TRENDS IN EXPERIMENTAL DATA OF INTERMEDIATE SOILS FOR EVALUATING DYNAMIC STRENGTH

K. Dahl¹, R.W. Boulanger² and J. T. DeJong³

ABSTRACT

Intermediate soils such as low plasticity silty clays or clayey sands span a range of soil properties and exhibit behavior from clay-like (fines-controlled) to sand-like (coarse-controlled). There is no consensus on the amount of fines and plasticity that defines a soil’s fundamental behavior. This makes it difficult to predetermine the most appropriate method for estimating seismic strengths of these intermediate soils (i.e., liquefaction correlations or cyclic softening procedures). Case-by-case judgement is required and often both methods are used for soils with fines content FC of 20-50% and plasticity index (PI) of 4-12. The compiled consolidation, monotonic and cyclic experimental data provides a reference for understanding the influence of these parameters on behavior of natural soils. Seismic strengths may possible be estimated using the updated relationship by Boulanger and Idriss [1] between cyclic resistance ratio (CRR) versus number of uniform cycles, which is extended to lower values of PI.

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ABSTRACT

Intermediate soils such as low plasticity silty clays or clayey sands span a range of soil properties and exhibit behavior from clay-like (fines-controlled) to sand-like (coarse-controlled). There is no consensus on the amount of fines and plasticity that defines a soil’s fundamental behavior. This makes it difficult to predetermine the most appropriate method for estimating seismic strengths of these intermediate soils (i.e., liquefaction correlations or cyclic softening procedures). Case-by-case judgement is required and often both methods are used for soils with fines content FC of 20-50% and plasticity index (PI) of 4-12. The compiled consolidation, monotonic and cyclic experimental data provides a reference for understanding the influence of these parameters on behavior on natural soils. Seismic strengths may possibly be estimated using the updated relationship by Boulanger and Idriss [1] between cyclic resistance ratio (CRR) versus number of uniform cycles, which is extended to lower values of PI.

Introduction

Intermediate soils such as low plasticity silty clays or clayey sands span a range of soil properties and exhibit behavior from clay-like to sand-like. This makes it difficult to predetermine the most appropriate method for estimating seismic strengths (i.e., liquefaction correlations or cyclic softening procedures). Laboratory testing to estimate seismic strengths is not appropriate for sand-like soils due to effects of sample disturbance while it is appropriate for clay-like soils. Use of laboratory testing for estimating seismic strengths of intermediate soils depends on site geology, field loading conditions and soil characteristics. Case-by-case judgement is required for estimating seismic strengths and often both liquefaction correlations and cyclic softening procedures are used.

The following presents experiment test data compiled from literature for the purposes of 1) to better understand behavior of intermediate soils with varying plasticity index (PI) and fines content (FC), (2) estimate possible behavior in advance of testing, and (3) compare against site knowledge to evaluate if laboratory testing is appropriate method to estimate seismic strengths. Correlations of seismic strengths for fine-grained soils are provided, which are consistent to, and

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The compiled experimental data on natural intermediate soils with FC of 25-100% and PI of 0-30 is listed in Table 1. Typical values are listed for FC and PI. Results of consolidation and monotonic and cyclic undrained tests are presented in the following figures. Results of identified clay-like soils [1] are also shown to bound characteristic behavior. Readers are referred to the individual studies for further details. Soils are generally organised into three groups based on index parameters (FC and PI) and characteristic behavior exhibited.

- Group A soils exhibit clay-like behavior that includes well-defined preconsolidation pressure ($\sigma'_p$), ductile stress-strain responses when normally consolidated (NC), and seismic strength may be estimated from correlations for clay-like soils. These soils are amendable to laboratory testing.
- Group B soils exhibit less well-defined $\sigma'_p$, slight strain-hardening responses when NC, which makes it difficult to define a strength ratio. The use of laboratory testing to estimate in-situ behavior depends on the field conditions and managing effects of sample disturbance.
- Group C soils often exhibit poorly defined $\sigma'_p$, and significant strain hardening responses. Effects of sample disturbance will likely render unreliable laboratory seismic strengths. Thus, seismic strengths should be based on correlations with in-situ test data (SPT, CPT, Vs).

**Consolidation Loading**

Fig. 1 shows the consolidation behavior of seven soils ranging in FC from 45-100% and PI from 2-45. Curves are from constant rate of strain (CRS) and incremental loading (ICL) consolidation tests. The $\sigma'_p$ is easily identifiable for higher FC and PI soils and become less defined (rounded) as these parameters decrease. Typical $C_c$ values ranged between 0.180-0.350. Soils with lower compressibility and more difficult to define $\sigma'_p$ typical had lower $C_c$ values of 0.17-0.28 for Group B soils and < 0.17 for Group C soils.

![Figure 1. Consolidation behavior of soils with varying FC and PI.](image-url)
Table 1. Summary of compiled experimental data and parameters

<table>
<thead>
<tr>
<th>Reference</th>
<th>Soil characteristics</th>
<th>Consolidation</th>
<th>Monotonic</th>
<th>Cyclic</th>
<th>For 10 uniform cycles</th>
<th>For 30 uniform cycles</th>
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<tbody>
<tr>
<td>[5]</td>
<td>San Francisco Bay Mud (SFBM)</td>
<td>MH 100</td>
<td>45</td>
<td>74</td>
<td>1.5</td>
<td>ICU TX</td>
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<td>[6]</td>
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<td>MH 100</td>
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<td>DIS</td>
</tr>
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<td>[7]</td>
<td>Greens Creek clayey silt</td>
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<td>11</td>
<td>ICU TX</td>
<td>460</td>
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<td>[8]</td>
<td>Stratum A</td>
<td>CL &amp; ML</td>
<td>93</td>
<td>18</td>
<td>180</td>
<td>0.33</td>
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<tr>
<td>[9]</td>
<td>Moss Landing</td>
<td>CL</td>
<td>88</td>
<td>18</td>
<td>36</td>
<td>0.326</td>
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<tr>
<td>[10]</td>
<td>Tuttle Creek</td>
<td>CL/ML</td>
<td>96</td>
<td>7</td>
<td>79</td>
<td>0.172</td>
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<td>[11, 12]</td>
<td>FRD silt</td>
<td>Site A</td>
<td>ML</td>
<td>90</td>
<td>4</td>
<td>80</td>
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<tr>
<td>[13]</td>
<td>Stratum B</td>
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<td>35-50</td>
<td>&lt;3</td>
<td>DIS</td>
<td>2</td>
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<td>[14]</td>
<td>Silt-Clay of Adapazari</td>
<td>CL &amp; ML</td>
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<td>16</td>
<td>ACU TX</td>
<td>90 to 170</td>
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<td>[15]</td>
<td>BC Sh</td>
<td>ML</td>
<td>80</td>
<td>7</td>
<td>DIS</td>
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<td>[16]</td>
<td>Adapazari Sites A, C, D, G, I, &amp; J</td>
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<td>150</td>
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<td>120</td>
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<tr>
<td></td>
<td>Site A middle lower</td>
<td>CHL, ML</td>
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<td>97</td>
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<td>Site C upper (loose)</td>
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<td>Site F upper (loose)</td>
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<td>Site C upper (loose)</td>
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<td>78</td>
<td>6</td>
<td>NA</td>
<td>ACU TX</td>
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<tr>
<td></td>
<td>Site C upper (loose)</td>
<td>CH</td>
<td>97</td>
<td>1</td>
<td>30</td>
<td>0.19</td>
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<tr>
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<td>Site C upper (loose)</td>
<td>CL &amp; ML</td>
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<td>5</td>
<td>175</td>
<td>0.23</td>
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<tr>
<td></td>
<td>Site C middle upper (loose)</td>
<td>CL &amp; ML</td>
<td>70</td>
<td>5</td>
<td>175</td>
<td>0.23</td>
</tr>
<tr>
<td>[17]</td>
<td>Riccarton Sh</td>
<td>ML</td>
<td>68</td>
<td>6</td>
<td>DIS</td>
<td>3.0</td>
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<td>[18]</td>
<td>Innis-Clayey silt</td>
<td>CL &amp; ML</td>
<td>57</td>
<td>8</td>
<td>95</td>
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<tr>
<td>[19]</td>
<td>PD Shallow</td>
<td>BD to SC-SH</td>
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<td>9</td>
<td>100-450</td>
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</table>
Monotonic Undrained Loading

Figs. 2 (a, b, and c) show the stress-strain responses of NC (OCR=1-1.4) and overconsolidated (OCR=1.5-4) Group A, B and C specimens, respectively. Monotonic undrained testing was performed under direct simple shear (DSS) and isotropically or anisotropically consolidated triaxial (ICU & ACU TX). Group A soils with FC of 80-100% and PI of 5-45 exhibited more ductile stress-strain response. Group B soils with PI of 1-14 and FC of 55-96% exhibit slight strain-hardening response. Group C soils with lower PI of 1-13 and FC of 27-93% exhibited generally strain-hardening response.

Figure 2. Monotonic undrained loading of Group A, B and C soils.
Fig. 3 shows the compiled normalized undrained strength ratio \( \frac{s_u}{\sigma'_{vc}} \) versus PI for Group A and B soils. The \( s_u \) is typically defined at \( \gamma = 15\% \). In general, Group A specimens have \( \frac{s_u}{\sigma'_{vc}} \) of 0.24-0.29 at OCR = 1.0 and 0.44-0.53 at OCR = 2.0. Group B have \( \frac{s_u}{\sigma'_{vc}} \) of 0.24-0.29 for NC and \( \frac{s_u}{\sigma'_{vc}} \) of 0.44-0.53 at OCR = 2.0.

**Figure 3. Undrained strength ratio by PI for Group A, B and C soils.**

**Cyclic Undrained Loading**

Figs. 4 (a and b) shows the cyclic resistance ratio (CRR) to trigger ~3\% shear strain (5\% axial strain) versus number of uniform cycles (N) for NC Group A soils with PI of 21-73 and 6-18, respectively.

**Figure 4. Cyclic resistance ratio versus number of cycles for Group A NC soils.**
For direct comparison, triaxial results were corrected to an equivalent DSS in Fig. 4 (c and d) using a correction factor ($c_r$) of 0.65 [2] [3]. Plots of CRR$_{DSS}$ versus N for specimens with a PI of 21-73 converge together with (CRR$_{DSS}$)$_{N=30}$ of 0.146 to 0.224 (average 0.183) for soils with OCR = 1-1.4, which is the same value as recommended by for NC clays [1]. Plots for specimens with PI of 6-18 also converge together with slightly lower (CRR$_{DSS}$)$_{N=30}$ of 0.120 to 0.217 (average of 0.165).

Figs. 5 (a and b) shows the CRR versus N for OC Group A soils with PI of 20-27 and 10-18, respectively. These results were converted to equivalent DSS in Figs. 5(c and d) using $c_r=0.65$ and OCR.

Figs. 6 (a and b) shows the CRR versus N of Group B and C soils with FC of 85-94% and FC of 57-79%, respectively. Figs. 6 (c and d) shows the equivalent CRR$_{DSS}$ versus N.

Figure 5. Cyclic resistance ratio versus number of cycles for Group A OC soils.

Figs. 6 (a and b) shows the CRR versus N of Group B and C soils with FC of 85-94% and FC of 57-79%, respectively. Figs. 6 (c and d) shows the equivalent CRR$_{DSS}$ versus N.
Figs. 7 (a and b) shows the CRR at N of 10 and 30 versus PI, respectively. The CRR of NC Group B soils tend to decrease with PI but are generally consistent with values for Group A soils for a PI ≥ 7. Above PI ≥ 7 the CRR is 0.206 at N=10 and 0.184 at N=30, which is similar to the CRR of 0.183 recommended by Boulanger and Idriss [1] for NC clay. As the PI decreases to zero the CRR decreases to 0.105 and 0.093 for N of 10 and 30, respectively, which are consistent with values estimated from liquefaction correlations charts [4].

Figure 7. CRR with PI for NC Group A and B soils (a) N=10 and (b) N=30.

The cyclic strength ratio ($\tau_{cyc}/s_u$) at N of 10 and 30 versus PI are shown in Figs. 8 (a and b), respectively, for Group A and clay-like soils [1]. The $\tau_{cyc}/s_u$ ratios are generally similar for
Group A soils with a PI ≥ 18 and tend to decrease with decreasing PI. Results are fitted with a \( \tau_{cyc/su} \) trendline using the power fit b parameter \([CRR=aN^{-b}]\) of 0.104 from DSS tests. The \( \tau_{cyc/su} \) trendline at N = 10 in Fig. 8(a) is 0.897 for PI ≥ 18, and decreases to 0.685 at PI of 10. The respective \( \tau_{cyc/su} \) trendline in Fig. 8(b) at N = 30 is 0.800 for a PI ≥ 18, and decreases to 0.611 at PI of 10. This trendline is consistent with that recommended by Boulanger and Idriss [1] for soils with a PI ≥ 18 and is extended with data with PI of 6. Figs. 8 (c and d) show the \( \tau_{cyc/su} \) at N of 10 and 30 versus PI for selected Group B soils using the average \( s_u \) defined at a strain of 15%. The \( \tau_{cyc/su} \) of Group B soils tends to follow the same trendline with lower values than Group A soils.

**Figure 8.** Values of \( \tau_{cyc/su} \) with PI for Group A (a) N=10, (b) N=30 and Group B soils (c) N=10, (d) N=30.

**Conclusions**

The experimental trends in consolidation, monotonic and cyclic test data may be used to better understand characteristic behavior of intermediate soils. This compilation may serve to estimate behavior in advance of testing. Deciding if laboratory testing is appropriate method for estimating seismic strengths of an intermediate soils depends the soil characteristics, site geology, managing effects of sample disturbance and field loading conditions. Readers are referred to each study for details related to the experimental data.

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