COMPILATION OF DR. KALINSKI’S RECENT PUBLICATIONS


Kalinski, M. E., 2002, Guidelines for Geotechnical Investigation and Analysis of New and Existing Earth Dams, Kentucky Natural Resources and Environmental Protection Cabinet, Dam Safety and Floodplain Compliance Section.

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Hydraulic conductivity of compacted cement–stabilized fly ash

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Abstract

When combined with portland cement and compacted, fly ash is a high-strength material. In some instances, it may also be desirable to control the hydraulic conductivity \( k \) of the compacted mixture. Therefore, a study was performed to assess the effects of water content \( w \), cement content, curing time, and compaction effort on the hydraulic conductivity of compacted cement–stabilized fly ash. When compacting relatively dry mixtures \( w < 20\% \), \( k \) is independent of compaction effort, and is on the order of \( 10^{-5} \text{ cm/s} \). When compacting between \( w \) of 20\% and optimum water content \( w_{\text{opt}} \), compaction effort affects \( k \), and, at a given \( w \), \( k \) decreases by about an order of magnitude when increasing from standard to modified proctor effort. When wet of \( w_{\text{opt}} \), \( k \) is on the order of \( 10^{-6} \text{ cm/s} \) regardless of compaction effort or water content. With respect to curing time, extended curing time has relatively little effect on \( k \) within a 60-day time frame. Based on the results of this study, an approach to construction quality assurance testing can be applied to estimate \( k \) based on in situ measurement of dry density \( \rho_d \) and \( w \).

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Keywords: Fly ash; Compaction; Hydraulic conductivity

1. Introduction

Ash is produced from the combustion of coal at a global rate of approximately 480 million metric tons per year [1]. This includes bottom ash and flue gas desulfurization (FGD) material, but about two thirds of the ash is in the form of airborne particles, also known as fly ash. Fly ash is a fine-grained, dusty material that consists of \( \text{SiO}_2 \), \( \text{Al}_2\text{O}_3 \), \( \text{CaO} \) (quicklime), and other minor constituents, and is primarily produced by electric power plants. Due to its abundance, it is advantageous for fly ash producers to identify practical uses for fly ash rather than dispose of it in landfills at a significant cost. Numerous uses have been identified for fly ash in construction, including soil stabilization and portland cement supplementation [2].

Fly ash usage is also gaining in popularity in the agriculture industry for the construction of high-strength cattle feedlot pads [3]. In the European Union, over 90% of coal combustion byproducts (CCBs), including fly ash, are recycled [1]. In other parts of the world, a much smaller percentage of fly ash (for instance, about one quarter in the United States) is utilized, with the remaining fly ash disposed in landfills [2]. Clearly, there is significant room for improvement with respect to utilizing fly ash, and it is of benefit to identify uses in which fly ash is the primary constituent, rather than an additive, to accelerate its consumption.

Class F fly ash is a product of the combustion of older, harder bituminous and anthracite coal. In contrast, class C fly ash is a product of the combustion of younger lignite and sub-bituminous coal. According to ASTM C618, Class F fly ash must satisfy the following criteria: (1) the \( \text{SiO}_2 + \text{Al}_2\text{O}_3 + \text{Fe}_2\text{O}_3 \) content must be greater than 70\% by weight, (2) the \( \text{SO}_3 \) content must be less than 5\% by weight, and (3) the loss of ignition (LOI) must be less than
Coal combustion byproducts have also been used as low-permeability hydraulic barriers in a manner similar to compacted clay. Wolfe et al. [4] demonstrated the use of compacted FGD material for the construction of low-permeability liners for animal waste lagoons. In their research, FGD material was mixed with fly ash and compacted to form a hydraulic barrier at the bottom of the lagoon. Hydraulic conductivity \( k \) ranging from \( 3.6 \times 10^{-5} \) to \( 1.0 \times 10^{-8} \) cm/s was measured in laboratory specimens prepared using standard proctor compaction effort (ASTM D698), while large-scale field measurements yielded steady state \( k \) around \( 4.5 \times 10^{-7} \) cm/s. Hydraulic conductivity, \( k \), is defined using Darcy’s law (Fig. 1):

\[
\frac{Q}{t} = k \frac{\Delta h}{L A},
\]

where \( Q/t \) through a specimen of cross-sectional area \( A \) and length \( L \) is measured while maintaining a constant fluid head loss \( \Delta h \) across the specimen.

Ghosh and Subbarao [5] also demonstrated that fly ash, when combined with hydrated lime and compacted using standard proctor effort, possesses \( k \) on the order of \( 1.0 \times 10^{-5} \) cm/s. They further demonstrated that \( k \) decreases with increasing hydrated lime content and increasing water content \( w \), and that with the addition of around 1% gypsum (CaSO\(_4\) \cdot 2H\(_2\)O), \( k \) can be reduced to around \( 1.0 \times 10^{-7} \) cm/s. With respect to the \( 1.0 \times 10^{-7} \) cm/s criterion commonly prescribed in regulations for compacted clay liners (CCLs) for waste containment, these results indicate that, in some instances, compacted CCBs, including compacted fly ash, may perform satisfactorily. These studies also demonstrated that the water quality of the effluent permeating through these materials is not significantly impacted by the CCB, and that acceptable water quality standards are achievable.

Like compacted clay, the dry density of compacted fly ash varies with water content. Dry density, \( \rho_d \), is defined as

\[
\rho_d = \frac{M_s}{V},
\]

where \( M_s \) is the mass of dry solids in a volume \( V \). Water content, \( w \), is defined as

\[
w = \frac{M_w}{M_s} \times 100\%.
\]

where \( M_w \) is the mass of water. For a given compaction effort, the relationship between \( \rho_d \) and \( w \), referred to as a compaction curve, is a concave-downward plot of \( \rho_d \) versus \( w \), with a maximum dry density, \( \rho_{d_{\text{max}}} \), at a corresponding optimum water content, \( w_{\text{opt}} \). Ghosh and Subbarao [5] demonstrated that \( k \) of compacted fly ash is less when compacting wet of \( w_{\text{opt}} \) and recommend that \( w \) be wet of \( w_{\text{opt}} \) to minimize \( k \). Similar observations have been made for compacted clay by Mitchell et al. [6]. They demonstrated that \( k \) decreases by several orders of magnitude when going from dry of \( w_{\text{opt}} \) to wet of \( w_{\text{opt}} \), and that \( k \) less than \( 1.0 \times 10^{-7} \) cm/s can be achieved when wet of \( w_{\text{opt}} \).
Daniel and Benson [7] developed an acceptance window approach for specifying $\rho_d$ and $w$ for compacted clay to minimize $k$ by combining two or more compaction curves generated at different compaction efforts. By compacting within an acceptance window that includes the locus of points wet of $w_{opt}$, $k$ will be low. This approach can be expanded by measuring $k$ of each compacted specimen, superimposing constant $k$ contours over the compaction curves, and delineating an acceptance window that satisfy a desired $k$ criterion (e.g. $k < 1.0 \times 10^{-7}$ cm/s). This approach is particularly attractive because it eliminates the requirement that the field compaction effort be equivalent to a specific laboratory compaction effort.

As mentioned previously, fly ash compacted with portland cement is currently being used as a construction material for livestock feedlot pads due to its strength and durability [3]. To provide information for the analysis of surface water and infiltration rates for this material, accurate estimate of $k$ is required. Therefore, the study detailed herein was performed to measure $k$ of this material using various cement contents, water contents, and compaction efforts. Another purpose of this study was to elaborate on existing research regarding the hydraulic characteristics of compacted fly ash–cement mixtures to develop relationships between $\rho_d$, $w$, and $k$ in a manner similar to the approach used for compacted clay. By establishing this relationship, in situ $k$ of compacted fly ash–cement mixtures can be predicted based on field measurement of $\rho_d$ and $w$ using a number of in situ methods.

2. Testing procedures

Specimens were prepared using fly ash obtained from combustion of coal at the Kentucky Utilities electric power plant in Tyrone, Kentucky. The composition of two representative samples of the fly ash, listed in Table 1, indicates that the fly ash was class F, with less than 10% quicklime. The specific gravity of the fly ash solids, $G_s$, was 2.24 as measured using ASTM D854. As mentioned previously, ASTM C618 requires that the loss of ignition (LOI) for class F fly ash be less than 6%. The LOI criterion is intended for fly ash used in concrete, where excessive uncombusted coal may lead to air entrainment problems and problems with the performance of concrete. However, the application described herein is different, so uncombusted coal was not considered to be a critical issue and LOI testing was not performed.

The fly ash was combined with portland cement using cement contents of 0%, 5%, 10%, and 15%. Cement content, $c$, is defined as

$$c = \frac{M_c}{M_a} = \frac{M_c}{M_c + M_a},$$

where $M_a$ is the mass of fly ash and $M_c$ is the mass of the added cement. Fly ash and cement were mixed, and water was added to achieve a desired $w$. The fly ash, cement, and water were thoroughly blended in a 15-qt (14-l) bench mixer. After mixing and prior to compaction, the mixture was allowed to “mellow” for one hour. Specimens were compacted using standard and modified proctor compaction effort (ASTM D698 and D1557, respectively). Proctor compaction involves dropping of a weight a fixed number of times from a fixed height. For standard proctor testing, a 24-N weight is dropped 75 times from a height of 30 cm. For modified proctor testing, a 45-N weight is dropped 125 times from a height of 45 cm. Thus, the standard test corresponds to low compaction effort, while the modified test corresponds to higher compaction effort.

Several specimens were compacted over a range in $w$ that included $w_{opt}$ to fully define each compaction curve. Based on previous research with the same material [3], it was expected that $w_{opt}$ would be around 35% and 25% for specimens compacted using standard and modified proctor effort, respectively. After compaction, the specimens were placed in sealed plastic bags and allowed to cure for seven days. The specimens were compacted in a cylindrical mold, with a length and diameter of 11.6 cm and 10.2 cm, respectively.

After curing, the specimens were placed in a permeameter, and flexible wall permeability testing was performed in accordance with ASTM D5084 to measure $k$ as illustrated in Fig. 2. To perform the test, a cell pressure of 59 kPa was used, along with an influent and effluent pressure of 48 kPa and 0 kPa, respectively. Herein, all reported pressures are gauge pressures rather than absolute pressures. The net pressure gradient of 48 kPa was equivalent to a static head of 4.9 m. Deaired tap water was used as a permeant, and permeation was in an upward direction to help dislodge air bubbles trapped in the specimen. To assess the effect of long-term curing on $k$, specimens were stored in plastic bags after permeation, and retested after 60 days.

<table>
<thead>
<tr>
<th>Constituent</th>
<th>% by Weight (Sample 1)</th>
<th>% by Weight (Sample 2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO$_2$</td>
<td>54.45</td>
<td>57.63</td>
</tr>
<tr>
<td>Al$_2$O$_3$</td>
<td>26.67</td>
<td>24.73</td>
</tr>
<tr>
<td>Fe$_2$O$_3$</td>
<td>6.91</td>
<td>7.78</td>
</tr>
<tr>
<td>CaO</td>
<td>2.39</td>
<td>1.76</td>
</tr>
<tr>
<td>K$_2$O</td>
<td>2.03</td>
<td>2.02</td>
</tr>
<tr>
<td>TiO$_2$</td>
<td>1.64</td>
<td>1.57</td>
</tr>
<tr>
<td>MgO</td>
<td>1.12</td>
<td>0.81</td>
</tr>
<tr>
<td>P$_2$O$_5$</td>
<td>0.25</td>
<td>0.32</td>
</tr>
<tr>
<td>Na$_2$O</td>
<td>0.21</td>
<td>0.19</td>
</tr>
<tr>
<td>SO$_3$</td>
<td>0.10</td>
<td>0.11</td>
</tr>
<tr>
<td>Barium</td>
<td>0.12</td>
<td>0.11</td>
</tr>
<tr>
<td>Strontium</td>
<td>0.12</td>
<td>0.11</td>
</tr>
<tr>
<td>Zirconium</td>
<td>0.05</td>
<td>0.05</td>
</tr>
<tr>
<td>Vanadium</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>Copper</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>Zinc</td>
<td>0.02</td>
<td>0.01</td>
</tr>
<tr>
<td>Manganese</td>
<td>0.01</td>
<td>0.03</td>
</tr>
<tr>
<td>Arsenic</td>
<td>0.01</td>
<td>0.01</td>
</tr>
<tr>
<td>Nickel</td>
<td>0.01</td>
<td>0.01</td>
</tr>
<tr>
<td>Chromium</td>
<td>0.01</td>
<td>0.01</td>
</tr>
<tr>
<td>Others</td>
<td>3.84</td>
<td>2.70</td>
</tr>
</tbody>
</table>

Table 1 Composition of representative samples of the fly ash used for this study.
3. Results and discussion

Results are summarized in Tables 2–5 for each $c$, indicating $w$, $\rho_d$, and $k$ for each specimen. As illustrated in Fig. 3, the fly ash–cement mixtures exhibited compaction behavior similar to that of compacted clay, with $\rho_{d\text{max}}$ and corresponding $w_{\text{opt}}$. As compaction effort increased from standard to modified proctor effort, $\rho_{d\text{max}}$ increased and $w_{\text{opt}}$ decreased. The zero air voids (ZAV) curve on each plot corresponds to a degree of saturation, $S$, of 100%. Degree of saturation is defined as

$$S = \frac{V_w}{V_v},$$

where $V_w$ and $V_v$ are the volume of water and volume of voids, respectively. As expected, the compaction curves are shifted to the left relative to the ZAV curve, indicating that $S$ was initially less than 100%. At the conclusion of hydraulic conductivity testing, specimens were weighed, and $S$ was recalculated. The average degree of saturation after permeation was 94%.

Hydraulic conductivity is also plotted as a function of $w$ in Fig. 3. As indicated in these plots, $k$ decreased by about an order of magnitude when going from dry to wet of $w_{\text{opt}}$. When compacting wet of $w_{\text{opt}}$, $k$ on the order of $10^{-6}$ cm/s was measured, and $k$ was independent of compaction effort. When increasing from standard to modified proctor compaction effort, the $k$ versus $w$ curve shifts to the left in a manner similar to the compaction curves. Thus, compaction effort plays a role in $k$, and $k$ is a function of both $\rho_d$ and $w$. With respect to the $k$ criterion of $1.0 \times 10^{-5}$ cm/s commonly used for waste containment lin-

### Table 3
Data for specimens compacted with $c = 5$

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>$w$ (%)</th>
<th>$\rho_d$ (kg/m$^3$)</th>
<th>7-day $k$ (cm/s)</th>
<th>60-day $k$ (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Standard proctor (ASTM D698)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5s1</td>
<td>21.2</td>
<td>1091</td>
<td>$2.99 \times 10^{-5}$</td>
<td>$2.40 \times 10^{-5}$</td>
</tr>
<tr>
<td>5s2</td>
<td>24.6</td>
<td>1132</td>
<td>$2.31 \times 10^{-5}$</td>
<td>$1.88 \times 10^{-5}$</td>
</tr>
<tr>
<td>5s3</td>
<td>29.1</td>
<td>1144</td>
<td>$1.20 \times 10^{-5}$</td>
<td>$0.99 \times 10^{-5}$</td>
</tr>
<tr>
<td>5s4</td>
<td>32.5</td>
<td>1179</td>
<td>$0.35 \times 10^{-5}$</td>
<td>$0.27 \times 10^{-5}$</td>
</tr>
<tr>
<td>5s5</td>
<td>36.7</td>
<td>1143</td>
<td>$0.28 \times 10^{-5}$</td>
<td>$0.25 \times 10^{-5}$</td>
</tr>
<tr>
<td>5s6</td>
<td>41.0</td>
<td>1093</td>
<td>$0.20 \times 10^{-5}$</td>
<td>$0.17 \times 10^{-5}$</td>
</tr>
<tr>
<td><strong>Modified proctor (ASTM D1557)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5m1</td>
<td>17.3</td>
<td>1213</td>
<td>$1.75 \times 10^{-5}$</td>
<td>$1.42 \times 10^{-5}$</td>
</tr>
<tr>
<td>5m2</td>
<td>21.2</td>
<td>1223</td>
<td>$1.02 \times 10^{-5}$</td>
<td>$0.91 \times 10^{-5}$</td>
</tr>
<tr>
<td>5m3</td>
<td>25.3</td>
<td>1246</td>
<td>$0.34 \times 10^{-5}$</td>
<td>$0.29 \times 10^{-5}$</td>
</tr>
<tr>
<td>5m4</td>
<td>28.1</td>
<td>1256</td>
<td>$0.24 \times 10^{-5}$</td>
<td>$0.21 \times 10^{-5}$</td>
</tr>
<tr>
<td>5m5</td>
<td>32.6</td>
<td>1196</td>
<td>$0.34 \times 10^{-5}$</td>
<td>$0.32 \times 10^{-5}$</td>
</tr>
</tbody>
</table>

### Table 4
Data for specimens compacted with $c = 10$

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>$w$ (%)</th>
<th>$\rho_d$ (kg/m$^3$)</th>
<th>7-day $k$ (cm/s)</th>
<th>60-day $k$ (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Standard proctor (ASTM D698)</strong></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>10s1</td>
<td>19.3</td>
<td>1030</td>
<td>$5.06 \times 10^{-5}$</td>
<td>$4.06 \times 10^{-5}$</td>
</tr>
<tr>
<td>10s2</td>
<td>24.1</td>
<td>1012</td>
<td>$5.04 \times 10^{-5}$</td>
<td>$4.10 \times 10^{-5}$</td>
</tr>
<tr>
<td>10s3</td>
<td>28.1</td>
<td>1020</td>
<td>$4.16 \times 10^{-5}$</td>
<td>$3.60 \times 10^{-5}$</td>
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<tr>
<td>10s4</td>
<td>34.0</td>
<td>1030</td>
<td>$2.86 \times 10^{-5}$</td>
<td>$2.19 \times 10^{-5}$</td>
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<tr>
<td>10s5</td>
<td>36.7</td>
<td>1049</td>
<td>$1.39 \times 10^{-5}$</td>
<td>$1.22 \times 10^{-5}$</td>
</tr>
<tr>
<td>10s6</td>
<td>41.5</td>
<td>1056</td>
<td>$0.44 \times 10^{-5}$</td>
<td>$0.40 \times 10^{-5}$</td>
</tr>
<tr>
<td>10s7</td>
<td>43.7</td>
<td>1035</td>
<td>$0.42 \times 10^{-5}$</td>
<td>$0.37 \times 10^{-5}$</td>
</tr>
<tr>
<td><strong>Modified proctor (ASTM D1557)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10m1</td>
<td>16.8</td>
<td>1129</td>
<td>$2.42 \times 10^{-5}$</td>
<td>$2.16 \times 10^{-5}$</td>
</tr>
<tr>
<td>10m2</td>
<td>19.4</td>
<td>1134</td>
<td>$2.43 \times 10^{-5}$</td>
<td>$2.12 \times 10^{-5}$</td>
</tr>
<tr>
<td>10m3</td>
<td>24.5</td>
<td>1151</td>
<td>$1.39 \times 10^{-5}$</td>
<td>$1.29 \times 10^{-5}$</td>
</tr>
<tr>
<td>10m4</td>
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<td>1197</td>
<td>$0.32 \times 10^{-5}$</td>
<td>$0.29 \times 10^{-5}$</td>
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<td>10m5</td>
<td>32.9</td>
<td>1197</td>
<td>$0.34 \times 10^{-5}$</td>
<td>$0.29 \times 10^{-5}$</td>
</tr>
<tr>
<td>10m6</td>
<td>36.8</td>
<td>1142</td>
<td>$0.32 \times 10^{-5}$</td>
<td>$0.27 \times 10^{-5}$</td>
</tr>
</tbody>
</table>

### Table 5
Data for specimens compacted with $c = 15$

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>$w$ (%)</th>
<th>$\rho_d$ (kg/m$^3$)</th>
<th>7-day $k$ (cm/s)</th>
<th>60-day $k$ (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Standard proctor (ASTM D698)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15s1</td>
<td>20.7</td>
<td>1151</td>
<td>$2.88 \times 10^{-5}$</td>
<td>$2.46 \times 10^{-5}$</td>
</tr>
<tr>
<td>15s2</td>
<td>25.4</td>
<td>1181</td>
<td>$1.68 \times 10^{-5}$</td>
<td>$1.52 \times 10^{-5}$</td>
</tr>
<tr>
<td>15s3</td>
<td>29.9</td>
<td>1208</td>
<td>$0.62 \times 10^{-5}$</td>
<td>$0.57 \times 10^{-5}$</td>
</tr>
<tr>
<td>15s4</td>
<td>33.4</td>
<td>1213</td>
<td>$0.41 \times 10^{-5}$</td>
<td>$0.38 \times 10^{-5}$</td>
</tr>
<tr>
<td>15s5</td>
<td>38.3</td>
<td>1169</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td><strong>Modified proctor (ASTM D1557)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15m1</td>
<td>17.3</td>
<td>1279</td>
<td>$1.52 \times 10^{-5}$</td>
<td>$1.24 \times 10^{-5}$</td>
</tr>
<tr>
<td>15m2</td>
<td>20.9</td>
<td>1274</td>
<td>$0.92 \times 10^{-5}$</td>
<td>$0.82 \times 10^{-5}$</td>
</tr>
<tr>
<td>15m3</td>
<td>25.0</td>
<td>1318</td>
<td>$0.33 \times 10^{-5}$</td>
<td>$0.28 \times 10^{-5}$</td>
</tr>
<tr>
<td>15m4</td>
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<td>1289</td>
<td>$0.18 \times 10^{-5}$</td>
<td>$0.16 \times 10^{-5}$</td>
</tr>
<tr>
<td>15m5</td>
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<td>1217</td>
<td>$0.33 \times 10^{-5}$</td>
<td>$0.31 \times 10^{-5}$</td>
</tr>
<tr>
<td>15m6</td>
<td>35.9</td>
<td>1176</td>
<td>$0.20 \times 10^{-5}$</td>
<td>$0.19 \times 10^{-5}$</td>
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</tbody>
</table>
ers, this material is not acceptable, although Ghosh and Subbarao demonstrated that the addition of 1% gypsum can reduce $k$ to this level [5]. Nevertheless, relatively low values of $k$ that may be suitable for other applications are attainable.

By superimposing contours of constant $k$ over the compaction curves, $k$ can be predicted based on in situ measurement of $\rho_d$ and $w$. As seen in Figs. 3 and 4, $c$ has little effect on $k$, but there is a clear relationship between $\rho_d$, $w$, and $k$. Thus, for a given acceptance criterion (e.g. $k < 1.0 \times 10^{-5}$ cm/s), $\rho_d$ and $w$ can be measured in situ, and a $\rho_d$–$w$–$k$ relationship such as those shown in Fig. 4 can be used to determine whether or not the material satisfies the criterion. There are a number of methods for measuring $\rho_d$ and $w$ of field compacted materials, including the nuclear gauge (ASTM D3017 and D2922), balloon test (ASTM D2167), and sand cone test (ASTM D1556). By using one or more of these methods on field compacted fly ash–cement mixtures, $k$ can be accurately estimated.

To assess the effect of curing time on $k$, hydraulic conductivity of the fly ash–cement specimens was remeasured after a curing period of 60 days. Measured values for $k$ are also included in Tables 3–5, and illustrated graphically in Fig. 5. These data indicate that with additional curing time, only minor decreases in $k$ are observed, with $k$ decreasing by an average of approximately 13%.

Fig. 3. Compaction curves and hydraulic conductivity data for fly ash–cement mixtures (7-day curing time).
4. Conclusion

The hydraulic conductivity of compacted portland cement–fly ash mixtures is affected by compaction effort and \( w \). When compacting relatively dry mixtures (\( w < 20\% \)), \( k \) appears to be independent of compaction effort, and is on the order of \( 10^{-5} \) cm/s. When compacting between \( w \) of 20\% and \( w_{\text{opt}} \), compaction effort affects \( k \), and, at a given \( w \), \( k \) decreases by about an order of magnitude when increasing from standard to modified proctor effort. When wet of \( w_{\text{opt}} \), \( k \) is on the order of \( 10^{-6} \) cm/s regardless of compaction effort or water content. With respect to curing time, extended curing time has relatively little effect on \( k \) within a 60-day time frame. Since most strength gain and pozzolanic reaction occurs within the first 28 days of portland cement hydration, it is unlikely that \( k \) would change significantly beyond 60 days.

Based on the results of this study, an approach to construction quality assurance (CQA) testing of compacted portland cement–fly ash mixtures can be applied to estimate \( k \) based on in situ measurement of \( \rho_d \) and \( w \) using a number of different methods. Hydraulic conductivities of compacted portland cement–fly ash mixtures on the order of \( 10^{-5} \)–\( 10^{-6} \) cm/s were readily achieved. While it is expected that mixtures using fly ashes of different origin would be comparable, variations may exist, and testing of fly ashes significantly different than the fly ash used in this study should be repeated to validate the results presented herein. Nevertheless, the hydraulic conductivities for the specimens measured during this study are not adequate with respect to the criterion of \( 1.0 \times 10^{-7} \) cm/s often used for waste containment layers, but this material may be suitable for other geotechnical applications such as earth dams and highway base courses. Herein, portland cement was used as a cementing agent due to its rapid hydration, high strength, and availability. However, it is expected that similar measurements and behavior would be observed if either hydrated lime or quicklime were used.

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References


The effect of water content and cement content on the strength of portland cement-stabilized compacted fly ash

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Abstract

Class F fly ash, when combined with portland cement and hydrated, forms a high-strength material whose strength increases with increasing cement content and compaction effort, and is highest near optimum water content of around 20–30%. However, fly ash in stockpiles can be at a water content of around 50%, so completely drying to optimum water content may not be practical. If the material is left at its stockpile water content, the cement content required to achieve a given strength is about 2.5–3 times higher than the cement content required if compacting at optimum water content. However, strength can be predicted as a function of water content for water contents between the stockpile water content and the optimum water content.

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Keywords: Fly ash; Portland cement; Compaction; Strength; Construction quality assurance

1. Introduction

Ash is produced from the combustion of coal at a global annual rate of approximately 480 million metric tons [10]. This includes bottom ash and flue gas desulfurization (FGD) material, but about two thirds of the ash is in the form of suspended ash particles, better known as fly ash. Fly ash is a fine-grained material with spherically shaped particles consisting primarily of SiO2, Al2O3, and Fe2O3. Class C fly ash is derived from the combustion of younger lignite and sub-bituminous coals. It generally contains more than 20% CaO, and forms a high-strength cementitious material when hydrated [1]. Class F fly ash is derived from the combustion of older bituminous and anthracite coals. It generally contains less than 10% CaO, and requires the addition of a cementing agent for cementation [1]. Fly ash is captured using electrostatic precipitators and is often hydraulically transported in a slurry to detention ponds, where it is subsequently removed and stockpiled. As a result, fly ash may exist in stockpiles at water contents as high as 50%. Water content, w, is defined herein in gravimetric terms as:

\[ w = \frac{M_w}{M_s} \times 100\% \]  

where \( M_w \) and \( M_s \) are the mass of water and solids, respectively.

Due to its abundance, it is advantageous for fly ash producers to identify practical construction uses for fly ash rather than dispose of it in landfills at a significant cost. Numerous uses have been identified for fly ash in construction, including soil stabilization [2,3], base course construction [4], engineered fill [5], and portland cement supplementation [6]. Fly ash usage is also gaining in popularity in the agricultural industry for the construction of high-strength cattle feedlot pads to increase daily gain, reduce disease, and facilitate feedlot maintenance [7–9]. In the European Union (EU), over 90% of coal combustion byproducts, including fly ash, are recycled [10], although usage rates vary significantly between EU nations. In other parts of the world, a much smaller percentage of fly ash (for instance, about one quarter in the United States) is utilized, with the remaining fly ash disposed in landfills [1].
Thus, there is significant room for improvement with respect to utilizing fly ash, and it is of benefit to identify uses in which fly ash is the primary constituent, rather than an additive, to accelerate its consumption.

The strength of fly ash is significantly increased by compaction and addition of portland cement. As a result, compacted cement-stabilized fly ash is a high-strength alternative to soil. Fly ash can be compacted over a range of \( w \) to develop the relationship between dry density \( \rho_d \) and \( w \) in the form of a compaction curve. Dry density is defined as:

\[
\rho_d = \frac{M_s}{V},
\]

where \( M_s \) is the mass of solids in a volume \( V \) of soil. [11] demonstrated that like soil, fly ash compaction curves exhibit a concave-downward shape, with a maximum dry density, \( \rho_{d,max} \), and corresponding optimum water content, \( w_{opt} \). They also demonstrated that: (1) the maximum unconfined compressive strength \( q_u \) of fly ash is achieved near \( w_{opt} \), (2) \( q_u \) increases with increasing compaction effort, \( w \), and \( c \) on \( q_u \). The fly ash used had a specific gravity of solids \( (G_s) \) of approximately 2.24, and was composed primarily of \( \text{SiO}_2 \) and \( \text{Al}_2\text{O}_3 \) with other constituents (Table 1). Since the \( \text{SiO}_2 + \text{Al}_2\text{O}_3 + \text{Fe}_2\text{O}_3 \) content was greater than 70% and the \( \text{SO}_3 \) content was greater than 5%, the fly ash is defined as Class F fly ash according to American Society for Testing and Materials (ASTM) test standard C618. The low CaO content is also an indication that the fly ash is Class F, and therefore not self-cementing. ASTM C618 also requires that the loss of ignition (LOI) for Class F fly ash be less than 6%. The LOI criterion is intended for fly ash used in concrete, and for various design purposes, target values for \( q_u \) may be required. For instance, [12] stated that \( q_u \) of approximately 72 kPa is required to support the hoof of a walking 680-kg cow. By performing laboratory strength testing of specimens compacted with known attributes (compactive effort, \( w \), and \( c \)), \( q_u \) can be correlated to these attributes to establish what combinations of attributes will generate compacted materials with adequate strength to satisfy design criteria.

Therefore, the thrust of this study was to quantify relationships between these attributes and \( q_u \) to identify suitable combinations for achieving a given target \( q_u \) in compacted, cement-stabilized Class F fly ash. The initial impetus for this study was the development of quality assurance criteria for the construction of feedlot pads in the agriculture industry using compacted cement stabilized fly ash [7]. However, the results of this study can be used for other applications where the recycling of larger volumes of fly ash is desired.

### 2. Experimental work

#### 2.1. Materials

Fly ash obtained from the combustion of coal mined in the Appalachian region of Kentucky at the Kentucky Utilities Tyrone Power Plant near Lexington was combined with portland cement and water, compacted, and strength tested to assess the effects of compaction effort, \( w \), and \( c \) on \( q_u \). The fly ash used had a specific gravity of solids \( (G_s) \) of approximately 2.24, and was composed primarily of \( \text{SiO}_2 \) and \( \text{Al}_2\text{O}_3 \) with other constituents (Table 1). Since the \( \text{SiO}_2 + \text{Al}_2\text{O}_3 + \text{Fe}_2\text{O}_3 \) content was greater than 70% and the \( \text{SO}_3 \) content was greater than 5%, the fly ash is defined as Class F fly ash according to American Society for Testing and Materials (ASTM) test standard C618.

#### Table 1

Composition of representative samples of materials used for this study

<table>
<thead>
<tr>
<th>Constituent</th>
<th>% by Weight (fly ash specimen 1)</th>
<th>% by Weight (fly ash specimen 2)</th>
<th>% by Weight (portland cement*)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO$_2$</td>
<td>54.45</td>
<td>57.63</td>
<td>20.9</td>
</tr>
<tr>
<td>Al$_2$O$_3$</td>
<td>26.67</td>
<td>24.73</td>
<td>5.2</td>
</tr>
<tr>
<td>Fe$_2$O$_3$</td>
<td>6.91</td>
<td>7.78</td>
<td>2.3</td>
</tr>
<tr>
<td>CaO</td>
<td>2.39</td>
<td>1.76</td>
<td>64.4</td>
</tr>
<tr>
<td>K$_2$O</td>
<td>2.03</td>
<td>2.02</td>
<td>–</td>
</tr>
<tr>
<td>TiO$_2$</td>
<td>1.64</td>
<td>1.57</td>
<td>–</td>
</tr>
<tr>
<td>MgO</td>
<td>1.12</td>
<td>0.81</td>
<td>2.8</td>
</tr>
<tr>
<td>P$_2$O$_5$</td>
<td>0.25</td>
<td>0.32</td>
<td>–</td>
</tr>
<tr>
<td>Na$_2$O</td>
<td>0.21</td>
<td>0.19</td>
<td>–</td>
</tr>
<tr>
<td>SO$_3$</td>
<td>0.10</td>
<td>0.11</td>
<td>2.9</td>
</tr>
<tr>
<td>Barium</td>
<td>0.12</td>
<td>0.11</td>
<td>–</td>
</tr>
<tr>
<td>Strontium</td>
<td>0.12</td>
<td>0.11</td>
<td>–</td>
</tr>
<tr>
<td>Zirconium</td>
<td>0.05</td>
<td>0.05</td>
<td>–</td>
</tr>
<tr>
<td>Vanadium</td>
<td>0.02</td>
<td>0.02</td>
<td>–</td>
</tr>
<tr>
<td>Copper</td>
<td>0.02</td>
<td>0.02</td>
<td>–</td>
</tr>
<tr>
<td>Zinc</td>
<td>0.01</td>
<td>0.01</td>
<td>–</td>
</tr>
<tr>
<td>Manganese</td>
<td>0.01</td>
<td>0.03</td>
<td>–</td>
</tr>
<tr>
<td>Arsenic</td>
<td>0.01</td>
<td>0.01</td>
<td>–</td>
</tr>
<tr>
<td>Nickel</td>
<td>0.01</td>
<td>0.01</td>
<td>–</td>
</tr>
<tr>
<td>Chromium</td>
<td>0.01</td>
<td>0.01</td>
<td>–</td>
</tr>
<tr>
<td>Others</td>
<td>3.84</td>
<td>2.70</td>
<td>1.5</td>
</tr>
</tbody>
</table>

* Typical Type I portland cement [14].
where excessive uncombusted coal may lead to air
entrainment problems and problems with the performance
of concrete. However, the application described in this
manuscript is different, so uncombusted coal was not
considered to be a critical issue and LOI testing was not
performed.

Grain size distribution data for representative samples of
the fly ash are illustrated in Fig. 1. Dry sieve (ASTM D422),
hydrometer (ASTM D422), and wet sieve (ASTM D1140)
methods were used to develop a composite gradation curve.
With respect to the ASTM C618 criterion that more than
66% of particles be smaller than 45 μm, the gradation curve
passes through the 45 μm – 66% point (indicated by an
open square) and satisfies this criterion.

Type I portland cement was used as a cementing agent
due to its ready availability and rapid hydration rate.
Portland cement has a \( G_s \) of 3.15, and the composition
of portland cement is included in Table 1. Cement contents
between 0 and 15% were initially used to assess the effect of
c on the strength of the material.

### 2.2. Compaction methods

Fly ash was combined with cement and water and
compacted in the laboratory using standard and modified
proctor compaction effort as described in ASTM D698 and
D1557, respectively. The tests consist of compacting
material inside a cylindrical mold with height and diameter
of 11.6 and 10.2 cm, respectively, using a weight dropped
from a fixed height a prescribed number of times (Table 2).

As indicated in Table 2, the modified proctor test
responds to a larger compaction effort. The proctor
compaction methods are commonly specified laboratory
methods for geotechnical and construction purposes, and
correspond reasonably well to the compaction effort
achieved using typical earthwork construction equipment.
When compaction energy is increased, \( \rho_{\text{dmax}} \) increases,
while \( w_{\text{opt}} \) decreases.

### 2.3. Strength testing methods

To measure \( q_u \) in the laboratory, unconfined compressive
strength testing was performed in accordance with
ASTM D5102. This test consists of placing the compacted
specimen without confinement in a load frame, and
loading at a constant strain rate to failure. Unconfined
compressive strength (\( q_u \)) is defined as the peak load
divided by the cross-sectional area of the cylindrical
specimen.

For this study, it was expected that \( w_{\text{opt}} \) would fall in
the range of 25–35% based on compaction curves for
similar materials presented by [11]. It was also expected
that for a given compaction effort and \( c \), maximum
strength would be obtained near \( w_{\text{opt}} \). Since the fly ash
came from the stockpile of the power plant at \( w \) of around
50%, it was oven-dried in the laboratory, and water was
subsequently added to the material to achieve a target \( w \).
For a given \( c \) and compaction effort, 4–7 specimens were
compacted over a range of \( w \) to encompass the anticipated
value for \( w_{\text{opt}} \), and each specimen was strength tested.
Specimens were allowed to cure for 7 days before
strength testing. Note that values for \( w \) reported herein
represent \( w \) at the time of compaction, which is presumed
to decrease during hydration of the cement. During
curing, the specimens were placed in sealed plastic bags
to prevent moisture loss.

As mentioned above, it was expected that the \( w_{\text{opt}} \) for the
fly ash would be in the 25–35% range based on previous
work with similar materials. However, fly ash at the Tyrone
Power Plant was transported to a detention pond by slurry
and subsequently stockpiled after a period of dewatering, so
material from the stockpile was typically at \( w \) closer to 50%.
While drying the fly ash and compacting near \( w_{\text{opt}} \) was
expected to maximize strength for a given \( c \), this approach
may not be practical in the field because of the time required
to dry the material. Therefore, a second set of tests was
performed to measure \( q_u \) of compacted fly ash with \( w \) of
50% to simulate the practice compacting the fly ash-cement
mixture without drying the material back to \( w_{\text{opt}} \). For these
tests, specimens were compacted using modified proctor
effort with \( c \) ranging from 7.5 to 30%. These specimens
were also allowed to cure for 30, 60, and 90 days to assess
whether or not significant strength gain could be expected in
the long term.
3. Results

3.1. Overview

As described in Sections 2.2 and 2.3, there were two phases of this research. In Phase 1, the fly ash was oven dried, water and portland cement were added, compaction curves were developed to identify $w_{\text{opt}}$, and strength testing was performed on each compacted specimen. For Phase 2, fly ash with $w$ of 50% was used to simulate the as-is stockpile material, and larger amounts of portland cement were added. The two phases represent a construction tradeoff of time versus money: Phase 1 equates to cheaper construction in terms of less cement, while Phase 2 equates to cheaper construction in terms of less time.

3.2. Phase 1 testing

Fig. 2 shows compaction curves for the cement-stabilized fly ash along with the zero air void (ZAV) curve, which corresponds to a degree of saturation of 100%. To calculate the ZAV curve, $G_s$ of the fly ash-cement mixture was calculated as a weighted average of $G_s$ of the fly ash (2.24) and $G_s$ of portland cement (3.15). For $c$ of 0, 5, 10, and 15%, $G_s$ of 2.24, 2.29, 2.33, and 2.38, respectively, was used to calculate the ZAV curve. As expected, the compaction curves are concave-downward, with $w_{\text{opt}}$ decreasing and $\rho_{\text{max}}$ increasing with increasing compaction effort. Optimum water content of cement-stabilized fly ash varied between 20–30%, which is less than the 50% value typically found in the stockpile. Maximum dry density varied between 1,070 and 1,410 kg/m$^3$, which highlights a unique feature of compacted fly ash. Since $G_s$ of fly ash is less than $G_s$ of soil (which is typically in the 2.65–2.80 range), $\rho_d$ of fly ash is also less. In contrast, $\rho_d$ for compacted soil is typically in the range of 1,760–2,080 kg/m$^3$. This makes compacted fly ash an attractive alternative to compacted soil with respect to reducing settlement in underlying layers.

Strength testing was performed on each specimen after a curing period of seven days, and results from strength testing are presented in Fig. 3. These results indicate that maximum strength was achieved at $w_{\text{opt}}$ as expected. When plotting $q_u$ at $w_{\text{opt}}$ versus $c$, an approximate linear relationship exists (Fig. 4), and the modified proctor specimens were stronger than the standard proctor specimens. These results indicate that significant strength can be achieved with the addition of relatively small amounts of portland cement when the material is compacted at $w_{\text{opt}}$.

3.3. Phase 2 testing

The Phase 1 tests were conducted by drying the ash and subsequently adding water to vary $w$. As indicated in Fig. 3,
was maximized in cement-stabilized fly ash near $w_{opt}$, which was between 20–30%. However, stockpiled fly ash may have $w$ of 50% or more prior to addition of cement, and it may not be practical to dry the fly ash prior to compaction. Therefore, a second phase of tests was conducted using fly ash with $w$ of 50% prior to cement addition, and $c$ as high as 30%. Specimens were compacted using modified proctor effort, cured for 30, 60, or 90 days, and strength tested. Note that when cement is added, $w$ of the wet fly ash decreases by a factor of $(1 - c)$. Thus, when $c$ of 7.5, 15, and 30% was used, $w$ decreased from 50 to 46, 42, and 35%, respectively.

Results of the Phase 2 testing are illustrated in Fig. 5, along with results for $q_u$ measured at $w_{opt}$ during the Phase 1 tests. These results indicate that when $w = 50\%$ prior to addition of cement, $c$ must be increased by a factor of about 2.5–3 to achieve a level of $q_u$ comparable to $q_u$ observed when $w = w_{opt}$. These results also indicate that most of the strength develops within 30 days of initial hydration, and relatively little additional strength is gained afterwards.

Note that the results obtained with a curing period of 60 days and $c$ of 30% shown in Fig. 5 are anomalous compared to the other data. When preparing specimens for strength testing, it is important to achieve thorough mixing of the fly ash, cement, and water. However, if the materials are not adequately mixed, heterogeneity in the compacted specimen may lead to the creation of a weak zone in the specimen. This is a possible cause of the low measured strength of the specimen.

### 4. Discussion

As indicated in the previous section, there are two approaches for compacting cement-stabilized fly ash that...
represent a cost tradeoff of materials versus time. Portland cement costs are approximately $0.11/kg. For material compacted with $cZ10\%$ using modified effort at $w_{opt}$ with $r_d$ of 1,380 kg/m$^3$ (see Fig. 2), the portland cement cost would be approximately $15/m^3$. This would, of course, require that the material be dried prior to compaction. If a decision were made to compact the specimen using fly ash with initial $w$ of 50% (closer to the stockpile $w$) and increase portland cement content by a factor of 2.5 to achieve comparable strength, cement costs would increase to approximately $37/m^3$. This cost is significant, but should be compared to the cost in construction time associated with drying the fly ash. While the construction time cost is more difficult to quantify, factors such as ambient temperature, precipitation, and efforts associated with tilling the fly ash during drying would all have an impact on the overall time required to dry the fly ash.

If it is assumed that the fly ash from the stockpile is wet of $w_{opt}$, the data from Phase 1 and Phase 2 can be combined to provide guidance with respect to how dry the fly ash should be for a given $c$ to achieve a given $q_u$. These data are combined in Fig. 6. In Fig. 6, data points from Phase 1 wet of $w_{opt}$ are combined with data points from Phase 2 (corresponding to the ‘$w = 50\%$ prior to addition of cement’ line) with a 30-day curing period. Unconfined compressive strength is plotted versus $c$, and contours of $w$ are superimposed. This plot can be used to determine, for a given $c$, how dry the fly ash should be to achieve a target $q_u$. For example, if a target $q_u$ of 3,000 kPa is desired using $c = 15\%$, $w$ of the fly ash-cement mixture should be approximately 26%. Since addition of cement reduces $w$ by a factor of $(1 - c)$, the wet fly ash should be dried until $w = 31\%$ such that addition of the cement further reduces $w$ to 26% (Fig. 7). Figure 6 also indicates that increasing $c$ has a two-fold effect to increase strength: (1) more cementation occurs because there is more cement, and (2) $w$ decreases and the compacted material is closer to $w_{opt}$. This plot also indicates that relatively little strength is gained by drying to 30%, but significant additional strength is gained by continued drying.

The plot shown in Fig. 6 is valid for material compacted using compaction effort that is approximately equal to modified proctor effort. Experience may provide some guidance as to what combination of equipment and number of passes will provide compaction effort that is equivalent to standard or modified proctor effort, but this is still a rough estimate. Regardless of compaction effort, it has been observed that $q_u$ is higher at or below $w_{opt}$ than above $w_{opt}$. Therefore, an approach developed for compacted soil can be adopted for compacted cement-stabilized fly ash that allows $q_u$ to be estimated independent of compaction effort. [13]

Fig. 5. Unconfined compressive strength versus $c$ for specimens compacted at $w_{opt}$ (Phase 1), and for specimens compacted at $w = 50\%$ prior to compaction (Phase 2) using modified proctor effort.

Fig. 6. Unconfined compressive strength versus $c$, with contours of constant $w$ superimposed, for specimens compacted using modified proctor effort.

Fig. 7. Table for estimating required $w$ prior to addition of cement to achieve a target $w$ after addition of cement (example: to achieve $w$ of 26% for fly ash-cement mixture with $c = 15\%$, $w$ of the wet fly ash from stockpile should be 31%).
the compaction curves to identify, for a given target $q_u$, acceptance windows of $r_d$ and $w$ that would result in $q_u$ that meets or exceeds the target $q_u$. Since $r_d$ and $w$ are measured in the field as part of construction quality assurance (CQA) testing, the approach provides a way to indirectly estimate $q_u$ of in situ compacted material based on field measurement of $r_d$ and $w$.

The approach presented by Daniel and Wu [13] was applied to the compacted cement-stabilized fly ash data obtained during Phase 1 of this study as shown in Fig. 8. These data show that for a given $c$, $q_u$ can be predicted in fly ash if $r_d$ and $w$ are known. For example, if $c = 10\%$, and $w$ and $r_d$ are measured in a field compacted material and found to be 25% and 1,300 kg/m$^3$, respectively, Fig. 7 indicates that $q_u$ is approximately 2,300 kPa. This approach provides a methodology to estimate $q_u$ in compacted cement-stabilized fly ash that is independent of compaction effort.

There is some variability in fly ash composition as indicated in the two test specimens described in Table 1. The two specimens were recovered and tested several months apart, and indicate that variability at this particular plant may be relatively minor. However, it may be reasonable to expect more variability at other power plants, particularly if coal is obtained from different regions and/or geologic deposits. This variability may have some effect on the measured strength, particularly if fly ash with significantly different amounts of quicklime is used, as fly ash with more quicklime will have greater strength.

The results presented herein were obtained using Class F fly ash due to its local availability, and Class C fly ash was not tested. Class C fly ash contains a sufficient amount of quicklime to be self-cementing, and does not require the addition of a cementing agent such as portland cement to achieve significant strength. However, it is likely that the addition of portland cement to Class C fly ash would provide additional strength, and accelerate strength gain because portland cement hydrates faster than quicklime. Class F fly ash, on the other hand, does not gain significant strength unless a cementing agent (either portland cement, quicklime, or hydrated lime) is added.

5. Conclusions

Compaction of Class F fly ash combined with portland cement produces a high-strength material, and use of this material should be considered as an alternative to landfill disposal when consumption of large amounts of fly ash is desirable. For a given application, strength criteria in terms of $q_u$ may be specified. The results presented herein indicate

![Fig. 8. Contours of constant $q_u$ (in kPa) superimposed over compaction curves for prediction of $q_u$ based on $c$, $w$, and $r_d$ in portland cement stabilized compacted fly ash.](image-url)
that $q_u$ of cement stabilized fly ash is affected by $c$, $w$, and compaction effort, and by knowing these parameters, $q_u$ can be predicted. Compaction of the material over a range in $w$ produces compaction curves similar to those observed in soil, with maximum $p_d$ and maximum $q_u$ observed at $w_{opt}$ in the range of 20–30%. When compacted at $w_{opt}$, $q_u$ in the range of 1,100–5,500 kPa can be achieved, depending on $c$ and compaction effort. By correlating $q_u$ to $p_d$ and $w$, a method has been presented where $q_u$ can be predicted for a given $c$ based on values for $p_d$ and $w$ measured during CQA testing. However, $w$ of fly ash in stockpiles at power plants may be 50% or more, and it may not feasible to dry completely dry the material to $w_{opt}$. For these wet materials, it was found that by increasing $c$ by a factor of 2.5 to 3, $q_u$ comparable to $q_u$ at $w_{opt}$ can be achieved. Furthermore, for material that is partially dried to somewhere between $w$ of the stockpile and $w_{opt}$, $q_u$ can also be predicted for a given $c$.

Acknowledgements

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References

Estimating the Shear Modulus of Gravelly Soils

Bobby O. Hardin¹ and Michael E. Kalinski²

Abstract: A special large-scale torsional resonant column apparatus was used to test gravelly soils. Values of small strain elastic shear modulus (G_{max}) and modulus reduction relationships [shear modulus (G) versus shear strain (γ)] were measured for specimens of uniform and graded crushed limestone gravel, graded river gravel, standard Ottawa and crushed limestone sands, and gravel–sand–silt mixtures. Measurements of G_{max} were used to modify existing three-dimensional constitutive equations for soil elasticity for application to gravelly soils. The value of G_{max} for relatively clean uniform and graded gravels was found to increase with particle size. Soils with a variety of gradations were tested to identify the particular particle size in a graded material that is effective in determining G_{max}. With respect to modulus reduction, the need to normalize both modulus and strain is demonstrated. Normalization of both modulus and strain (G/G_{max} versus γ/γ_s, where γ_s is defined as reference strain) leads to relationships that are approximately independent of stress level for a given material.

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Introduction

When predicting the dynamic response of a soil site to earthquake excitation, nonlinear soil behavior is often approximated by an equivalent linear analysis, where soil constitutive behavior is defined by the secant shear modulus (G), Poisson’s ratio (ν), and damping ratio (Idriss and Seed 1968). Since G decreases with increasing strain amplitude (γ), iteration is necessary to match G used in the linear analysis, and corresponding computed shear strains, to modulus reduction (i.e., G versus γ) relationships measured for soils. The purpose of this paper is to present and model modulus reduction relationships measured for gravelly soils.

Two separate aspects of soil behavior, both related to strain amplitude, are generally considered when modeling modulus reduction relationships for soils. For small-strain cyclic loading (γ < ∼ 0.001%), slippage at particle contacts is negligible, and soil response results primarily from elastic deformation of particle material near particle contacts. The corresponding elastic shear modulus is usually called G_{max}, and modeling G_{max} contributes to the definition of soil elasticity. Measurements of G_{max} for gravelly soils reported herein will be used to expand the applicability of the three-dimensional constitutive equations for elasticity of particulate materials originally proposed by Hardin and Blandford (1989).

Larger strains involve significant slippage at particle contacts and possibly rearrangement of particles in addition to elastic deformation of particles, constituting elastic–plastic behavior. When cyclic loading response is computed by equivalent linear analysis, elastic–plastic soil behavior is modeled in an ad hoc way by defining shear modulus reduction curves. From the point of view of constitutive equation formulation, the modulus reduction model to be presented for gravelly soils is less fundamental than the G_{max} model, because elastic and plastic strains are not considered separately. However, modulus reduction relationships are important for current practice where soil response to earthquakes is computed by equivalent linear analysis.

Estimates of G_{max} may be obtained in practice by in situ methods such as cross-hole seismic or surface wave testing. Values of G_{max} measured in situ by these tests can be used to calibrate the stiffness coefficients in the three-dimensional model for soil elasticity. The model can then be used to predict changes in G_{max} that occur with loading, resulting from changing stresses and soil density. Shear modulus reduction is not commonly defined by in situ tests, and is more typically measured in the laboratory or estimated using modulus reduction models.

Over the past 35 years, many researchers have extensively investigated modulus reduction relationships for soils through laboratory experiments using resonant column and cyclic triaxial methods (e.g., Seed and Idriss 1970; Hardin and Drnevich 1972; Seed et al. 1986; Vucetic and Dobry 1991). Most of the research has been done in sands, silts, and clays. In cohesive soils, the effects of plasticity index (PI), overconsolidation ratio (OCR), void ratio (e), number of cycles of loading (N), and mean effective confining stress (σ_0') have all been well documented. In sands, the effects of σ_0' and e on G and G_{max} are also well documented.

Unlike sands, silts, and clays, less effort has been dedicated toward understanding the dynamic behavior of gravelly soils. This is mainly due to the inherent difficulties in recovering undisturbed specimens of gravelly soils for laboratory testing, and the size requirements for test specimens expressed as a function of particle size. For instance, ASTM D4015 (ASTM 2003) requires that for resonant column testing of specimens with diameters larger than 70 mm, the largest particle size (D_{10}) should be less...
than one-sixth of the specimen diameter. Previous research, as described in the following section, has demonstrated the relationship between $G/G_{\text{max}}$ and $\gamma$ in gravelly soils, but an empirical method to synthesize modulus reduction data using basic properties of gravelly soils has not been developed.

Torsional resonant column testing of specimens of uniform and graded crushed limestone gravel, graded river gravel, standard Ottawa and crushed limestone sands, and gravel–sand–silt mixtures, including crushed limestone known as “dense graded aggregate,” were performed to identify the relationship between $G$, $g$, grain size, and soil gradation. As a result, the three-dimensional constitutive equations for soil elasticity originally developed by Hardin and Blandford (1989), and modulus reduction equations originally developed by Hardin and Drnevich (1972) for sands, silts, and clays, have been expanded to include gravelly soils.

### Previous Research to Measure Shear Modulus in Gravelly Soils

Seed and Idriss (1970) observed the relationship between $G$ and $\sigma'_0$ using laboratory cyclic triaxial testing, and expressed the relationship in the form of

$$G = 1,000 K_2 \sqrt{\sigma'_0}$$

where $G$ and $\sigma'_0$ are expressed in terms of pounds per square foot (psf), and $K_2$ is an empirical soil modulus coefficient that is a function of $e$ and $\gamma$. The value of $K_2$ at small strains ($K_{2\text{max}}$) was found to range from 30 to 75 for sands, and $K_{2\text{max}}$ was found to increase with decreasing $e$. Seed and Idriss (1970) also recognized that gravelly soils are stiffer than sands in the small-strain range (less than around 0.001%) based on in situ measurements.

Seed et al. (1986) expanded upon the Seed and Idriss (1970) study by performing cyclic triaxial testing on 30-cm-diameter specimens of gravelly soils. This study also demonstrated that $K_{2\text{max}}$ is significantly higher for gravels than for sands, and quantified the range as being between 80 and 180. Seed et al. (1986) showed that the shape of the modulus reduction curves is different for sands and well-graded gravels (Fig. 1). However, an empirical approach to estimating modulus reduction curves for gravelly soils was not presented.

### Testing Method

A large resonant column testing apparatus, shown schematically in Fig. 2, was designed to subject cylindrical specimens to torsional vibration. The specimen diameter and length were 15 and 30 cm, respectively, and specimens were confined by vacuum. This type of resonant column apparatus is called “free–free” because both ends of the specimen vibrate, and neither end is fixed. Excitation is at the bottom of the specimen and resonance is detected by motion at the top of the specimen.

The vibrating table, with arms and attached magnets (Fig. 2), is attached by means of springs to a fixed reaction frame. The outer ends of four steel springs are attached to the reaction frame, which provides a fixed boundary condition. The other ends of the springs are attached to the vibrating table. Thus, the rigid mass of the vibrating table and relatively weightless springs fixed by the reaction frame comprise a single-degree-of-freedom (SDOF) system.

The torsional stiffness of the springs, $K_0$, is determined by measuring the change in resonant frequency of the SDOF system as an additional steel mass with known polar moment of inertia $J_A$ is fixed to the vibrating table. Without the additional mass, the resonant frequency is $f_1$. With the additional mass, the resonant frequency is $f_A$. The torsional stiffness of the springs is then calculated as

$$K_0 = \left(4\pi^2 - J_A f_A^2\right) / \left(1-f_A^2/f_1^2\right)$$

The resonant frequency of the SDOF system with the bottom platen rigidly attached ready for specimen construction is $f_b$, and
the mass polar moment inertia of the lower vibrating mass during testing (including vibrating table and bottom platen), \( J_0 \), is:

\[
J_0 = K_0/(4\pi^2 \omega^2)
\]  

(3)

The mass polar moment of inertia of the top mass (including top platen, accelerometers, and other fixed hardware), \( J_t \), is computed by calculating and superimposing the mass polar moment of inertia of each of the individual parts. Values of \( K_0 \), \( J_0 \), and \( J_t \) are required to compute \( G \) from measured resonant frequency, \( f_0 \), of the complete resonant column device with soil specimen in place. The theory for computing \( G \) from \( f_0 \) is given in ASTM D4015, the standard for resonant column testing (ASTM 2003), and is also presented by Drnevich et al. (1978).

Measurements of \( G_{\text{max}} \) with a small resonant column device were made for two specimens of crushed limestone sand and one specimen of Ottawa sand. This device, called the “Hardin oscillator” (Richart et al. 1970), is also a SDOF system before a specimen is attached. The Hardin oscillator is similar in concept to the SDOF part of the apparatus in Fig. 2, except that it attaches to the top of the specimen while the bottom of the specimen is fixed. This smaller vibration device and specimen were placed within a pressure chamber.

**Testing Program**

Fourteen specimens were tested in the large resonant column apparatus shown in Fig. 2, and three were tested with the small apparatus. Specimen materials are identified in Fig. 3. Particle size distribution curves are shown, and specimen size, void ratio, percent saturation, and water content are listed for each specimen, along with an icon illustrating the resonant column device that was used for each specimen.

**Modeling the Elastic Shear Modulus \( G_{\text{max}} = G_{ij}^* \) for Gravelly Soils**

Three-dimensional incremental constitutive equations defining the elasticity of particulate materials were formulated by Hardin and Blandford (1989). However, the proposed constitutive matrix was not symmetric. In this paper, the off-diagonal elements of the Hardin and Blandford matrix are modified to make the matrix symmetric as required by the laws of thermodynamics. Each symmetric pair of off-diagonal elements in the modified matrix is obtained from the square root of the product of the same pair in the Hardin and Blandford (1989) equations.

For the case where principal axes of stress and principal axes of soil fabric coincide, the relationship between the stress increment vector \( \{d\sigma_i\} \) and elastic strain increment vector \( \{d\varepsilon_i\} \) is defined as

\[
\begin{align*}
\{d\varepsilon_1\} & = \frac{F(e)}{OCR} \rho_a^{1/2} \\
\{d\varepsilon_2\} = \frac{C_{11} + \nu^e C_{12} - \nu^e C_{13}}{2(1 + \nu^e) C_{12}} \\
\{d\varepsilon_3\} = \frac{-\nu^e C_{21} + C_{22} - \nu^e C_{23}}{2(1 + \nu^e) C_{22}} \\
\{d\varepsilon_4\} = \frac{\nu^e C_{31} - \nu^e C_{32}}{2(1 + \nu^e) C_{33}}
\end{align*}
\]  

(4a)

where

\[
C_{ij} = \frac{1}{(S_i dS_j/\sigma_{ij})^{1/2}}
\]  

(4b)

\[
C_{ij}^* = \frac{1}{S_i (\sigma_i^e \sigma_j^e)^{1/2}}
\]  

(4c)

\[
F(e) = 0.3 + 0.7e^2
\]  

(4d)

In Eqs. (4a)–(4d), \( k \) depends on PI; \( p_a \) = atmospheric pressure; and \( \nu^e \) = elastic Poisson’s ratio. The elastic constant \( n \) is approximately 0.5 for soil, and is analogous to the one-third power in the constitutive equations for assemblies of elastic particles. The \( S \) terms are elastic stiffness coefficients.

Because particulate materials yield during unloading, exclusively elastic behavior is limited to infinitesimal increments of unloading, providing that creep strains are eliminated (Hardin and Blandford 1989). Elastic properties of particulate materials are isolated by measuring limiting values of stiffness for cyclic loading at small strains, where energy dissipation (hysteresis) is essentially zero.

Eqs. (4a)–(4d) is the result of synthesis of about 30 years of
research results including tests of a variety of uncedmented sands, silts, and clay soils, and it has been shown to approximate the behavior of other particulate materials such as wheat (Ziołkowski et al. 1985). In formulating Eqs. (4a)–(4d), the objective was to explicitly account for as many factors as possible to make the stiffness coefficients $S_{ij}$ approximately constant. Values of $S_{ij}$ for clean sands with different gradations, particle shapes and mineral contents vary from approximately 1.200 to 1.600. Values of $S_{ij}$ measured in the laboratory for silty sands, silts, and clays, including undisturbed samples of low and high plasticity cohesive soils, sensitive clays, and remolded specimens of pure silt, kaolinite, or bentonite, vary approximately from 700 to 2,000 (Hardin 1978). Values of $S_{ij}$ increase with time under constant effective stress for soils that contain even a small percentage of silt and/or clay particles. After long periods of in situ aging, $S_{ij}$ may be expected. The scalar functions $F(e)$ and $OCR^4$ account for effects of void ratio and preconsolidation.

For Eqs. (4a)–(4d) to represent the behavior of gravelly soils with approximately constant $S_{ij}$, it is necessary to add a new scalar function $f(D)$ to account for effects of particle size when grain size (D) is larger than sand size. The necessary modification of Eqs. (4a)–(4d) and the nature of the function $f(D)$ is based on measurements of elastic shear moduli, which are defined in Eq. (4a) ($G_{ij}'=dF_{ij}'/dy_{ij}'$). Insertion of the proposed particle size function $f(D)$ into Eqs. (4a)–(4d) yields:

$$G_{ij}' = \frac{OCR^4 f(D) Y_{ij}}{F(e)} S_{ij} \left[1 + d_{ij}^P + (\alpha_n' \sigma_n^P)^{(n-1)/2} \right]$$

where $f(D)=1$ for sands, silts, and clays. The model for sand, silt, and clay soils will be referred to herein as “baseline behavior.”

### Analysis of Test Results and Modeling of $G_{max}$

The objective of this section is to extend the soil elasticity model [Eqs. (4a)–(4d) and (5)] to gravelly soils. This requires definition of the proposed particle size function $f(D)$. The proposed form of the function $f(D)$ is based on test results for clean uniform limestone sand and gravels in Fig. 4, which shows a log–log plot of $F(e)G_{ij}'$ versus $(\alpha_n' \sigma_n^P)^{(n-1)/2}$ for tests of four different uniform gradations of crushed limestone. Particle size distribution curves for the four materials are shown at the top of Fig. 4. The largest particle size is 19 mm, and particle sizes for the sand specimen are between 0.6 and 1.18 mm.

The heavy dashed straight line in Fig. 4, approximating the baseline behavior of uniform limestone sand, is defined by Eq. (5) with $S_{ij}=1,600$, $\nu=0.1$, $OCR^4=1$, $n=0.5$, and $f(D)=1$. Specimens of uniform sand (materials 1, 15, and 16 from Fig. 3), indicated by dashed lines with triangle symbols, are plotted for comparison, and agree well with the baseline curve. This representation of sand behavior serves as a baseline for the definition of $f(D)$ by considering results for uniform gravel specimens. The test data for uniform gravels show a tendency to approach sand behavior as stress level increases, up to about one atmosphere (the gravels were confined by vacuum). At extremely high stress levels, where significant particle crushing has occurred, it is reasonable to expect that gravel behavior would approach that of sand as particle sizes are reduced by crushing. This kind of behavior is modeled by

$$f(D) = \frac{n_s}{30} \left( \frac{\sigma_n^P}{P_s} \right)$$

where $f_0$=constant that depends on particle size distribution and $n_s$=shape number that quantifies particle shape (Table 1).

The shape numbers in Table 1 were introduced by Hardin (1985) in an analysis of crushing of soil particles, and were also used to model one-dimensional strain in cohesionless soils (Hardin 1987). Test results presented herein for angular crushed limestone ($n_s=25$) and rounded river gravel ($n_s=15$) indicate their potential usefulness in modeling other soil properties.

Baseline behavior for sand, silt, and clay soils is defined by $f_0=0$, which corresponds to $f(D)=1$ based on Eq. (6). The behavior of each of three uniform gravels (materials 2, 3, and 4 from Fig. 3) is approximated in Fig. 4 by a heavy solid line defined by Eq. (6) with $n_s=25$. Values of $f_0$ are 0.43, 0.91, and 1.22, for particle size ranges of 2.36–4.75, 4.75–9.5, and 12.5–19 mm, respectively.

Having shown that $f_0$ increases with particle size for uniform materials, the next step is to identify the specific particle size in a graded material that is effective in determining the value of $f_0$. The test results in Fig. 5 are for graded crushed limestones (materials 6, 7, and 8 from Fig. 3) with the three different particle size distributions shown at the top of Fig. 5. Baseline behavior of crushed limestone sand ($S_{ij}=1,600$, $\nu=0.1$, and $n=0.5$) is defined by the heavy dashed line. Heavy solid lines approximating the data are defined by Eq. (6) with $f_0=0.28, 0.52$, and 0.67, where $f_0$ increases with particle size.

Test results for two different gradations of river gravel (materials 10 and 11 from Fig. 3) are presented in Fig. 6 as solid lines. Baseline behavior for materials with round particles is defined by results for Ottawa sand (materials 9 and 17), which are represented by triangle symbols. The heavy dashed straight line ap-

### Table 1. Definition of Shape Number

<table>
<thead>
<tr>
<th>Particle shape</th>
<th>$n_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angular</td>
<td>25</td>
</tr>
<tr>
<td>Subangular</td>
<td>20</td>
</tr>
<tr>
<td>Subrounded</td>
<td>17</td>
</tr>
<tr>
<td>Round</td>
<td>15</td>
</tr>
</tbody>
</table>

Fig. 4. $G_{max}$ versus stress level for clean uniform crushed limestone sand and gravels
proximating the behavior of Ottawa sand is defined by Eq. (5) with $S_3 = 1,300, \nu' = 0.1, \text{OCR}^2 = 1, n = 0.5,$ and $f(D) = 1$. The heavy solid lines approximating the behavior of two graded river gravel materials are defined by $f_0 = 0.84$ and $0.97$, with $n_s = 15$. Values of $f_0$ are greater than zero, i.e., $f_s = 1$ for uniform and graded, angular and rounded, gravelly soils shown in Figs. 4–6. Test results will now be presented to show that soils containing large amounts of gravel, but that also contain fines, may exhibit behavior similar to the sand, silt and clay category, where $f_0 = 0$ and $f_s = 1$. Data for four gravel-sand-silt mixtures (materials 5, 12, 13, and 14 from Fig. 3) are presented in Fig. 7. Particle size distribution curves are shown at the top of Fig. 7. The material identified by a “+” symbol (material 5 from Fig. 3) is a well-graded crushed limestone often referred to as “dense graded aggregate.” The nature of the particles for the other three materials, with the exception of size distribution, is unknown. The heavy solid line in Fig. 7 is assumed to represent baseline behavior as defined by Eq. (5) with $S_3 = 1,400, \nu' = 0.1, \text{OCR}^2 = 1, n = 0.5,$ and $f(D) = 1$. Shear moduli for these four materials are at or below baseline behavior notwithstanding the fact that they contain large fractions of gravel sized particles. The percentages of particles that are finer than 1 mm varies from approximately 19 to 62%, indicating that the amount of finer material (sand, silt, or clay) necessary to produce behavior similar to baseline behavior is less than or equal to 19%.

Parenthetically, the materials in Fig. 7 contain small amounts of moisture. Possible existence of matric suction was neglected in computing effective stresses. The inclusion of matric suction would increase effective stress and shift plotted points toward the right, making data fall further below the baseline curve.

The variation of $f_0$ with particle size distribution for gravelly soils is shown in Fig. 8. For each material a horizontal line is drawn at $f_0$, extending from $D_0$ to $D_{100}$, to make a judgment with respect to the specific particle size that is effective in determining $f_0$. The data presented in Fig. 7 for gravel–sand–silt mixtures indicate that the effective size is less than the particle size corresponding to 19% passing ($D_{19}$). Data in Fig. 8 do not precisely define the effective diameter, but the particle size corresponding to 5% passing ($D_5$) has been chosen and identified by symbols.

The heavy solid line approximating the variation of $f_0$ with $D_5$ is defined as

$$f_0 = f_0^\text{max} \frac{\alpha \left( D_5 - D_\text{sand} \right) \gamma_w}{p_a} \frac{p_a}{1 + \alpha \left( D_5 - D_\text{sand} \right) \gamma_w}$$

where $f_0^\text{max} = 1.6, \alpha = 2.800,$ and $D_\text{sand} = 1$ mm. The ratio $p_a/\gamma_w$, where $\gamma_w =$ unit weight of water, can be used in empirical equations to normalize parameters that have length units. Its use in Eq. (7) renders $\alpha$ dimensionless.

**Fig. 5.** $G_{\text{max}}$ versus stress level for graded crushed limestone materials

**Fig. 6.** $G_{\text{max}}$ versus stress level for Ottawa sand and two gradations of river gravel

**Fig. 7.** $G_{\text{max}}$ versus stress level for four gravel–sand–silt mixtures
Modeling the Reduction of Secant Shear Modulus with Strain for Gravely Soils

Nonlinear soil behavior is often approximated using an equivalent linear analysis procedure, where by iteration, $G$ used in the analysis and computed strains are made to match modulus reduction relationships measured for soils. Shear modulus reduction curves used for these analyses are often normalized by defining $G/G_{\text{max}} = G/G_0$ versus $\gamma$ without normalizing strain (Kramer 1996).

Values of $G/G_{\text{max}}$ for graded crushed limestone gravel (material 8 from Fig. 3) are plotted versus $\gamma$ in Fig. 9(a), where the data for each of four stress levels can be approximated by a single curve. The reference strain parameter $\gamma_r$ used to normalize strain in Fig. 9(b), was proposed by Hardin and Drnevich (1972) in their hyperbolic model for shear modulus reduction:

$$
\frac{G}{G_{\text{max}}} = \frac{1}{1 + \gamma_r \left[ 1 + a \exp(-b\gamma_r^c) \right]^{n}}
$$

where

$$
\gamma_r = \frac{\tau_{\text{max}}}{G_{\text{max}}}
$$

The maximum shear stress parameter $\tau_{\text{max}}$ represents the shear strength of the soil for the stress path being applied. Since pore pressures do not develop for dry or nearly dry specimens of gravelly soils, $\tau_{\text{max}}$ can be computed from the major and minor initial effective principal stresses ($\sigma'_1$ and $\sigma'_2$), and the effective friction angle ($\phi'$):

$$
\tau_{\text{max}} = \frac{1}{2} \sqrt{(\sigma'_1 + \sigma'_2) \sin^2 \phi' - (\sigma'_1 - \sigma'_2)^2}
$$

Eq. (9) with estimated $\phi'$ of 40° was used to normalize strain in Fig. 9(b).

It is instructive to consider the effect of isotropic states of stress (i.e., $\sigma'_1 = \sigma'_2 = \sigma'_0$) in Eqs. (5) and (9) on $\gamma_r$. Substitution of these stresses into Eqs. (5) and (9) with $n=0.5$, and then into Eq. (8b), yields:

$$
\gamma_r = \frac{F(c)}{OCR^2 f(D)} - \frac{2(1 + \nu')}{S_{ij}} \sin \phi' \sqrt{\frac{\sigma'_0}{p_a}}
$$

Eq. (10) demonstrates that $\gamma_r$ increases with the square root of initial effective stress for isotropic confinement. Values of $\gamma'_r$ can be used as an approximation for anisotropic states of stress, and $(\sigma'_0/p_a)^{1/2}$ can be used to normalize strain as shown in Fig. 9(c), where the scatter is slightly more than in Fig. 9(b).

Use of $\gamma_r$ for normalization of strain should apply more generally to all soils and conditions because the evaluation of $\tau_{\text{max}}$ may be expected to account for stress path differences and for effects of the cohesion component of strength. Use of $(\sigma'_0/p_a)^{1/2}$ is attractive because of its simplicity.

The development of pore water pressure with loading along a stress path should be considered when computing $\tau_{\text{max}}$. A relationship similar to Eq. (9) must be developed for the stress path being followed. Additionally, when pore pressures increase with cycles of loading in an equivalent linear analysis, values of $\tau_{\text{max}}$ and $\gamma_r$ should be adjusted before computing new values of $G$.

Shear modulus reduction curves measured for twelve different gravelly soils and two sands are presented in Fig. 10. Curves are shown for four different stress levels. For the large resonant column test, stress was applied by means of vacuum, with additional axial stress (10–11 kPa) produced by the weight of the apparatus attached to the top of the specimen and the weight of the speci-
men itself. For the small resonant column test, the weight of the apparatus was supported by a counterweight and the weight of the specimen was negligible, making the state of stress approximately isotropic. The symbols for each of the combinations of $\sigma_3/\rho_a$ and $\sigma_1/\rho_a$ are identified in Fig. 10. Particle size distribution curves are shown for each material. The four curves for each material are normalized with respect to shear modulus, but not with respect to shear strain in Fig. 11. Normalization of shear modulus without normalizing shear strain results in four distinct curves for each material. The relationship between $G/G_{\text{max}} = G/G_{ij}$ and $g/g_r$ for a given material clearly depends on stress level. In Fig. 12, the data are normalized with respect to $g_r$ and $G_{\text{max}}$, and reference strain is calculated using Eqs. (6a)–(6d) with estimated values of $f_s$. Values of $f_s$ were not measured. However, values are estimated in 5°, 35°, and 40° estimates. The $G/G_{\text{max}} = G/G_{ij}$ versus $g/g_r$ relationship for a given material is nearly independent of stress level.

Normalized shear modulus versus normalized shear strain relationships are approximated by a single dashed curve for each material in Fig. 12. These curves are defined by Eqs. (8a) and (8b) using the values of $a$ and $b$ shown in Fig. 12. Values of $b$ vary from 1 to 2. Twelve of the fourteen dashed curves are defined by $b=1$. The lowest value of $a=0.3$ is for Ottawa sand. Values of $a$ range from 1.1 to 2.5 for clean crushed limestone gravels (both uniform and graded materials), and the highest values of $a$ range from 5.7 to 6.2 for gravel–sand–silt mixtures, including the dense graded aggregate.

**Summary and Conclusions**

A special large-scale torsional resonant column test has been used to test gravelly soils. Values of $G_{\text{max}}$ and modulus reduction relationships were measured for specimens of uniform and graded crushed limestone gravel, graded river gravel, standard Ottawa and crushed limestone sands, and gravel–sand–silt mixtures including dense graded aggregate.

Three-dimensional constitutive equations for elasticity of particulate materials developed by Hardin and Blandford (1989) are modified herein to make the elastic constitutive matrix symmetrical [Eqs. (4a)–(4d)]. Measurements of $G_{\text{max}}$ for gravelly soils have been used to add the particle size function $f_sD_d$, defined by Eq. (6), to these equations in order to extend their applicability to gravelly soils.

The value of $G_{\text{max}}$ for relatively clean uniform and graded gravels increases with particle size. Soils with a variety of gradations were tested in an effort to determine the particular particle size in a graded material that is effective in determining $G_{\text{max}}$. Analysis of the data indicates that $D_5$ can be used as an approximate effective diameter. The behavior of materials with large gravel content, but that contain relatively small amounts of sand, silt, and/or clay, is similar to the sand–silt–clay category with $f(D)=1$. Tests of very angular crushed materials and materials with round particles have isolated the effect of particle shape on $f(D)$.

With respect to modulus reduction, the need to normalize both $G$ and $\gamma$ is demonstrated. Relationships normalized with respect to modulus but not with respect to strain vary with stress level.
The value of $\gamma_r$ for cohesionless soils subjected to isotropic states of stress increases with the square root of effective stress. As an approximation, strain can be normalized using $(\sigma'_i/p_s)^{1/2}$ rather than $\gamma_s$ to avoid the need to estimate $\tau_{\text{max}}$. However, the use of $\gamma_r$ should apply more generally to all soils and conditions, because evaluation of $\tau_{\text{max}}$ may be expected to account for stress path differences and for the cohesion component of strength.

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A Geophysical Approach to Construction Quality Assurance Testing of Compacted Soil Using Electrical Conductivity Measurements

Michael E. Kalinski, Ph.D., P. E.¹ and Sharath C. Vemuri²

ABSTRACT

Construction quality assurance (CQA) testing is typically performed in compacted fine-grained soils to measure in situ values of dry density ($d$) and water content ($w$). The nuclear gauge is the most common method. However, electrical conductivity ($\sigma$) measurement is an attractive alternative. Previous researchers have developed relationships between $\sigma$ and volumetric water content ($\theta$), but the effect of compaction effort has not been investigated. Other researchers have related $\sigma$ to degree of saturation ($S$), and the relationship is independent of compaction effort. However, specimens compacted at the same $S$ can possess different values for $\theta$, so $\sigma$ cannot correlate equally well to both parameters. Therefore, a laboratory study was performed to assess the effects of compaction effort, $S$, and $\theta$ on $\sigma$ in specimens of fine-grained soil. By measuring $\sigma$ over a range of $w$ and compaction effort, it was observed that $\sigma$ correlates well with $\theta$ independent of compaction effort, while the correlation between $\sigma$ and $S$ is dependent upon compaction effort. It was also observed that $\theta$ is a reasonable indicator of whether or not a soil specimen is wet or dry of $w_{opt}$, a common CQA acceptance criterion. Therefore, a new CQA method based on $\theta$ is proposed. Since $\theta$ can be predicted based on the measurement of $\sigma$, field geophysical measurement of $\sigma$ may be used to quickly determine whether or not soil has been compacted in accordance with specifications.

INTRODUCTION

Construction Quality Assurance Testing of Soils

The relationship between dry density ($\rho_d$), gravimetric water content ($w$), and the engineering properties of compacted fine-grained soils, is well understood. For a given clay and compaction effort, a compaction curve of $\rho_d$ versus $w$ can be developed by compacting specimens over a range of $w$. From the compaction curve, a maximum $\rho_d$ and corresponding optimum water content, $w_{opt}$, are identified. The properties of compacted soil generally depend upon whether $w$ is drier or wetter than

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\( w_{opt} \). Maximum strength is achieved dry of \( w_{opt} \), while minimum hydraulic conductivity \( (k) \) is achieved wet of \( w_{opt} \) (Daniel and Benson, 1990).

Compaction curves are typically developed as a result of laboratory compaction testing using the standard and modified proctor methods (ASTM D698 and D1557). However, it is difficult to equate laboratory and field compaction effort. Therefore, when soil is compacted in the field, it is practical to specify a window of \( \rho_d \) and \( w \) that will produce acceptable performance (Daniel and Benson, 1990). This acceptance window is bounded by the Line of Optimums (LOI). To confirm that soil is compacted in accordance with design specifications, a construction quality assurance (CQA) program is implemented that dictates the frequency of tests, type of tests, and acceptance criteria for in situ compacted material.

Nuclear gauge testing (ASTM D2922 and D3017) is a common method to rapidly obtain estimates for \( \rho_d \) and \( w \) for CQA purposes. Nuclear gauges can be operated in a non-intrusive “backscatter” mode or an intrusive “direct transmission” mode, although operation of the nuclear gauge in the direct transmission mode may compromise the integrity and performance of compacted clay liners (CCLs). Since nuclear gauges contain radioactive material, they must be licensed with the Nuclear Regulatory Commission. As a result, there is a significant amount of administrative burden associated with the operation and maintenance of nuclear gauges.

**The Electrical Conductivity of Soil**

The electrical conductivity of soil, \( \sigma \), is dependent upon several parameters, including volumetric water content \( (\theta) \), electrical conductivity of the pore fluid, \( \sigma_w \), electrical conductivity of the soil matrix, \( \sigma_s \), and soil texture (i.e. flow path tortuosity). Rhodes et al. (1976) expressed \( \sigma \) in undisturbed fine-grained soils as:

\[
\sigma = \sigma_w \left( a \theta^2 + b \theta \right) + \sigma_s, \tag{1}
\]

In Eqn. 1, the constants \( a \) and \( b \) are soil-specific regression coefficients. More recent research by Rinaldi and Cuestas (2002) also indicates that \( \sigma \) is a function of \( \theta \). Since \( \theta \) can be expressed in terms of \( \rho_d \) and \( w \):

\[
\theta = \frac{V_w}{V} = \frac{w \rho_d}{\rho_w}, \tag{2}
\]

\( \sigma \) is a function of both \( \rho_d \) and \( w \). In Eqn. 2, \( \rho_w \) is the mass density of water, and \( V_w \) is the volume of water in a volume of soil \( V \).

Equation 1 demonstrates that the solids and pore water in soil can be viewed as parallel conductors, and the overall conductivity of the system \( (i.e. \sigma) \) is a function of the conductivity of the two conductors \( (i.e. \sigma_w \) and \( \sigma_s) \). Since \( \sigma_w \) is higher than \( \sigma_s \), pore water plays a more significant role than mineral solids. However, the contribution of \( \sigma_w \) to \( \sigma \) in soil is complicated by the effects of flow path tortuosity,
and by the relative interconnectedness of the pore water in the soil. As a result, a soil-specific empirical relationship such as Eqn. 1 is required. Nevertheless, $\theta$ is a good indicator of $\sigma$, because it is a direct measure of the available volume of pore water $V_w$ per unit volume of soil. Values for $\sigma_s$, $a$ and $b$ reported by Rhodes et al. (1976) for typical samples of undisturbed soil are summarized in Table 1. Typical values for $\sigma_s$ may range from 0.2 mS/cm to over 50 mS/cm, depending on hardness.

### Table 1. Summary of test results obtained by Rhodes et al. (1976) in undisturbed specimens and Kalinski and Kelly (1993) in compacted specimens

<table>
<thead>
<tr>
<th>USDA Soil Classification</th>
<th>USCS Equivalent</th>
<th>$\sigma_s$ (mS/cm)</th>
<th>$a$</th>
<th>$b$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Undisturbed Soils</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fine Sandy Loam</td>
<td>SM or SC</td>
<td>0.18</td>
<td>1.38</td>
<td>-0.09</td>
</tr>
<tr>
<td>Very Fine Sandy Loam</td>
<td>SM or SC</td>
<td>0.25</td>
<td>1.29</td>
<td>-0.12</td>
</tr>
<tr>
<td>Loam</td>
<td>ML or SM</td>
<td>0.40</td>
<td>1.40</td>
<td>-0.06</td>
</tr>
<tr>
<td>Clay Loam</td>
<td>ML or CL</td>
<td>0.45</td>
<td>2.13</td>
<td>-0.24</td>
</tr>
<tr>
<td><strong>Compacted Soils</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clay</td>
<td>CH</td>
<td>0.24</td>
<td>1.04</td>
<td>-0.09</td>
</tr>
</tbody>
</table>

Values for $\sigma_s$, $a$ and $b$ reported by Kalinski and Kelly (1993) for compacted soil specimens are included in Table 1, indicating that Eqn. 1 is also valid for compacted soil. For their measurements, each specimen was compacted at a given initial $w$ and saturated, and $\theta$ was varied using a pressure membrane apparatus. Thus, $\sigma$ was measured over a range of $\theta$, but the same specimen was used for each measurement. However, the texture of compacted fine-grained soil is dependent upon the initial $w$ at the time of compaction. Since $a$ and $b$ are a measure of soil texture, Eqn. 1 was not necessarily demonstrated to be valid for comparing different soil specimens compacted over a range of $w$, although separate research by Kalinski and Kelly (1994) indicates at least qualitatively that Eqn. 1 is valid for different soil specimens compacted with different initial $w$. Nevertheless, the effects of compaction effort were not investigated.

Abu-Hassanein et al. (1996) performed similar research on compacted clays. In this research, $\sigma$ was measured as a function of degree of saturation ($S$) in laboratory specimens compacted over a range of $w$. They observed that $S$ could be predicted based on measurement of $\sigma$. The relationship was independent of compaction effort, and $\sigma$ was observed to increase with increasing $S$. Degree of saturation and $\theta$ are related by porosity, $n$:

$$\theta = Sn.$$  

(3)

Porosity can be expressed in terms of $\rho_d$ as

$$n = 1 - \frac{\rho_d}{G_s \rho_w},$$  

(4)
where \( \rho_w \) is the mass density of water and \( G_s \) is the specific gravity of soil solids. Thus, two soil specimens with the same \( S \) but different \( \rho_d \) will have different values for \( \theta \). For example, if two specimens are compacted using standard and modified proctor effort using pore water with \( \sigma_w \) of 1.0 mS/cm and \( S = 80\% \), at \( \rho_d \) of 1,770 and 2,010 kg/m\(^3\), respectively, \( \theta \) of the specimens will be 27\% and 21\%. According to Abu-Hassanein et al. (1996), they will possess the same \( \sigma \). According to Rhodes et al. (1976), they will possess values for \( \sigma \) that differ by around 9-18\%.

**DESCRIPTION OF RESEARCH STUDY**

In light of the previous discussion, it is apparent that CQA testing is necessary to compare the soil parameters \( \rho_d \) and \( w \) to construction specifications for fine-grained compacted soils. Established methods exist for performing CQA testing, but these methods possess some limitations. Electrical conductivity is an indicator of \( \theta \) in compacted soil, but the effects of compaction effort and compaction \( w \) have not been addressed. Electrical conductivity is also an indicator of \( S \) in compacted soil, and the relationship has been demonstrated to be independent of compaction effort. However, soils compacted at the same \( S \) but with different values for \( \rho_d \) will possess different values for \( \theta \), so \( \sigma \) cannot correlate equally well to both \( S \) and \( \theta \). Therefore, the thrust of this study was to assess the applicability of Eqn. 1 to soils compacted at different compaction efforts and initial \( w \), compare the \( \sigma - \theta \) and \( \sigma - S \) relationships to assess which parameter is a better indicator of \( \sigma \), and develop a practical approach to CQA testing of compacted soils using an in situ geophysical method.

For this study, a lean clay (CL) from Morehead, Kentucky was compacted using standard and modified Proctor effort (ASTM D698 and ASTM D1557). A cylindrical compaction mold with a diameter and length of 10.2 and 11.6 cm, respectively, was used. Specimens were prepared by mixing air-dried soil with water obtained from the Lexington, Kentucky municipal water supply. This water has an average electrical conductivity at room temperature, \( \sigma_w \), of 0.373 mS/cm that varies seasonally from 0.2-0.6 mS/cm (Combs, 2004). Since testing was completed within a time frame of a few months, it was assumed that \( \sigma_w \) remained relatively constant between specimens, although \( \sigma_w \) was not measured directly. Dry density (\( \rho_d \)) and \( w \) were calculated for each specimen, and compaction curves were developed.

The electrical conductivity of each specimen, \( \sigma \), was calculated by measuring the longitudinal resistance of the specimen, \( R \) using a Radio Shack Model 22-163 digital multimeter. The multimeter operates with an open-circuit voltage of 0.86 V across the leads, and a closed-circuit current of 335 \( \mu \)A. The meter has a closed-circuit resistance of approximately 0.3 ohms. This is negligible compared to the range in \( R \) measured during the study, which was on the order of hundreds to thousands of ohms. Given a cylindrical specimen of soil with cross-sectional area \( A \) and length \( L \) as shown in Fig. 1, \( \sigma \) can be calculated using the following relationship:
\[ \sigma = \frac{L}{RA} \] 

(5)

Figure 1. Measurement of \( \sigma \) in a cylindrical specimen of compacted soil

Steel wire screens were placed at each end of the specimens and used as electrodes. To achieve intimate electrical contact between the electrodes and the ends of the specimen, the specimens were placed in latex membranes with PVC end caps and subjected to pore vacuum of 10.6-37.4 kPa. Vacuum level was varied to assess its effect and identify what vacuum level was necessary to achieve intimate electrical contact. To further improve electrical contact, the ends of the specimen were moistened with salt water. As a result of this study, data regarding \( \theta \), \( S \), and \( \sigma \) were acquired over a range of compaction \( w \) and compaction effort for analysis.

When making electrical measurements in soil, it is important to take precautions to minimize electrode polarization. Typically, low-frequency alternating current (AC) measurements are made. However, direct current (DC) resistance measurements were made to allow usage of an inexpensive multimeter. DC measurements are generally considered undesirable because electrode polarization may occur, which biases the measurement. However, average \( \sigma_w \) for the specimens tested was expected to be around 0.4 mS/cm, which is equivalent to a NaCl solution with a concentration of around 0.02% (Asquith and Gibson, 1982). As indicated by Rinaldi and Cuestas (2002), the electrode polarization effect is minor at such low concentrations. Therefore, it was assumed that the electrode polarization effects would be minor for the specimens tested herein, and DC measurements were considered acceptable.

RELATIONSHIP BETWEEN \( \theta \), \( S \), AND \( \sigma \) IN COMPACTED SOIL

Compaction curves were developed as shown in Fig. 2 along with the Zero Air Voids (ZAV) curve and lines of constant \( \theta \). These data indicate, as expected, a decrease in \( w_{opt} \) with increasing compaction effort, and values for \( w_{opt} \) and maximum \( \rho_d \) that are typical of compacted fine-grained soils. When measuring \( R \), it was found...
that $R$ was independent of vacuum level over the range of 10.6-37.4 kPa. Thus, a vacuum level of 10.6 kPa was sufficient to achieve intimate electrical contact.

![Graph](image)

Figure 2. Compaction curves with lines of constant $\theta$ (theta interval = 2%)

Electrical conductivity is plotted as a function of $\theta$ and $S$ in Figs. 3 and 4, respectively. The relationship between $\theta$ and $\sigma$, shown in Fig. 3, indicates that $\theta$ can be predicted in compacted clay based on measurement of $\sigma$. Furthermore, the relationship between $\theta$ and $\sigma$ is independent of compaction effort and independent of whether or not the soil is compacted wet or dry of $w_{opt}$. The relationship between $S$ and $\sigma$, shown in Fig. 4, indicates a dependence upon compaction effort, with standard proctor specimens being more conductive. At a given $S$, the specimen with the higher value for $n$ (the standard proctor specimen) will have the higher value for $\sigma$ as demonstrated by Eqn. 3. Since this specimen has more water per unit volume of soil, it is more electrically conductive. As a result, the standard proctor data are shifted towards higher $\sigma$ relative to the modified proctor data.

Results of the regression analyses by fitting the data to Eqn. 1 indicate values for $\sigma_{a},\sigma_{b},$ and $\sigma_{c}$ of −0.84, 1.59, and −0.085, mS/cm, respectively. The average prediction error (difference between measured $\sigma$ and $\sigma$ predicted by Eqn. 1 divided by measured $\sigma$) was 5.8%. The relatively low value for $\sigma_{c}$ indicates that the dry soil matrix of unsaturated compacted soil acts more or less as an insulator. This is also apparent when visually inspecting the plots in Fig. 3 as $\sigma_{c}$ represents the y-intercept of the data. This is in contrast to the range of 0.18-0.45 mS/cm for $\sigma_{s}$ reported by Rhodes et al. (1976) for undisturbed soil specimens, and for the value of 0.24 mS/cm reported by Kalinski and Kelly (1993) for compacted specimens. The apparent lack of conductivity of the dry matrix of the unsaturated compacted soil is attributed to a lack of intimate electrical contact between particles in compacted soil, where soil dry of $w_{opt}$ tends to remain as clods even when compacted in accordance with ASTM D698 or D1557 specifications (i.e. dry soil passed through a #4 sieve, and soil
allowed to hydrate overnight prior to compaction). Undisturbed soil specimens, on the other hand, are more homogeneous in nature, so the dry soil matrix is more electrically continuous. For the compacted specimens tested by Kalinski and Kelly (1993), the specimens were initially saturated and then dried back using the pressure membrane apparatus to achieve a desired level for $\theta$. This initial saturation likely provided the opportunity for the dry soil matrix to develop some continuity, resulting in a significant value for $\sigma$. Therefore, saturation history appears to play a role in the ability of the dry soil matrix of compacted soil to conduct electricity.

Figure 3. Electrical conductivity versus $\theta$ for the Morehead Clay

Figure 4. Electrical conductivity versus $S$ for the Morehead Clay

The electrical conductivity of the pore water ($\sigma_w$) can be estimated by using the limiting condition of $\theta = 1.00$. If $\theta = 1.00$, the entire volume of the mass being measured is water, so $\sigma$ theoretically approaches $\sigma_w$. Substituting $\theta = 1.00$ into Eqn.
yields an estimate for $\sigma_w$ of 0.66 mS/cm, which is comparable to the measured range of 0.2-0.6 mS/cm for the Lexington, Kentucky tap water. Using this estimate, values for $a$ and $b$ of $-1.3$ and $2.4$, respectively, were derived. These values are somewhat different the values reported by previous researchers (Table 1), which indicates the different nature of the soil specimens tested for this study.

A NEW METHOD FOR CQA TESTING OF COMPACTED SOIL

As demonstrated in the previous section, Eqn. 1 is valid for soils compacted over a range of compaction effort and initial $w$. For CQA purposes, the LOI is a key parameter in defining acceptance criteria. Therefore, it is necessary to compare the LOI to lines of constant $\theta$ to develop a method to use $\sigma$ measurements to predict $\theta$ for CQA purposes. As shown in Fig. 2, lines of constant $\theta$ are steeper than the LOI, but coincide somewhat, so measurement of $\theta$ is still useful in determining whether soil is wet or dry of the LOI. Based on this observation, a new method for CQA testing of compacted soil is proposed. The new method consists of the following steps.

1. Perform laboratory compaction testing of representative material in accordance to ASTM D698 and D1557 to derive standard and modified proctor compaction curves.

2. Measure $\sigma$ of each compacted specimen.

3. Measure $\sigma_w$ of the water used to prepare the specimens using a standardized procedure such as ASTM D1125, and record the temperature. Since $\sigma_w$ is temperature-dependent, $\sigma$ measurements should be performed at this temperature, and water with the same $\sigma_w$ should be used for each specimen.

4. Plot $\sigma$ versus $\theta$ and derive $a'$ and $b'$ using a normalized version of Eqn. 1:

$$\frac{\sigma}{\sigma_w} = a' \theta^2 + b' \theta,$$

where $\sigma_w$ is assumed to be zero because the soil specimens have not been saturated.

5. Perform field compaction testing. Measure $\sigma_w$ of the construction water at a known temperature $T_1$ (in degrees Fahrenheit), taking care to observe and account for seasonal and source variations. Since $\sigma_w$ of the construction water may vary seasonally or with different water sources, it is prudent to repeat this measurement frequently to account for such variability.

6. Perform in situ $\sigma$ measurement using field geophysics. Given the pore water temperature at the time of measurement $T_2$ (in degrees Fahrenheit), calculate
the temperature-corrected pore water conductivity of the construction water, \( \sigma_{w_{corr}} \), using the following relationship (Asquith and Gibson, 1982):

\[
\sigma_{w_{corr}} = \frac{\sigma_w (T_2 + 6.77)}{(T_1 + 6.77)}.
\]  

(7)

7. Use the in situ measured value for \( \sigma \), the temperature-corrected construction water conductivity, \( \sigma_{w_{corr}} \), and Eqn. 6, to estimate \( \theta \) of the in-place compacted material.

8. Superimpose lines of constant \( \theta \) over the compaction curves to establish a limiting criterion for \( \theta \), and compare the estimate for \( \theta \) derived in Step 7 to this criterion to assess the acceptability of the soil at the test point.

To perform Step 6, geophysical electrical conductivity measurements would be made at points in the same manner that nuclear gauge measurements are made at a point, but the geophysical measurements would be non-intrusive and non-contacting, and would be made “on the fly.” Shallow terrain conductivity measurement using an instrument with a coil spacing approximately equal to the thickness of the CCL would be an appropriate selection for a field method because it provides an estimate for \( \sigma \) approximates an average value within the CCL. Such devices have been used extensively to characterize soils for agricultural purposes (Mankin et al., 1997), and could be applied for geotechnical purposes as well.

This is a simplified approach that may be slightly conservative under certain circumstances (e.g. if you are compacting for low permeability and your field compaction effort is greater than the compaction effort at which your target \( \theta \) occurs at \( w_{opt} \)). The method may be enhanced by combining with a second method. If a second method could be developed or applied that allowed for accurate measurement of either \( \rho_d \) or \( w \), results from the second method could be combined with results from terrain conductivity testing to uniquely determine \( \rho_d \) and \( w \) at a given point. The pavement quality indicator (PQI) is a new method where the bulk dielectric properties of asphalt concrete are measured, and the measurement is used to quantify the various fractions in the material (i.e. aggregate, asphalt, and air) to measure the bulk density of the material. This technology has been developed for asphalt, but shows promise as a method to measure \( \rho_d \) in soil.

**CONCLUSIONS**

The relationship between \( \sigma \) and \( \theta \) developed by Rhodes et al. (1976) was found to be applicable to fine-grained soils compacted over a range of initial \( w \) and at different compaction efforts. The electrical conductivity of the dry soil matrix, \( \sigma_s \), appears to approach zero in compacted soil that has remained in an unsaturated state. However, saturation of the soil after compaction causes \( \sigma_s \) to be significant as
evidenced by research presented by Kalinski and Kelly (1993). When comparing \( \sigma - \theta \) relationships and \( \sigma - S \) relationships, it appears that \( \theta \) is a better indicator of \( \sigma \) because \( \theta \) is a more direct measure of the volume of water available to conduct electricity per unit volume of soil. Based on these observations, a simple, non-nuclear CQA approach for quickly and non-intrusively estimating whether or not soil is compacted dry or wet of \( w_{opt} \) is proposed. This approach should be tested in the field using test pads to assess the capabilities and limitations of the method.

REFERENCES


In Situ Estimate of Shear Wave Velocity Using Borehole Spectral Analysis of Surface Waves Tool

M. E. Kalinski and K. H. Stokoe

Abstract: In situ field testing has been performed over the past several years at a silty sand site in Austin, Tex. using the borehole spectral analysis of surface waves (SASW) tool to develop the technique and assess the validity of the method. The borehole SASW tool is an inflatable pressuremeterlike device that allows surface wave measurements to be performed along the wall of an uncased borehole while varying the in situ state of stress. Field results demonstrate the applicability of borehole SASW testing as a method to characterize soil sites and provide information about in situ shear wave velocity and the relationship between shear wave velocity and state of stress. Results from a borehole SASW test conducted at the Austin site are presented herein to demonstrate the applicability and validity of the method.

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CE Database subject headings: In situ tests; Velocity; Surface waves; Shear waves; Soil dynamics; Boreholes.

Introduction

Knowledge of the in situ shear wave velocity ($V_s$) of soil at a site is required for the calculation of the site’s response to dynamic excitation, such as an earthquake or machine vibrations. Nonintrusive geophysical methods commonly used today for estimating in situ $V_s$ include spectral analysis of surface waves (SASW) testing (Stokoe et al. 1994) and refraction seismic testing (Reynolds 1997). Borehole methods include downhole and crosshole seismic testing, suspension logging (Kitsunizaki 1980), and seismic cone penetration testing (Robertson et al. 1986). Another borehole method, the torsional cylindrical impulse shear method (Henke 1996), has been used to derive in situ modulus reduction data (modulus and damping versus shearing strain), which are also used for estimating site response. However, all of the existing methods are restricted to evaluating $V_s$ under the in situ state of effective stress.

Circumstances exist where the in situ state of effective stress in a soil mass can change. These circumstances include groundwater level fluctuations, increasing the overburden load, or excavating. It has been demonstrated that $V_s$ is dependent upon the state of effective stress in the soil (Hardin and Drnevich 1972; Stokoe et al. 1985; Bellotti et al. 1996), so changing the state of stress affects $V_s$, which in turn affects how the site responds to dynamic excitation. Shelby tube samples can be recovered and resonant column testing (Drnevich et al. 1978) or bender element testing (Dyvik and Madshus 1985) can be performed to estimate laboratory relationships between $V_s$ and the state of stress. However, prior to the development of the method presented herein, there were no known techniques for varying the in situ state of stress and estimating the in situ relationship between $V_s$ and the state of stress.

The borehole SASW tool was recently developed at the University of Texas at Austin to perform in situ measurements of $V_s$ in uncased boreholes in soil (Kalinski 1998; Kalinski and Stokoe 2000). The borehole SASW tool is an inflatable pressuremeterlike device that allows surface wave measurements to be performed along the wall of a borehole while varying the in situ state of stress. Thus, the borehole SASW tool can be used to estimate the in situ relationship between $V_s$ and the state of stress.

Borehole SASW testing has been conducted at a silty sand site in Austin, Tex., to develop the technique and assess the validity of the method. In this paper, the borehole SASW method and borehole SASW tool are outlined and described. Results from a test at the Austin site are presented. These results are combined with results from crosshole seismic and resonant column testing to help characterize the site.

Rationale Behind Borehole Spectral Analysis of Surface Waves Method

Introduction

The borehole SASW method is an extension of the traditional SASW method that has historically been applied to estimate variations in $V_s$ with depth in a flat, layered soil, rock, or pavement profile (Stokoe et al. 1994). Variations in $V_s$ with depth cause variations in surface wave velocity with wavelength or “dispersion.” By experimentally measuring surface wave dispersion with the SASW method and comparing the experimental results with theoretical dispersion data derived using the dynamic stiffness matrix (Kausel and Roesset 1981), variations in $V_s$ with depth in a flat, layered system are quantified.

The dynamic stiffness matrix $K$ describes the relationship between applied dynamic forces and resulting displacements at interfaces in a flat, layered system.
**Estimation of** $V_s$ **in Soil Surrounding Borehole**

To estimate $V_s$ in the soil surrounding an uncased borehole, the SASW method is applied using sources and receivers inside the borehole SASW tool. In this application, $V_s$ is estimated as a function of depth into the borehole wall, and concentric variations in $V_s$ are quantified. However, special considerations must be made to account for the cylindrical geometry of the borehole. For surface waves propagating inside a cylindrical cavity, there are two dispersion mechanisms. The first mechanism is concentric variations in $V_s$ in the material surrounding the borehole. In the same manner, variations in $V_s$ with depth in a flat, layered system cause surface wave dispersion when performing conventional flat-surface SASW testing. The second mechanism, geometry-induced dispersion, is unique to cylindrical cavities (Biot 1952). For surface waves propagating in a cylindrical cavity in a homogeneous material, the velocity of an axially propagating wave ($V_p$) increases slightly with increasing wavelength ($\lambda$) from the Rayleigh wave velocity at very short wavelengths to $V_s$ at wavelengths equal to about three times the cavity radius (Fig. 2). At longer wavelengths, mode conversion occurs as the surface wave energy converts into shear wave energy that propagates at a velocity equal to $V_s$. Given the boundary conditions of the problem, satisfaction of the wave equation requires that the longer-wavelength energy propagate as a shear wave.

Surface wave propagation in cylindrical cavities is governed by the wave equation as described by Biot (1952). Transformed into cylindrical coordinates $(r, \theta, z)$ and treated as an axisymmetric problem, the solution of the wave equation for waves propagating inside a cylindrical cavity requires solution of the following equations:

\[
\frac{\partial^2 \phi}{\partial r^2} + \frac{1}{r} \frac{\partial \phi}{\partial r} + \frac{\partial^2 \phi}{\partial z^2} = \frac{1}{V_p^2} \frac{\partial^2 \phi}{\partial t^2} \tag{2}
\]

and

\[
\frac{\partial^2 \psi}{\partial r^2} + \frac{1}{r} \frac{\partial \psi}{\partial r} - \frac{\psi}{r^2} + \frac{\partial^2 \psi}{\partial z^2} = \frac{1}{V_s^2} \frac{\partial^2 \psi}{\partial t^2} \tag{3}
\]

where $\phi=$ compressional potential function; $\psi=$ shear potential function; $r=$ radial distance from cavity center; $t=$ time; and $V_p=$ compressional wave velocity. For unattenuated waves propagating in the axial ($z$) direction, the solutions to Eqs. (2) and (3) are

\[
\phi = \phi_0 K_0(mr) \cos(kz - \omega t) \tag{4}
\]

and

\[
\psi = \psi_0 K_1(pr) \sin(kz - \omega t) \tag{5}
\]

where $\phi_0$, $\psi_0=$ constants; $k=$ wave number=$2\pi/\lambda$; $\omega=$ angular frequency=$2\pi f$; $K_0$=modified Bessel function of the second

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**Fig. 1.** Borehole spectral analysis of surface waves tool

**Fig. 2.** Dispersion of axially propagating surface wave in borehole (Poisson’s ratio=0.25)
kind of zero order; and \( K_1 = \text{modified Bessel function of the second kind of first order.} \) The parameters \( m \) and \( p \) in Eqs. (4) and (5) are defined as

\[
m = k \sqrt{1 - \left( \frac{V_r}{V_p} \right)^2}
\]

and

\[
p = k \sqrt{1 - \left( \frac{V_r}{V_p} \right)^2}
\]

Given the boundary conditions of zero normal and shear stress at the cavity wall, Eqs. (4) and (5) are used to derive an implicit relationship between \( V_r \) and \( \lambda \)

\[
4(1 - \xi_1^2) \left[ \frac{1}{pr_0^2} + \frac{K_0(pr_0)}{K_1(pr_0)} \right] - \frac{2(2 - \xi_1^2) \sqrt{(1 - \xi_2^2)}}{mr_0} = 0
\]

where \( r_0 = \text{cavity radius;} \) \( \xi_1 = V_p/V_r \); and \( \xi_2 = V_r/V_p \). The relationship between \( V_r \) and \( \lambda \) shown in Fig. 2 is derived from Eq. (8) using a Poisson’s ratio of 0.25.

From the governing equations for wave propagation in a cylindrical cavity [Eqs. (2) and (3)], a finite-element formulation in cylindrical coordinates with Fourier expansion in the wave propagation direction \( z \) was implemented to provide a two-dimensional axisymmetric solution in a manner analogous to the Green’s function approach for flat, layered systems (Cheng 1997). Analyses were performed at discrete frequencies using steady-state harmonic forcing functions applied directly to the cavity wall inside the cavity, and phase information at distances between two and four wavelengths from the source was derived. Since the phase velocity varies with distance from the source due to nearfield and modal superposition effects, linear regression analysis of the phase data was performed to calculate the rate of change of phase with respect to distance, \( d\Phi/dz \). The surface wave velocity \( V_r \) was then calculated as a function of frequency \( f \)

\[
V_r = \left( \frac{2\pi}{d\Phi/dz} \right) f
\]

By repeating the analysis over a range of frequencies, model dispersion curves were derived for comparison with the experimental data and concentric variations in \( V_r \) around the borehole were estimated.

**Estimation of State of Stress in Soil Surrounding Borehole**

To relate \( V_r \) to the state of stress, the effective stresses in the direction of wave propagation and particle motion must be known. For surface waves propagating axially in a vertical borehole, knowledge of the vertical and radial effective stresses \( (\sigma'_v \text{ and } \sigma'_r) \) is required (Fig. 3). To estimate \( \sigma'_r \) around a pressurized borehole at a given radial distance \( r \) from the center of the borehole, the following equation is applied (Timoshenko and Goodier 1970):

\[
\sigma'_r = C (\sigma'_h \sigma'_v)^n
\]

In Eq. (11), \( \sigma'_h \) and \( \sigma'_v \) are the effective stresses in the direction of particle motion and the propagation direction (Fig. 4), \( C \) is a constant related to soil structure and \( n \) is typically around 0.1. By taking the logarithm of both sides of Eq. (11), Eq. (11) is transformed into a linear equation where \( \log(C) \) and \( n \) are the slope and intercept of the line that defines the relationship between \( \log(V_r) \) and \( \log(\sigma'_v \sigma'_r) \).
be made and composite dispersion curves can be derived. By receiving pressures, several measurements can be made using small solenoids as impact sources and accelerometers as receivers. With the given configuration, several measurements can be made and composite dispersion curves can be derived. By briefly pulsing the solenoids with a short burst of current, the metal core in the solenoid taps the inside of the borehole and surface wave energy is generated. Frequencies as high as 20 kHz have been generated under exceptional conditions, but in most cases frequencies do not exceed 10 kHz. The solenoids are positioned against the inside of the membrane using a pneumatic piston system during testing, while the accelerometers are permanently fixed to the inside of the membrane using a silicone adhesive.

To calculate the amount of pressure delivered to the borehole wall by the tool \([\sigma_i, \text{in Fig. 3 and Eq. (10)}]\), the pressure delivered to the tool must be separated into (1) pressure mobilized by the membrane as it stretches (“membrane pressure”) and (2) pressure mobilized by the borehole wall (\(\sigma_i\)). To quantify membrane pressure, the membrane diameter must be known. By measuring the diameter during unconfined expansion in the laboratory, a relationship between membrane diameter and membrane pressure is established. Calipers (not shown in Fig. 5) inside the borehole SASW tool are used to measure membrane diameter during testing, which is converted into membrane pressure using the established relationship. By subtracting membrane pressure from the pressure delivered to the tool, the net pressure delivered to the borehole wall, \(\sigma_i\), is calculated.

Field Testing with Borehole Spectral Analysis of Surface Waves Tool

Borehole SASW testing was conducted at a sand pit operated by Capitol Aggregates in Austin, Tex. The site consists of a silty sand (SM) overlain by approximately 2.5 ft (0.76 m) of stiff clay. Borehole SASW testing was conducted in the silty sand unit at a depth to the center of the tool of approximately 8.5 ft (2.6 m). The silty sand has a total unit weight of approximately 129pcf (2,070 kg/m³), water content of 14%, degree of saturation of 80%, void ratio of approximately 0.46, and an assumed coefficient of earth pressure at rest of 0.5. The depth of the groundwater table was approximately 12 ft (3.6 m) at the time of testing, so a negligible

\[
\log(V_s) = \log(C) + n[\log(\sigma'_i \sigma'_r)] \quad (12)
\]

The constant \(C\) is most conveniently expressed as a unit stress product, where \(\sigma'_i \sigma'_r\) is set equal to unity and Eq. (12) is reduced to

\[
V_s = C \quad (13)
\]

By replacing \(\sigma'_i\) and \(\sigma'_r\) with \(\sigma'_i\) and \(\sigma'_r\) in Eq. (11), the expression for \(V_s\) of an axially propagating wave is expressed as

\[
V_s = C(\sigma'_i \sigma'_r)^n \quad (14)
\]

By performing borehole SASW testing over a range of tool pressures, \(V_s\) can be estimated at different states of stress. The resulting velocity and stress data can then be input into Eq. (14) and the parameters \(n\) and \(C\) can be calculated using a linear regression analysis. By plotting \(V_s\) versus \(\sigma'_i \sigma'_r\) on a log-log scale, the resulting curve is a straight line with a slope equal to \(n\) and an intercept corresponding to \(C\).

Description of Borehole Spectral Analysis of Surface Waves Tool

The borehole SASW tool is a cylindrical device approximately 36 in. (91 cm) long with a nominal inflated diameter of approximately 6.0 in. (15 cm) as shown in Fig. 5. It is designed to operate in an uncased borehole. The overall design consists of several key elements, including membrane, accelerometers, impact energy sources, and a caliper system.

The membrane of the borehole SASW tool is made from polyurethane sheeting with a thickness of approximately 0.031 in. (0.79 mm) and a Shore durometer hardness of A70 (ASTM D 2240). This material was selected due to its thinness, flexibility, ruggedness, and similarity in stiffness to a typical soil deposit. These attributes make the membrane virtually invisible to surface wave energy propagating inside the borehole.

The SASW sources and receivers are centered at the midheight of the tool and extend over a length of approximately 10 in. (25 cm). The sources and receivers are configured so that SASW arrays with receiver spacings of 1.0, 2.0, and 4.0 in. (25, 50, and 100 mm) are used. Since the distance between the instrumentation and the ends of the tool is greater than the borehole diameter, end effects due to pressure discontinuities at the top and bottom of the tool are minimized.

Spectral analysis of surface wave measurements are performed using small solenoids as impact sources and accelerometers as receivers. With the given configuration, several measurements can be made and composite dispersion curves can be derived. By
pore water pressure was assumed at the test depth. Based on these parameters, estimated vertical and horizontal effective stresses (\(\sigma'_v\) and \(\sigma'_h\)) at the test depth were 7.8 and 3.9 psi (54 and 27 kPa), respectively.

To prepare the borehole for testing, a pilot hole was initially drilled using a 4.5-in. (11-cm) solid-stem auger. Specially designed hand-trimming rings (Kalinski 1998) were then used to incrementally ream the borehole to its final test diameter of 6.375 in. (16.2 cm). By reaming to a diameter slightly greater than the 6.0-in. (15-cm) nominal diameter of the tool, the membrane is allowed to stretch into place and folds in the membrane are avoided. Use of the trimming rings also helps minimize borehole disturbance.

Testing by SASW was conducted using tool pressures of 4.0, 7.0, 10.0, and 13.6 psi (28, 48, 69, and 90 kPa). Based on measurements performed using the caliper system, these tool pressures correspond to mobilized borehole pressures (\(\sigma_i\)) of 3.0, 6.0, 9.0, and 12.0 psi (21, 41, 62, and 83 kPa), respectively. Measurements were made using a number of different source-receiver combinations and results were combined to derive composite dispersion curves.

**Results**

Composite dispersion curves derived from surface wave testing, shown in Fig. 6, reveal that surface wave velocity increased with increasing borehole pressure as expected. This indicates qualitatively that with increasing confining stress the stiffness of the silty sand also increased. Forward modeling of the dispersion curves allows this increase to be quantified. As shown in Fig. 7, the soil surrounding the borehole is separated into different annuli labeled A–G. Note that, at distances from the borehole wall greater than 0.35 ft (11 cm), \(V_s\) is not measurably affected by changes in borehole pressure. Velocities in this range are around 670 ft/s (204 m/s), which is consistent with velocities in the range of 600–700 ft/s (183–213 m/s) measured by crosshole seismic testing at the site. Both sets of data [SASW-derived \(V_s\) of soil greater than 0.35 ft (11 cm) from the borehole wall and the crosshole-derived \(V_s\)] are “free-field” velocities that are a measure of the stiffness of undisturbed soil under the in situ state of stress.

Using Eq. (10), \(\sigma'_v\) was estimated at different distances from the borehole wall at each applied pressure (Fig. 8). These curves indicate that at mobilized borehole pressures (\(\sigma_i\)) of 6.0, 9.0, and 12.0 psi (21, 41, 62, and 83 kPa) \(\sigma'_v\) decreased with increasing distance from the borehole wall. Accordingly, \(V_s\) should have also decreased with increasing distance from the borehole wall. However, \(V_s\) increased with increasing distance from the borehole wall within annuli A and B as shown in Fig. 7. Eq. (14) is based on the assumption of small-strain linear behavior with no change in soil structure and no change in \(C\), but shearing probably occurred in the soil near the borehole wall as a result of excavation and stress relief. Given the in situ void ratio of 0.46, this would have probably led to dilation and an increase in void ratio. Since C is a function of void ratio, it would also have changed during the test. Thus, shear wave velocities in the soil near the borehole wall (annuli A and B) were apparently affected by changes in both the state of stress and the degree of disturbance. Soil further away from the borehole wall (annuli C–G) was less disturbed and its stiffness was apparently affected primarily by changes in the state of stress.

The stress and \(V_s\) data were combined to plot \(V_s\) as a function of \(\sigma'_v\) (Fig. 9). In Fig. 9, data points are shown along with best-fit curves from linear regression analyses for annuli A, B, and C–G, and resonant column testing of a Shelby tube specimen recovered from the site. Annuli C–G were grouped together for analysis because they all appear to follow the same trend. The
A new in situ testing method using the borehole SASW tool has been presented herein as an effective technique for providing reasonable estimates for in situ $V_s$ and the in situ relationship between $V_s$ and the state of stress. Based on these results, an annulus of disturbed soil within 0.1 ft (30 mm) of the borehole wall was identified. Since the structure of the soil within this interval was permanently altered by excavation, its stiffness could not be restored to the free-field $V_s$ by increasing the state of stress. Relationships between $V_s$ and the state of stress derived for this interval are therefore not indicative of in situ conditions. Beyond this interval, the soil appeared to be less disturbed, so the relationships between $V_s$ and the state of stress derived for this interval were more indicative of undisturbed, in situ conditions.

The study presented herein was an initial attempt to demonstrate the validity of the method. Testing was performed under controlled conditions at a relatively shallow depth to facilitate the research. Application of this method in practice, however, would include testing at greater depths, possibly in a fluid-filled borehole, and in different soil types. In these instances, the trimming rings developed in this study may not be an appropriate choice for final preparation of the borehole and additional techniques would need to be developed. Such techniques may include development of a “self-boring” tool such as a pressuremeter (Briaud 1992).

Although the results presented in this study demonstrate the validity of the method, borehole disturbance is a limiting factor in obtaining data that are representative of true in situ conditions. Future developments may focus on reducing disturbance in the soil adjacent to the borehole through alternative borehole installation or preparation methods. However, even a self-boring tool such as the one described in the previous paragraph may not reduce borehole disturbance. Alternatively, reconfiguration of the tool may be an option. By reconfiguring the geometry of the tool, characterization of undisturbed soil further away from the borehole may be improved. Reconfiguration may include using a larger borehole with larger receiver spacings to measure longer-wavelength data.

Finally, the testing presented herein was performed on cohesionless soils, so a relationship between $V_s$ and state of stress developed for cohesionless soils [Eq. (11)] was used. However, if borehole SASW testing is performed on cohesive soils, an alternative relationship developed for cohesive soils that accounts for parameters such as overconsolidation ratio and plasticity index, as presented by Hardin and Drnevich (1972), may be more appropriate.

### Acknowledgments

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### Table 1. Summary of Linear Regression Analysis of Data Presented in Fig. 9

<table>
<thead>
<tr>
<th>Soil</th>
<th>$n$</th>
<th>$C$ at $\sigma'_s = 1.00$ psi $^a$</th>
<th>Average error in $V_s$ $^a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Annulus A</td>
<td>0.15</td>
<td>279</td>
<td>5</td>
</tr>
<tr>
<td>Annulus B</td>
<td>0.11</td>
<td>412</td>
<td>2</td>
</tr>
<tr>
<td>Annuli C–G</td>
<td>0.14</td>
<td>418</td>
<td>10</td>
</tr>
<tr>
<td>Resonant column specimen</td>
<td>0.10</td>
<td>427</td>
<td>2</td>
</tr>
</tbody>
</table>

Note: 1.00 ft = 0.305 m; 1.00 psi = 6.90 kPa.

$^a$Average difference between measured $V_s$ shown in Fig. 9 and calculated $V_s$ using Eq. (14) with $n$ and $C$ derived from linear regression analysis.

### Conclusions

The study presented herein was an initial attempt to demonstrate the validity of the method. Testing was performed under controlled conditions at a relatively shallow depth to facilitate the research. Application of this method in practice, however, would include testing at greater depths, possibly in a fluid-filled borehole, and in different soil types. In these instances, the trimming rings developed in this study may not be an appropriate choice for final preparation of the borehole and additional techniques would need to be developed. Such techniques may include development of a “self-boring” tool such as a pressuremeter (Briaud 1992).

Although the results presented in this study demonstrate the validity of the method, borehole disturbance is a limiting factor in obtaining data that are representative of true in situ conditions. Future developments may focus on reducing disturbance in the soil adjacent to the borehole through alternative borehole installation or preparation methods. However, even a self-boring tool such as the one described in the previous paragraph may not reduce borehole disturbance. Alternatively, reconfiguration of the tool may be an option. By reconfiguring the geometry of the tool, characterization of undisturbed soil further away from the borehole may be improved. Reconfiguration may include using a larger borehole with larger receiver spacings to measure longer-wavelength data.

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The following symbols are used in this paper:

- $f$: frequency;
- $K$: dynamic stiffness matrix of layered soil system;
- $K_0$: modified Bessel function of second kind of zero order;
- $K_1$: modified Bessel function of second kind of first order;
- $k$: wave number;
- $m, p$: symbols for grouped parameters used as Bessel function arguments;
- $n, C$: constants relating $V_s$ to in situ state of stress in soil;
- $P$: vector of applied loads at interfaces in layered soil system;
- $r$: radial distance from center of cylindrical cavity;
- $r_0$: cylindrical cavity radius;
- $t$: time;
- $U$: vector of displacements at interfaces in layered soil system;
- $V_p$: compressional wave velocity of soil;
- $V_r$: velocity of axially propagating surface wave in cylindrical cavity;
- $V_s$: shear wave velocity of soil;
- $z$: axial distance along cylindrical cavity;
- $\theta$: angular coordinate of cylindrical coordinate system;
- $\lambda$: wavelength;
- $\xi_1$: $V_s/V_p$;
- $\xi_2$: $V_r/V_p$;
- $\sigma_{h}^\prime$: horizontal effective stress in soil;
- $\sigma_{i}^\prime$: pressure applied to wall of cylindrical cavity;
- $\sigma_{w}^\prime$: radial effective stress in soil surrounding cylindrical cavity;
- $\sigma_{v}^\prime$: vertical effective stress in soil;
- $\sigma_{w}^\prime$: effective stress in direction of shear wave particle motion;
- $\sigma_{n}^\prime$: effective stress in direction of shear wave propagation;
- $\Phi$: phase;
- $\phi$: compressional potential function used in wave equation;
- $\phi_0, \psi_0$: constants used in solution of wave equation;
- $\psi$: shear potential function used in wave equation; and
- $\omega$: angular frequency.

References


GUIDELINES FOR GEOTECHNICAL INVESTIGATION AND ANALYSIS OF NEW AND EXISTING EARTH DAMS

1. PURPOSE

The purpose of this document is to provide dam owners and engineers with an overview of the procedures and considerations for performing geotechnical investigations and analyses of new and existing earth dams. The guidelines presented herein are written to provide a starting point for the evaluation of earth dams, but earth dams should be considered individually and engineering judgment should always be used to adapt the guidelines presented herein. The ultimate goal of any investigation should be to prudently evaluate the safety of the dam with respect to all identifiable and foreseeable failure mechanisms, and to monitor changes that may indicate development of a hazardous condition. Throughout this document, references are made to other documents that are readily available to dam owners and engineers. It is recommended that these other documents be consulted as needed during the geotechnical investigation process.

2. SUMMARY OF REGULATORY REQUIREMENTS FOR EARTH DAMS

Kentucky Revised Statute (KRS) 151.100 defines a “dam” in the State of Kentucky as follows:

(12) The word “dam” shall mean any artificial barrier, including appurtenant works, which does impound or divert water, and which either:

(a) Is or will be twenty-five (25) feet or more in height from the natural bed of the stream or watercourse at the downstream toe of the barrier, as determined by the cabinet; or
(b) Has or will have an impounding capacity at maximum water storage elevation of fifty (50) acre-feet or more.

Regulations establishing minimum safety and design criteria for dams in the State of Kentucky are described in Title 401, Chapter 4:030 of the Kentucky Administrative Regulations (401 KAR 4:030). Within 401 KAR 4:030, the procedures outlined in the latest edition of the United States Bureau of Reclamation (USBR) document “Design of Small Dams” (USBR, 1987) are incorporated by reference as the minimum design criteria. Although the 1973 edition of the document is cited in 401 KAR 4:030, use of the term “latest” in 401 KAR 4:030 implies that the most recent edition of the USBR document should be used.

University of Kentucky
Department of Civil Engineering
Division of Water Resources (DWR) Engineering Memorandum No. 5 (EM5) is also incorporated by reference into 401 KAR 4:030. Within EM5, dams are defined by hazard class as follows:

- **Class A (Low Hazard)** – Failure of the dam would result in little or no additional damage to other property;

- **Class B (Moderate Hazard)** – Failure of the dam may result in significant damage to property and project operation, but loss of human life is not envisioned; and

- **Class C (High Hazard)** – Failure of the dam may result in loss of life or serious damage to houses, buildings, utilities, highways, or railroads.

As stated in 401 KAR 4:030, all new Class B and Class C structures must incorporate a subsurface geotechnical investigation and analysis as part of their design for DWR approval. DWR is also required by KRS 151.295 to conduct an inspection program of all existing dams in the interest of public safety. If an existing dam is found to pose a threat to life or property, DWR is required through KRS 151.297 to order the owner to take such action as is necessary to render the dam safe. The process of rendering the dam safe may include a geotechnical investigation and analysis of Factors of Safety against various modes of failure, coupled with remedial action to increase the deficient Factors of Safety.

### 3. FAILURE MECHANISMS FOR EARTH DAMS

#### 3.1 Overview

With respect to earth dams, the term “failure” is defined herein as an occurrence of excessive erosion or deformation of the embankment that may result in an uncontrolled release of reservoir water or damage to appurtenant structures. To assess the safety of a dam and the possibility of failure, the different potential failure mechanisms must be recognized. Failure mechanisms are grouped into four general categories: slope stability, piping, overtopping, and foundation failures, as shown in Figure 1. By understanding these failure mechanisms, a geotechnical program of investigation, analysis, instrumentation, and monitoring can be developed to assess the safety of the dam with respect to each failure mechanism. A review of the different failure mechanisms for existing dams is also provided in USBR (2001) and FEMA (1987).
3.2 Slope Stability Failures

For a soil mass within an embankment, two forces act upon the soil mass. The driving force, due to the weight of the soil, tends to move the soil mass downslope. The resisting force, due to the strength of the soil along the base of the soil mass, or “slip surface,” tends to hold the soil mass in place. If the driving force is greater than the resisting force, the soil mass will slide along the slip surface and a slope stability failure will occur. The potential for failure for a given soil mass is quantified in terms of the Factor of Safety, which is defined as the resisting force divided by the driving force. If the Factor of Safety is greater than 1.0, the soil mass will not slide.

Factors of Safety are typically calculated assuming that the soil mass is crescent-shaped and the slip surface is a circular arc. This type of failure surface is generally consistent with observations of historically observed slope stability failures and facilitates the use of computer programs in the analysis, although soil masses with non-circular slip surfaces may also be evaluated. By using a computer to calculate Factors of Safety for a number of different soil masses and slip surfaces, the lowest overall Factor of Safety can be automatically derived.

For earth dams, there are three types of slope stability failures: steady-state, seismic, and rapid-drawdown. For the steady-state case, failure occurs on the downstream side of the dam under conditions of steady-state seepage. This type of failure may occur as a result of an increase in pore water pressure in the dam. For the rapid-drawdown case, failure occurs on the upstream side of the embankment as a result of a sudden lowering of the reservoir level.

For the seismic case, the driving force on the soil mass increases due to horizontal earthquake force, while the resisting force may be reduced if portions of the embankment or foundation liquefy. Liquefaction can occur during an earthquake in loose, saturated, sandy soils. During liquefaction, the soil particles are rearranged into a denser configuration, which tends to displace pore water. Since the pore water cannot vacate the pore spaces immediately, the pore water pressure temporarily increases. If this increase is sufficient, the soil particles become supported by the pore water, which has no shear strength. As a result, the shear strength of the soil approaches zero. When performing a seismic slope stability analysis, it may be found that at times during the earthquake when ground shaking is at a maximum, the Factor of Safety falls below 1.0 and some deformation occurs. A limited amount of deformation (e.g. less than 5 or 10 ft, depending on the height of the embankment) may be considered acceptable provided that there is no associated release of reservoir water.
3.3 Piping Failures

Properly designed earth dams are intended not to eliminate seepage completely, but to control seepage so that excessive water pressures within the embankment do not cause a steady-state slope stability failure. To control seepage, dams are often constructed with a core of fine-grained soil (to minimize seepage) flanked by zones of coarser-grained soil (to control seepage that does occur and prevent water pressure buildup). However, if measures are not taken to prevent the fine-grained soil particles from dislodging and seeping into the pore spaces of the coarse-grained soil, cavities can develop inside the dam. Cracks can also develop within the dam due to differential settlement within the embankment, especially if the depth to bedrock is highly variable. The cavities and cracks can act as preferential conduits for water to flow freely through the dam and erode the dam from the inside out. This phenomenon, referred to as “piping,” can cause a dam to fail suddenly and catastrophically. Cracks and fissures, high-permeability strata, and Karst features in the foundation and abutments may also act as preferential conduits and contribute to piping.

3.4 Overtopping Failures

Dams are designed with principal and emergency spillways to control the maximum reservoir elevation and prevent the reservoir from flowing over the top of the dam, or “overtopping.” When the spillways are not adequately designed, or if they become obstructed and cease to function, overtopping may occur. Overtopping can cause large amounts of erosion on the downslope side of the dam, which may compromise the stability of the dam.

3.5 Foundation Failures

When a new dam is constructed, the underlying foundation materials must bear a significant load due to the weight of the dam and reservoir. If the foundation consists of weak materials, such as soft clay, a foundation stability failure can occur, leading to significant deformation of the embankment. Failures may also occur under steady-state conditions in existing dams if a weak or permeable seam exists in the foundation. If seepage occurs along a seam, the elevated pore pressure and increased water content of the seam material may cause a reduction in strength along the seam. Piping within the seam may also be a contributing factor. Karst features may affect foundation capacity by allowing preferential flow, causing stress concentrations in the foundation rock, and presenting opportunities for limestone dissolution. Finally, liquefaction of granular soils during an earthquake may reduce the stability of the foundation.
4. TECHNICAL APPROACH TO EVALUATING EARTH DAMS

To evaluate a new or existing earth dam, a comprehensive approach of geotechnical field investigation, laboratory testing, and analysis is recommended. With this approach, the stability of the dam with respect to the identified failure mechanisms can be assessed under present conditions. However, a dam may not be stable tomorrow just because it is stable today. Changes may occur that reduce the stability of a safe dam and render it unsafe. To document these changes and prudently assess changes in dam stability with time, a program of instrumentation and monitoring is also recommended. In addition to instrumentation and monitoring, regular inspection of all dams in Kentucky is required by DWR.

For new and existing dams, the recommended approach to performing a geotechnical evaluation includes:

- Geological and geotechnical investigation;
- Geotechnical analyses;
- Instrumentation and monitoring; and
- Visual inspection.

Each component of the geotechnical evaluation is discussed in the following sections.

5. GEOLOGICAL AND GEOTECHNICAL INVESTIGATION

5.1 Overview

The purpose of a geological and geotechnical investigation is to obtain information necessary to perform stability analyses. Components of geological and geotechnical site investigation programs for new and existing earth dams are listed in Table 1. Further details regarding geological and geotechnical investigations can be found in USBR (2001) and USACE (2001).

5.2 Geological Reconnaissance

Geological reconnaissance involves the evaluation of geologic maps, supplemental field geologic mapping, field geophysical testing, and seismic hazard assessment, to characterize geologic features that may affect the stability of the dam. Recommended geological reconnaissance activities for new and existing dams are listed in Table 1.

Geologic maps contain information about geologic structure and stratigraphy, including age of rocks, type of rocks, and the presence of faulting or
other geologic features. Published geologic maps can be obtained from the United States Geological Survey (USGS) or the Kentucky Geological Survey. By evaluating an existing geologic map, a general understanding of the site geology can be obtained. To refine the information obtained from published geologic maps, field geologic mapping should be performed. Field geologic mapping will reveal local features that affect the stability of the dam, but do not appear on a larger-scale regional map. Weak or permeable strata and evidence of faulting or Karst features are important to identify and consider in the evaluation of dam stability.

To assist in delineating Karst features, which are prevalent in Kentucky, geophysical surveying may be used as a supplement to geologic mapping. Geophysics uses physical measurements of various parameters at the ground surface to characterize the subsurface. Many geophysical techniques are used today for delineating Karst features, but the DC resistivity and gravity methods are most common (Reynolds, 1997; USACE, 1995A).

As part of the geological investigation, a site-specific seismic hazard assessment should be performed. Seismic hazard assessment consists of selection of design earthquake criteria, estimation of peak ground acceleration, and estimation of spectral acceleration. USBR (2001) recommends that ground shaking from the Maximum Credible Earthquake (MCE) or an approximate probabilistic earthquake be used as the design earthquake (USCOLD, 1999). MCE ground shaking can be derived using an assumed source location, earthquake magnitude, and attenuation rate (Kramer, 1996; Toro et al., 1997). Probabilistic earthquake data can be obtained from the USGS Seismic Hazard Mapping Project Internet web site (Frankel et al., 1996). Representative seismograms may be required for more rigorous evaluations of critical dams or dams in areas of high seismicity and can be obtained from the USGS.

5.2 Geotechnical Exploration

Geotechnical exploration is necessary to obtain information to assess the stability of the dam. During the geotechnical exploration program, testing is performed in both the foundation and embankment materials. Components of the geotechnical exploration program are included in Table 1. As shown schematically in the example in Figure 2, boreholes should be located along the crest of the dam, along the center axis of the dam, and at other locations to adequately sample the core, drain, filters, abutments, foundation materials, and phreatic surface. Geological reconnaissance data, construction documents, and historical performance data should be used to help identify zones requiring additional boring and sampling. If drill locations are accessible, boreholes can be advanced using truck-mounted drill rigs. Smaller hand-operated power augers or manual augers may be used on side slopes that are inaccessible to trucks.
Boreholes should be drilled to penetrate the entire thickness of embankment and a thickness of foundation material equal to 50% of the embankment height.

Boreholes can be used for Standard Penetration Testing (SPT), disturbed split-spoon soil sampling, undisturbed Shelby tube soil sampling, and rock coring. Soil sampling should be performed at regular intervals in the boreholes as indicated in Table 1, but some borings may require more frequent or even continuous logging and sampling for adequate characterization. Boreholes can also be used to perform in situ hydraulic conductivity testing in suspected high-permeability zones (USBR, 1987).

Shear wave velocity testing should also be performed to derive profiles of shear wave velocity versus depth for seismic site response analysis. Numerous methods exist for deriving shear wave velocities, including borehole seismic, refraction, and surface wave methods (USACE, 1995A).

5.3 Geotechnical Laboratory Testing

Samples recovered during the geotechnical exploration program should be tested in the laboratory to characterize the material and derive parameters for subsequent stability analyses. Recommended tests are included in Table 1 with applicable American Society for Testing and Materials (ASTM) standards. Typical test frequencies are also given in Table 1. For all dams, test frequencies should be modified to address the unique conditions of the dam. Brief descriptions of each test are as follows:

- **Atterberg Limits Test** – measures plasticity of silts and clays;
- **Grain Size Analysis/Soil Classification** – measures particle size distribution in soil and provides classification based on general engineering properties;
- **Direct Shear Test** – measures shear strength of sands;
- **Consolidated-Undrained Triaxial Shear Test** – measures shear strength of silts and clays with pore water pressure measurements after consolidation (i.e. R- and S-envelopes);
- **Unconsolidated-Undrained Triaxial Shear Test** – measures undrained shear strength (i.e. Q-envelope) of silts and clays; and
- **One-Dimensional Consolidation Test** – measures susceptibility of silts and clays to settlement.
6. GEOTECHNICAL ANALYSES

6.1 Overview

After the geological and geotechnical investigation is completed, information obtained from the investigation is used to perform geotechnical analyses to assess the safety of the dam with respect to the failure mechanisms discussed in Section 3. Additional available information, such as construction drawings, construction quality assurance (CQA) test results, historical observations, and data acquired from previous inspections and monitoring, should also be incorporated into the analyses as appropriate.

For existing dams, the analysis process may be divided into two phases (USBR, 2001). A Phase I analysis may be performed prior to the geological and geotechnical investigation using existing information. If results of the Phase I analyses are inconclusive, then the geological and geotechnical investigation is performed, followed by a Phase II analysis.

The overall analysis process, either Phase I or Phase II, is summarized in Table 2 and should contain the following components:

- Slope stability analysis under steady-state conditions;
- Slope stability analysis under rapid drawdown conditions;
- Slope stability analysis under seismic loading conditions;
- Evaluation of piping potential;
- Foundation stability analysis; and
- Overtopping potential

Each component is discussed in the following sections. Further details regarding geotechnical analyses of dams can be found in USBR (2001).

6.2 Slope Stability Under Steady-State Conditions

A slope stability analysis should be performed on the downstream slope under long-term conditions of steady-state seepage. Existing and newly acquired geological and geotechnical data should be used to derive two or more representative cross-sections of the dam. Details such as slope height, steepness, effective strength parameters of different materials, and location of the phreatic surface, should be included in the cross-section to be analyzed.

As mentioned previously, most slope stability analyses are performed using computer programs that automatically search different potential slip surfaces to identify the slip surface that yields the lowest Factor of Safety. Numerous computer programs exist for performing slope stability analyses, (USCOLD, 1992), including the REAMES program developed at the University of Kentucky.

University of Kentucky
Department of Civil Engineering
Kentucky (Huang, 1982). However, if the user wishes to evaluate non-circular slip surfaces, such as a weak foundation layer, REAMES cannot be used.

6.3 Slope Stability Under Rapid Drawdown Conditions

Slope stability analyses should be performed on the upstream slope under conditions of rapid drawdown of the reservoir. When a reservoir level is lowered “rapidly” (e.g. a foot or more per day), conditions of undrained loading are imposed upon the embankment. To properly calculate the stability of the slope, a two-stage computation should be performed. The first stage involves consolidation of the embankment under long-term conditions and is performed to calculate pre-drawdown stresses in the embankment. The second stage involves undrained unloading of the embankment. In the approach implemented by Wright (1991) in the UTEXAS3 slope stability program, shear strength parameters are defined for both stages using the R- and S-envelopes derived from triaxial shear strength testing. To perform a rapid drawdown analysis, it is recommended that a program that allows for staged loading, such as UTEXAS3, be used.

6.4 Slope Stability Under Seismic Loading Conditions

Kentucky is surrounded by areas of active seismicity, including the New Madrid Seismic Zone, the Wabash Valley Fault Zone, and the Eastern Tennessee Seismic Zone. Therefore, slope stability under seismic loading should be evaluated. Numerous references exist that provide guidance for seismic stability analysis (Makdisi and Seed, 1978; Seed, 1979; Marcuson et al., 1990; Babbitt and Verigin, 1996; USCOLD, 1999; Deschamps and Yankey, 2001), and the approach used can range from simple to rigorous. Whether a simple or rigorous approach is selected, it is ultimately left to the discretion of the engineer to exercise prudent judgment in selecting an approach to adequately evaluate the seismic stability of the dam.

A simple analysis using conservative assumptions may be performed in areas where anticipated ground shaking levels are low. A simple analysis for evaluating seismic slope stability in zones of low seismicity may involve the use of peak ground acceleration levels derived from the seismic hazard assessment to perform a pseudostatic slope stability analysis. As is the case for rapid drawdown, earthquake loading is a two-stage problem. The first stage of loading corresponds to consolidation of the embankment under long-term conditions and is performed to calculate pre-earthquake stresses in the embankment. The second stage involves undrained loading of the embankment during the earthquake. It is recommended that a program that allows for staged loading be used to perform this analysis.
In areas where anticipated ground shaking is relatively high, such as western Kentucky, a more rigorous approach may be needed. A more rigorous approach may also be needed if a Factor of Safety less than 1.0 is calculated from a simple analysis, the dam is a Class C structure, or if liquefaction of the embankment or foundation materials is considered a possibility. The general steps of a more rigorous analysis may include:

- Use the results of the seismic hazard assessment and shear wave velocity data from the site to perform a site response analysis to estimate site amplification using the SHAKE91 (Idriss and Sun, 1992) or QUAD4M (Hudson et al., 1994) computer programs;
- Identify zones of liquefiable material based on anticipated ground shaking and SPT test results (Youd et al., 2001) and assign residual strengths to the liquefiable material (Seed and Harder, 1990);
- Perform slope stability analysis with residual strengths and staged earthquake loading of the embankment using numerical simulation (Seed, 1979) and re-calculate the zone of liquefaction after each stage;
- Perform staged Newmark-type deformation analysis to calculate cumulative deformation during the earthquake (Makdisi and Seed, 1979); and
- Calculate post-earthquake slope stability using residual strengths and elevated pore pressures (Marcuson et al., 1990).

6.5 Piping

Piping potential should be evaluated within the embankment using results from grain size analyses of disturbed samples recovered during the geotechnical exploration program. For two different soil types placed adjacent to one another within an embankment, such as a core and an adjacent filter, the grain sizes of the adjacent materials should be such that the finer-grained particles (base) are not allowed to pass through the pore spaces of the coarser-grained material (filter). To prevent this from occurring, the following filter criteria have been developed (USBR, 1987):

- \( \frac{D_{15\text{filter}}}{D_{15\text{base}}} \geq 5 \) and
- \( \frac{D_{15\text{filter}}}{D_{85\text{base}}} \leq 5 \),
where:

\[
\begin{align*}
D_{15\text{filter}} & \quad \text{sieve opening size that allows 15\% of the filter material to pass;} \\
D_{15\text{base}} & \quad \text{sieve opening size that allows 15\% of the base material to pass;} \\
D_{85\text{base}} & \quad \text{sieve opening size that allows 85\% of the base material to pass;} \\
\end{align*}
\]

Information obtained from the geological and geotechnical investigation, visual inspections, and monitoring, should also be considered in assessing piping potential. The following features may be indicators that piping potential exists or that piping is occurring:

- Cracks, cavities, or voids in the embankment, foundation, and abutments;
- Permeable strata in the foundation or abutment;
- Seepage observed in cracks in the abutment;
- Anomalously high flow rates or turbid discharge in the weirs; and
- Sand boils at the downstream toe of the dam.

If any of these features are observed, an assessment should be made as to the seriousness of the feature with respect to piping and potential failure of the dam.

Piping can also exist at the downstream toe of an embankment due to quick conditions and the development of sand boils. Flownets should be constructed using piezometric data to estimate pore pressures and upward seepage forces at the downstream toe of the embankment. A flownet is a graphical representation of flow lines and equipotential surfaces that illustrates pore pressure distribution within a cross-section that is experiencing steady-state seepage (USACE, 1993). Using the flownet, effective stresses can be calculated at the downstream toe. If effective stresses are found to be at or near zero, quick conditions may exist. The most obvious physical evidence of quick conditions at the downstream toe of the embankment would be the presence of sand boils, which are small (a foot or more in diameter) volcano-like features that may have water with suspended soil welling up from the center.

Results from consolidation testing of undisturbed samples of silts and clays recovered from the foundation should be used to estimate settlement beneath new dams. Settlement should be estimated at different points beneath the dam with varying depths to bedrock to quantify the potential for differential settlement beneath the dam. Differential settlement may result in cracking within the embankment and create the opportunity for piping. Results from CQA testing should also be reviewed to assess the uniformity of compaction throughout the embankment. Local variations in dry unit weight observed during CQA testing may be an indicator of local variations in the compressibility of the embankment, which could also lead to differential settlement.
6.6 Foundation Stability

Evaluation of foundation stability is similar to a slope stability problem, but the slip surface engages the foundation material. For new dams, a foundation failure may occur under short-term, undrained conditions as the dam is constructed and the reservoir is filled. To perform the analysis, a slope stability program that allows for the evaluation of circular and non-circular slip surfaces should be used to allow for flexibility in evaluating different types of potential failure surfaces. Since loading occurs under undrained conditions, the analysis should be performed using the undrained shear strengths of the foundation materials obtained from the geological and geotechnical investigation.

For existing dams, failure may occur along a seam in the foundation that is weak, permeable, or susceptible to piping. Information obtained from the geological and geotechnical investigation should be used to develop cross-sections that include any identified weak seams, and piezometric data should be used to define the phreatic surface. To perform the analysis, effective strength parameters should be used and non-circular slip surfaces that engage the weak seams should be evaluated. If the foundation contains soils susceptible to liquefaction, its stability should also be evaluated under seismic and post-seismic conditions in the same manner that embankment stability is evaluated.

6.7 Overtopping

The dam freeboard and spillways should be sufficient to accommodate the freeboard hydrograph or the emergency spillway hydrograph as defined in EM5 without overtopping. EM5 should be consulted for guidelines regarding design hydrographs and spillway criteria, and observations of the freeboard and spillways should be compared to the design guidelines in EM5 to assess the adequacy of the dam with respect to overtopping.

7. INSTRUMENTATION AND MONITORING

7.1 Overview

As mentioned previously, an earth dam that is safe under current conditions may not be safe in the future if conditions change. Conditions that may change and possibly indicate a reduction in stability include changes in the phreatic surface, embankment deformation, or changes in seepage patterns. By implementing a program of instrumentation and monitoring, parameters related to these conditions can be measured and preemptive action can be taken in response to these observations. Common types of instrumentation used to monitor the condition of earth dams are summarized in Table 3 and include:
• Piezometers;
• Inclinometers;
• Survey points;
• Weirs;
• Crack displacement measurements; and
• Seismometers.

Each type of instrumentation is illustrated schematically in Figure 3 and discussed in the following sections. Additional details regarding the instrumentation of earth dams can be found in USACE (1995B) and FEMA (1987).

7.2 Piezometers

Piezometers are used to measure pore water pressure and the location of the phreatic surface within an embankment. The family of instruments that serve as piezometers includes monitoring wells, where the position of the phreatic surface is directly measured, and transducers, where physical parameters, such as strain of internal components, are translated into fluid pressure. Piezometers should be deployed at regular intervals to adequately define the position of the phreatic surface and the pore pressure field within the embankment.

7.3 Inclinometers

Inclinometers are cylindrical tools that are placed in boreholes to measure borehole inclination in two orthogonal directions. By monitoring changes in borehole inclination over time, deformation within the embankment can be detected. Special casing with grooves on the inside is used in inclinometer boreholes. Wheels on the inclinometer fit into the grooves and align the inclinometer as it is lowered into the casing so that the inclination directions measured with the inclinometer are known. Inclinometer boreholes should be installed at locations within the embankment that may be susceptible to deformation. Inclinometer boreholes may also be used as monitoring wells for defining the phreatic surface if the inclinometer is submersible.

7.4 Survey Points

Survey points may be used to measure the elevation and position of the surface of an embankment. By periodically surveying the position of the survey points, changes in the shape of the embankment surface can be detected, which may be an indicator of settlement or deformation. A network of survey points should be deployed at regular intervals to adequately define the surface of the embankment. Survey point monuments should be embedded into the embankment as permanent features. When conducting a survey, the position of
the points should be measured relative to a fixed reference point that is not located on the embankment and not susceptible to deformation.

7.5 **Weirs**

Weirs may be installed at the downstream toe of the dam at locations that have historically exhibited seepage. Flow rates can be calculated using the weir data and compared to historical flow rates. Unusually high flow rates that cannot be associated with changes in pool elevation or recent precipitation may be an indication that preferential flow conduits and piping are developing. Unexplained turbidity (cloudy water) discharging from the weir may also be an indicator of piping. Samples of weir discharge should be recovered during each monitoring period and saved for qualitative comparison with samples recovered from subsequent monitoring periods to observe any changes in turbidity.

7.6 **Crack Displacement Measurements**

Displacement of existing cracks on the embankment may be monitored using measurement points installed on either side of the crack. Variations in distance across the crack over time may be an indication of slope movement. The control points should be embedded into the embankment as permanent features. By using two control points on either side of the crack, displacement can be measured parallel to the crack and perpendicular to the crack. Changes in elevation between the control points can also be used to estimate changes in vertical displacement across the crack.

7.7 **Seismometers**

For dams located in areas of high seismicity, or for dams of high hazard class, seismometers may be installed on the embankment. The purpose of installation of seismometers is to compare measured ground shaking to the predicted ground shaking that was used in the seismic stability analysis. Such an exercise serves as a confirmation that the seismic hazard assessment and site response analyses were performed using model ground shaking parameters that are representative of actual ground shaking, which is particularly useful in areas where the seismic stability analysis may have been performed with a high degree of uncertainty and conservatism.

8. **VISUAL INSPECTION**

8.1 **Overview**

Inspections are required by DWR as a permit condition for all existing earth dams in the State of Kentucky. Inspections should be carried out at regular
intervals at a frequency to be established by DWR. Inspection frequency is generally dictated by the risk involved with the dam. For example, Class C dams or dams that possess marginal stability would require more frequent inspections than other dams. If possible, dams should be inspected during periods of high loading, such as during the spring, to obtain observations of dam behavior under worst-case conditions.

Some common elements of a dam inspection program are briefly described in the following sections, illustrated in Figure 4, and summarized in Table 4. However, each dam is unique, and the inspection program presented herein and in other literature should be appropriately adapted to address the special considerations of the dam being inspected. Additional details regarding dam inspections, including examples of inspection checklist forms, can be found in DWR (1985), FEMA (1987), and USBR (2001).

8.2 Upstream Riprap

The upstream riprap should be inspected to assess its condition and its performance as a wave baffle. Evidence of wave erosion or missing portions of riprap should be documented. If it is anticipated that the reservoir level will be lower than the level during the time of inspection, the riprap should be inspected below the water line by boat (if water clarity allows) or by divers.

8.3 Condition of Relief Wells, Spillways, and Conveyance Structures

Spillways should be free of trash, debris, and other obstructions. The emergency spillway should appear to be in good condition. Ground cover in vegetated portions of the emergency spillway should be healthy, and the spillway should be free of erosion ruts and gullies. Concrete should be intact with no spalling. Relief wells should appear to be functioning properly. There should be little or no seepage between conveyance structures (e.g. spillway outfall) and the surrounding embankment.

8.4 Abutment and Weir Seepage

Seepage appearing in the abutments should be documented. The lines of intersection between the downstream slope of the embankment and the abutments (i.e. groins) may be particularly susceptible to seepage. There should be little or no surface erosion resulting from the abutment seepage and discharge from the abutments and weirs should have little or no suspended solids. Samples of abutment seepage may be collected and tested for total dissolved solids, which could be an indicator of limestone dissolution in the abutment.

8.5 Surface Erosion
The crest, downstream slope, downstream toe, abutment, and spillway areas should be inspected for evidence of erosion. Evidence of surface erosion should be documented along with possible causes of erosion (e.g. lack of vegetative cover, excessive grade, etc.). The groins may be particularly susceptible to surface erosion.

8.6 Cracking

The crest and sideslopes of the dam should be inspected for cracks, and cracking should be documented in terms of orientation, length, width, and depth. Transverse cracks (parallel to stream direction) may develop as a result of differential settlement, and may be more susceptible to surface erosion and piping. Longitudinal cracks (parallel to the dam) may be associated with slumping and indicate a slope stability failure. Cracks may occur as a result of surface desiccation.

8.7 Differential Movement

The crest and slopes of the dam should be visually inspected for evidence of differential movement. Differential movement may be a result of varying degrees of foundation settlement, which may also be associated with transverse crack development within the body of the embankment. Features such as sagging in the middle of the crest, bulging near the toe of the dam, or misalignment of the crest, may be an indicator of slope movement or lateral spreading of the dam and should be documented. Sagging in the crest may be of particular concern if it causes reduction in available freeboard and increases the potential for overtopping.

8.8 Biological Disturbance

Biological disturbance may lead to surface erosion or create opportunities for preferential flow and subsequent piping. Biological disturbance, including animal walking paths and burrows, should be noted. Evidence of human disturbance and human vandalism should also be recorded.

8.9 Cavity Formation

Unexplained cavities and voids appearing on the surface of the embankment are a strong indicator that piping is occurring within the embankment. Information about cavity size and location should be documented. Cavity development is particularly significant if other indicators of piping, such as zones of turbid seepage, are observed.
8.10 Sand Boils

Wet areas downstream of the embankment should be inspected for the presence of sand boils. Sand boils are small volcano-like features that may have water with suspended solids welling up from the center, and are an indication of quick conditions and piping. They would most likely be found in areas of standing water and may be accompanied by patches of wetland vegetation, such as cattails.

8.11 Vegetation Patterns

Unusual vegetation patterns may be an indicator of potential problems and should be noted. The presence of patches of wetland vegetation on the downstream side of the dam may be caused by uncontrolled or undesirable seepage patterns, and may be an indicator of excessive pore water pressures. Zones of unusually green and lush ground cover on the downstream side slope are an indicator that the phreatic surface is at or near the ground surface. Root systems of large shrubs and trees present on the slope of the embankment may also create opportunities for preferential flow and piping. The shade provided by trees may prevent surface cover from growing and create the potential for surface erosion.

9. DOCUMENTATION

At the conclusion of the investigation, the results of the geotechnical investigation, geotechnical analysis, monitoring activities, and inspection activities should be compiled in a report that can be used as a reference for future investigations. As a result of the investigation, a qualitative condition may be assigned to the dam consistent with USBR (2001) as follows:

- **Satisfactory** – No existing or potential dam safety deficiencies are recognized;
- **Fair** – Extreme loading conditions (e.g. MCE) would probably result in a dam safety deficiency;
- **Conditionally Poor** – Unusual, but reasonably expected loading conditions (e.g. moderate-sized earthquake) would probably result in a dam safety deficiency;
- **Poor** – Potential for a dam safety deficiency under normal loading conditions exists; and
- **Unsatisfactory** – A dam safety deficiency exists.
A list of conclusions and recommendations should also be provided. The recommendations should be a set of clear, concise, decisive actions intended to address deficiencies in the dam and improve the quality of future investigations.

10. REFERENCES


DWR, 1985, Guidelines for Maintenance and Inspection of Dams in Kentucky, Kentucky Natural Resources and Environmental Protection Cabinet Division of Water Resources.


Hudson, M., Idriss, I. M., and Beikae, M., 1994, User’s Manual for QUAD4M, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis.

Idriss, I. M., and Sun, J. L., 1992, User’s Manual for SHAKE91, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis.


Table 1. Components of a typical geotechnical investigation program for earth dams

<table>
<thead>
<tr>
<th>Component (with applicable ASTM standard)</th>
<th>Typical Test Frequencies (subject to the discretion of the engineer)</th>
<th>Purpose of Activity</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Geological Reconnaissance</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Geologic Mapping</td>
<td>• 1 per site</td>
<td>• Identification of weak or permeable zones</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Identification of faults</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Identification of Karst features</td>
</tr>
<tr>
<td>Geophysical Surveying</td>
<td>• As needed</td>
<td>• Delineation of identified Karst features</td>
</tr>
<tr>
<td>Seismic Hazard Assessment</td>
<td>• 1 per site</td>
<td>• Estimation of local earthquake shaking intensity</td>
</tr>
<tr>
<td><strong>Geotechnical Exploration</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil Borings</td>
<td>• 1 per 200 ft along dam crest alignment</td>
<td>• Recovery of undisturbed and disturbed soil specimens and rock cores</td>
</tr>
<tr>
<td></td>
<td>• Sufficient to adequately characterize embankment, abutment, and foundation materials</td>
<td>• Standard penetration testing</td>
</tr>
<tr>
<td></td>
<td>• Drill to a total depth equal to 50% of the dam height</td>
<td>• Estimation of depth to bedrock</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Undisturbed Soil Sampling (ASTM D1587)</td>
<td>• 1 per 5 ft of borehole</td>
<td>• Acquire samples for laboratory testing</td>
</tr>
<tr>
<td></td>
<td>• Sufficient to adequately characterize embankment, abutment, and foundation materials</td>
<td></td>
</tr>
<tr>
<td>Standard Penetration Test/Disturbed Soil Sampling (ASTM D1586)</td>
<td>• 1 per 5 ft of borehole</td>
<td>• Acquire samples for laboratory testing</td>
</tr>
<tr>
<td></td>
<td>• Sufficient to adequately characterize embankment, abutment, and foundation materials</td>
<td>• Acquire information for liquefaction analysis</td>
</tr>
<tr>
<td>Rock Coring</td>
<td>• 1 per 5 ft of rock</td>
<td>• Acquire samples for inspection</td>
</tr>
<tr>
<td></td>
<td>• Sufficient to adequately characterize abutment and foundation materials</td>
<td>• Identification of weak or permeable zones</td>
</tr>
<tr>
<td>Borehole Hydraulic Conductivity Testing</td>
<td>• As needed in critical zones</td>
<td>• Identification of high-permeability zones in foundation, abutments, and embankment</td>
</tr>
<tr>
<td>Shear Wave Velocity Testing (Borehole, Refraction, or Surface Wave Method)</td>
<td>• 1 per 3 acres</td>
<td>• Acquire information for seismic site response analysis</td>
</tr>
<tr>
<td></td>
<td>• 1 profile along dam crest alignment</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Use methods that will provide information down to bedrock</td>
<td></td>
</tr>
<tr>
<td>Component (with applicable ASTM standard)</td>
<td>Typical Test Frequencies (subject to the discretion of the engineer)</td>
<td>Purpose of Activity</td>
</tr>
<tr>
<td>------------------------------------------</td>
<td>-------------------------------------------------</td>
<td>-------------------</td>
</tr>
<tr>
<td><strong>Geotechnical Laboratory Testing</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Atterberg Limits (ASTM D4318)</td>
<td>2 per distinct soil unit</td>
<td>Use for soil classification</td>
</tr>
<tr>
<td>Grain Size Analysis/Soil Classification (ASTM D422, D1140, D2487)</td>
<td>2 per distinct soil unit</td>
<td>Use to estimate soil strength and permeability, Use for piping analysis, Use for liquefaction analysis</td>
</tr>
<tr>
<td>Direct Shear Test (ASTM D3080)</td>
<td>As needed to adequately characterize distinct units in the foundation and embankment</td>
<td>Use to estimate strength of cohesionless soils for foundation and slope stability analyses</td>
</tr>
<tr>
<td>Consolidated-Undrained Triaxial Shear Strength with Pore Pressure Measurements (ASTM D4767)</td>
<td>As needed to adequately characterize distinct units in the foundation and embankment</td>
<td>Use to estimate drained shear strength of cohesive soils for foundation and slope stability analyses</td>
</tr>
<tr>
<td>Unconsolidated-Undrained Triaxial Shear Strength (ASTM D2850)</td>
<td>As needed to adequately characterize the shear strength of the foundation</td>
<td>Use to estimate undrained shear strength of cohesive soils for foundation stability analysis</td>
</tr>
<tr>
<td>One-Dimensional Consolidation (ASTM D2435)</td>
<td>1 per distinct unit of cohesive soil</td>
<td>Use to estimate anticipated differential settlement of foundation beneath new dams</td>
</tr>
</tbody>
</table>
Table 2. Components of a typical geotechnical analysis program for earth dams

<table>
<thead>
<tr>
<th>Failure Mechanism</th>
<th>Description</th>
<th>General Analysis Method(s)</th>
<th>Acceptance Criteria</th>
</tr>
</thead>
</table>
| Slope failure (steady-state seepage conditions) | • Slope failure occurs on downstream slope under conditions of steady-state seepage, possibly due to buildup of pore water pressure in the embankment or removal of soil at the toe. | • Compute slope stability (Huang, 1982; Achilleos, 1988; Wright, 1991; USCOLD, 1992) | • Factor of Safety > 1.3 for new dams;  
• Factor of Safety > 1.5 for existing dams                                           |
| Slope failure (rapid drawdown conditions)   | • Slope failure occurs on the upstream side of the slope due to rapid drawdown of reservoir and associated undrained loading of the embankment.                                                             | • Compute slope stability using two-stage (consolidation and undrained loading) approach (Wright, 1991) | • Factor of Safety > 1.2                                                             |
| Slope failure (seismic conditions)          | • Slope failure occurs on the upstream or downstream slope under conditions of dynamic earthquake loading.  
• Factor of Safety during an earthquake decreases because 1) the driving force is increased due to the horizontal seismic load, and 2) the resisting force may be reduced due liquefaction.  
• If the Factor of Safety falls below 1.0 during a portion of the earthquake, the slope undergoes some deformation | • Perform site response analysis (Idriss and Sun, 1992; Hudson et al., 1994)  
• Evaluate liquefaction potential (Youd et al., 2001)  
• Calculate residual strength of liquefied material (Seed and Harder, 1990)  
• Compute slope stability using two-stage pseudostatic approach to compute Factor of Safety (Wright, 1991) or a numerical approach to compute deformation (Makdisi and Seed, 1978; Seed, 1979);  
• Compute post-earthquake slope stability using elevated pore water pressures and residual strengths (Marcuson et al., 1990) | • Factor of Safety > 1.0  
• Acceptable level of deformation subject to engineering judgment |
| Piping failure                              | • Fine-grained material is eroded from within the dam due to preferential flow along conduits and lack of adequate filtering of fine-grained particles.                                                      | • Identify possible conduits, such as voids, cracks, and high-permeability zones  
• Compare grain size information to                                                                                         | • Permeable zones and cracks in the foundation and abutment do not pose a significant threat |
<table>
<thead>
<tr>
<th>Failure Mechanism</th>
<th>Description</th>
<th>General Analysis Method(s)</th>
<th>Acceptance Criteria</th>
</tr>
</thead>
</table>
|                   | • Sand boils may be an indicator of piping.  
• Piping can lead to sudden and catastrophic dam failure | filter criteria (USBR, 1987)  
• Create flownet and calculate effective stress at the toe of the embankment  
• Perform settlement analyses to assess potential for differential settlement in new dams | • Physical evidence of piping is not found  
• Filter criteria are satisfied  
• Quick conditions do not exist  
• Differential settlement in new dams is not expected to be excessive |
| Foundation failure | • Weight of new embankment and reservoir exceed the load-bearing capacity of the foundation and induce failure under undrained conditions.  
• Foundation failure may occur under steady-state conditions due to seams in the foundation that are weak, permeable, or susceptible to piping  
• Foundation failure may occur due to liquefaction during an earthquake | • Perform slope stability analysis of new dams using undrained shear strengths  
• Perform slope stability analyses of new and existing dams using effective strengths and pore water pressures for foundations with suspected low-strength seams  
• Perform stability analyses on circular or non-circular slip surfaces as appropriate  
• Perform seismic analysis using slope stability procedures for seismic case | • Factor of Safety > 1.3 for new dams  
• Factor of Safety > 1.5 for existing dams  
• Factor of Safety > 1.0 for seismic stability  
• Acceptable level of seismic deformation is subject to engineering judgment |
| Overtopping failure | • Principal and emergency spillways cannot accommodate sudden rise in reservoir level.  
• Water level rises above freeboard and flows over the dam, resulting in erosion of downstream slope, compromise in dam integrity, damage to downstream appurtenant structures, and possible destruction of dam | • Compare spillway capacity to design hydrographs | • Freeboard and spillways should be sufficient to accommodate 1) the freeboard hydrograph or 2) the emergency spillway hydrograph as defined in EM5 |
### Table 3. Components of a typical instrumentation and monitoring program for earth dams

<table>
<thead>
<tr>
<th>Component</th>
<th>Typical Installation Frequency (subject to the discretion of the engineer)</th>
<th>Purpose</th>
</tr>
</thead>
<tbody>
<tr>
<td>Piezometers</td>
<td>• As needed to adequately define the phreatic surface and pore pressure field • Monitor changes in phreatic surface and pore pressure field</td>
<td></td>
</tr>
<tr>
<td>Inclinometers</td>
<td>• As needed to monitor areas susceptible to deformation • Monitor deformation within embankment</td>
<td></td>
</tr>
<tr>
<td>Survey Points</td>
<td>• 1 per 50 ft along crest • 10 per acre</td>
<td>• Monitor deformation of dam surface</td>
</tr>
<tr>
<td>Weirs</td>
<td>• Install where seepage has been historically observed • Monitor changes in seepage flow rate • Monitor changes in seepage turbidity</td>
<td></td>
</tr>
<tr>
<td>Crack Displacement</td>
<td>• Install across known cracks</td>
<td>• Monitor deformation of existing cracks</td>
</tr>
<tr>
<td>Measurement</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seismometers</td>
<td>• Optional, depending upon class of dam and site seismicity • Compare predicted versus actual earthquake shaking on embankment</td>
<td></td>
</tr>
<tr>
<td>Feature</td>
<td>Physical Description</td>
<td>Associated Problems</td>
</tr>
<tr>
<td>---------</td>
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<td>---------------------</td>
</tr>
<tr>
<td>Upstream Riprap</td>
<td>• Wave-induced erosion on upstream side due to loss of riprap, or riprap missing completely</td>
<td>• May create opportunities for surface erosion</td>
</tr>
<tr>
<td>Condition of Relief Wells, Spillways, and Conveyance Structures</td>
<td>• Floating debris (e.g. logs, branches, garbage) may have settled in spillway during episode of flooding • Silt may be obstructing spillways or relief wells • Seepage is occurring between conveyance structures and embankment</td>
<td>• Elevated pore water pressures in the embankment may result from obstructed spillways and relief wells and cause slope stability failure • Overtopping of dam may result from obstructed spillways • Piping between conveyance structures and embankment may occur</td>
</tr>
<tr>
<td>Abutment and Weir Seepage</td>
<td>• Seepage is appearing in the abutments on the downstream side of the dam • Groin area is particularly susceptible to abutment seepage • Abutment and weir seepage may have high turbidity</td>
<td>• High turbidity may be an indicator of piping • Chemical analysis of abutment seepage may reveal high total dissolved solids, which is an indicator of limestone dissolution • Development of excess pore pressures in the abutments may lead to slope instability</td>
</tr>
<tr>
<td>Surface Erosion</td>
<td>• Presence of surface erosion on the embankment or abutments due to seepage, excessive grade, or lack of vegetative cover • Groin areas may be particularly susceptible</td>
<td>• May result in loss of ground and reduction in slope stability • Erosional features may create opportunities for piping</td>
</tr>
<tr>
<td>Cracking</td>
<td>• Transverse, longitudinal, or desiccation cracking developing on the embankment surface</td>
<td>• Surface erosion • Longitudinal cracks may be a precursor to slope stability failure • Transverse cracks may be a precursor to piping</td>
</tr>
<tr>
<td>Differential Movement</td>
<td>• Obvious deviations from as-built ground surface shape</td>
<td>• May be an indicator of internal cracking, which could lead to piping</td>
</tr>
<tr>
<td>Feature</td>
<td>Physical Description</td>
<td>Associated Problems</td>
</tr>
<tr>
<td>--------------------------</td>
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<td>--------------------------------------------------------------------------------------</td>
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<tr>
<td></td>
<td>• Crest sagging in zones</td>
<td>• Crest sagging may cause overtopping of dam due to reduction in freeboard</td>
</tr>
<tr>
<td></td>
<td>• Bulging at the downstream toe</td>
<td>• Bulging may be an indicator of slope movement or lateral spreading</td>
</tr>
<tr>
<td>Biological Disturbance</td>
<td>• Cattle paths, rodent burrows, human excavation, or human vandalism</td>
<td>• Cattle paths may cause surface erosion</td>
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<tr>
<td></td>
<td></td>
<td>• Animal burrows may cause piping</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Excavation at toe may reduce slope stability</td>
</tr>
<tr>
<td>Cavity Formation</td>
<td>• Appearance of cavities, voids, and sinkholes on the surface of the embankment</td>
<td>• Strong indicator that piping is occurring within the dam</td>
</tr>
<tr>
<td>Sand Boils</td>
<td>• Small sand “volcanoes” are developing in standing water downstream of the dam</td>
<td>• Elevated pore water pressure in the embankment may cause a slope stability failure</td>
</tr>
<tr>
<td></td>
<td>• Water with suspended solids may be welling up from the center of the sand boils</td>
<td>• Piping is occurring</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Relief wells may not be functioning properly</td>
</tr>
<tr>
<td>Vegetation Patterns</td>
<td>• Cattails and other wetland vegetation appear in patches downstream of the dam</td>
<td>• Wetland vegetation is an indicator of uncontrolled seepage and inadequate drainage, which may lead to a slope stability failure</td>
</tr>
<tr>
<td></td>
<td>• Zones of unusually green and lush ground cover exist on the downstream slope</td>
<td>• Green zones indicate the phreatic surface is near the ground surface</td>
</tr>
<tr>
<td></td>
<td>• Large trees or shrubs are growing on the dam</td>
<td>• Tree and shrub root holes may create opportunities for piping</td>
</tr>
</tbody>
</table>
Figure 1. Failure mechanisms for earth dams

a) slope stability failures

b) piping failure

c) overtopping failure

d) foundation failure
Figure 2. Geotechnical site investigation program for an earth dam
Figure 3. Typical instrumentation for an earth dam
Figure 4. Some features of concern when performing visual inspection of an earth dam