from 6.2 to 7.4.

Using the coefficient of compression obtained from testing, we estimated the elastic and primary consolidation settlement that would result from constructing the hypothetical soil road. We ignored the significant settlements that would occur due to lowering the water table in the FGD material, and considered only the settlement that would result from placing the soil embankment depicted in Figure 9. Assuming 50 feet of FGD material
with a water table 21 feet below the surface, we estimated total settlements of approximately 11 inches.

The $C_v$ values measured for the FGD material ranged from 0.84 to 2.25 ft$^2$/day. This value is used to predict the duration of consolidation due to drainage. The $C_v$ depends on the material compressibility and coefficient of saturated hydraulic conductivity. NAVFAC (1986) indicates an upper value $C_v$ of 0.05 ft$^2$/day for completely remolded silts and clays with a liquid limit of 50. The values of $C_v$ for the FGD material are one to two orders of magnitude greater, meaning that the FGD material will consolidate much more rapidly than typical silts or clays.

**IMPLICATIONS FOR FIELD BEHAVIOR**

The laboratory tests completed on the FGD material suggest that the solid particles are weakly adhering large amounts of water. The adhered water, which may be squeezed out by force or removed by heat, has a significant effect on the behavior of the FGD material in a sluiced impoundment.

The test results indicate that the FGD material has negligible strength (corresponding to the viscosity of the slurry) when compressed at pressures less than about 500 psf. At these low pressures, the solid FGD particles may be separated by the adhered water, not in physical contact to transfer effective stresses, and exhibit negligible strength compared to typical soils. Hence, the upper 40 feet in an impoundment of this FGD material, where the effective stress will be less than 500 psf, will have negligible strength.

To construct a soil cover over impounded FGD material, the material must be compressed under an adequate pressure to develop the necessary bearing capacity. The minimum needed consolidation pressure is at least 500 psf (equivalent to a 5-foot or thicker layer of soil). Safely placing this material on top of the impounded, unconsolidated FGD material is probably not practical. Soil pushed out over the surface of saturated FGD would likely sink into the viscous sludge.

One way to develop the needed bearing capacity is to dewater the upper portions of the FGD, using extraction wells or other dewatering techniques. To generate enough strength in the surficial materials to support construction, it may be necessary to lower the water table approximately 20 feet below the surface of the FGD deposits. Geosynthetics (i.e., geotextiles and/or plastic geogrid) might then be used to improve stability of the surface and allow construction of a cover layer. Dewatering the FGD and placing a cover soil would result in substantial settlements, which must be considered in the design.
REFERENCES


