Proper Selection of Rigidity Index 
(Ir=G/Su) Relating to CPT Correlations

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Introduction

Cone Penetration testing has proven to be a very useful in-situ testing method, that can be used for many applications. These applications range from stratigraphic profiling to empirical correlations for a variety of engineering properties in both fine and coarse grained material. A substantial amount of research has been focused on developing theoretical and analytical methods that allow for proper modeling of the movement of soil around the advancing cone, of which the primary methods utilized are the Cavity Expansion Theory, Strain Path Theory, and Bearing Capacity Theories (Lu et. al. 2004, Yu et al. 1998, and Yu et al. 2000.) . Although each of these methods has simplifying assumptions, some have proven to provide fairly accurate predictions of in-situ parameters. The difficulty of the problem is the amount of factors involved with the physical process of deforming soil around an advancing cone. One such factor, that is very influential in the analysis of nearly all theories, is the Rigidity Index, G/Su. Although this
term has propagated through nearly every CPT analysis theory, there is much uncertainty in the proper selection of the shear modulus and undrained strength to be used in the analyses. This paper will discuss the effect of the Rigidity Index on dissipation tests and the cone factor, Nkt, original intent of the Rigidity Index, and the various methods used to determine the Rigidity Index. In conclusion, data from Newbury BBC will be used to compare lab values of G/Su to those predicted using various methods.

**Rigidity Index Background**

As a cone is advanced through soil, a region of soil is plastically deformed to some extent around the cone. This failure zone is typically defined by an elastic-plastic boundary, so that beyond the plastic deformation there is an elastic region of soil. As Teh and Houlsby (1991) and others have recognized, this failure zone depends on many factors, but it is largely a function of the rigidity index (Teh 1987, Schnaid et al. 1997, Yu et al. 1998). The figure below illustrates the plastic deformation caused by cone penetration as originally proposed by Vesic (1977) for the Cavity Expansion theory.

![Figure 2: Vesic (1977)](image)

Before the rigidity index is further defined, the application of the term in regards to the determination of the coefficient of consolidation, c_h, and the theoretical cone factor, N_kt, will be discussed.
**Dissipation tests (ch)**

The coefficient of consolidation, $c_h$, and time for 50% consolidation, $t_{50}$, are determined by performing dissipation tests in fine grained soils due to the excess pore pressure created by the advancing cone. Teh and Houlsby (1991) reanalyzed the dissipation of excess pore pressures from cone penetration and determined that the size of the zone, in which excess pore pressures are created, is directly a function of the Rigidity Index. Teh (1987) noted, as the Rigidity Index increases, the extent of the failure zone increases as well. Due to a larger volume of soil being displaced, the excess pore pressures also increase with Rigidity Index. This in turn increases the time of consolidation, or lowers the estimate of the coefficient of consolidation. Teh and Houlsby (1991) empirically developed the modified time factor equation, which allowed for unification of various dissipation curves regardless of the Rigidity Index. The dissipation curve is plotted against this modified time factor in Figure 1 below, illustrating that one unique curve can be obtained using $u_2$ element position.

$$T^* = \frac{c_h t}{a^2 \sqrt{t}} \quad \text{(Eq. 1)}$$

![Figure 1: Normalized Dissipation Curves plotted against $T^*$ (Teh & Houlsby 1991)](image)

One issue with the curve was that the coefficient of consolidation at 50% of consolidation is unknown and therefore the curve will shift to the left or right depending on the selection of the rigidity index. Due to the wide range of known rigidity index values ranging from approximately
50 to 500, a method for proper selection of rigidity index becomes vital to the analysis (Teh & Houlsby 1991, Robertson et. al. 1992, Schnaid et. al. 1997).

Robertson et. al. (1992) analyzed the effectiveness of the modified time factor equation proposed by Teh and Houlsby (1991) by comparing the theory to a large database of soils from around the world. Figure 3 illustrates the findings of Robertson et al (1992), showing the increased time of consolidation with an increase in Rigidity Index. Using the lines for Ir=50 and Ir=500, the effect of Rigidity Index on the determination of \(c_h\) and \(t_{50}\) was approximated.

<table>
<thead>
<tr>
<th>(t_{50}(Ir=500) / t_{50}(Ir=50)) ((c_h) constant)</th>
<th>(c_h(Ir=500) / c_h(Ir=50)) ((c_h) constant)</th>
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</thead>
<tbody>
<tr>
<td>2.22</td>
<td>3.33</td>
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</tbody>
</table>

**Table 1: Interpretation of Figure 3**

Figure 3: Average lab oedometer \(c_h\) results and CPTu results in terms of \(t_{50}\) for \(u_2\) pore-pressure location. Robertson (1992)
From the interpretation of Figure 3, it is apparent that the selection of the rigidity index can have a large effect on the prediction of the coefficient of consolidation. Since the rigidity index values of 50 and 500 represent the typical range of possible Ir values in clay, one would expect that the selection of rigidity index using known methods would allow you to reduce this range of possible values for t50 and ch. However, the range of Ir values estimated using typical methods can provide very inaccurate predictions of Ir. This is explained further in the sections on estimating the rigidity index. Another example of the importance of proper selection of the Rigidity Index, is for the theoretical cone factor, Nkt.

**Theoretical Cone Factor, Nkt**

One very common correlation utilized for interpreting CPT measurements, is the determination of the undrained strength using the cone factor, Nkt. The strength is estimated using the tip resistance, vertical initial stress, and a cone factor as illustrated in equation 2.

\[
Nkt = (q_c - \sigma_{vo})/S_u \quad (\text{Eq. 2})
\]

In practice, it is typical to utilize an empirical cone factor determined from reference strength values determined from lab or other in-situ tests. The cone factor can range from approximately 6 to 20 (Lu et. al. 2004). However, it is also useful to determine a theoretical cone factor for comparison and to better understand the physical processes involved with the determination of the cone factor. These theoretical Nkt factors have been successfully derived from both Cavity Expansion and Strain Path methods (Yu et. al. 1998). For the purposes of this paper, the most current theoretical derivation for Nkt will be used from Lu et. al. (2004).

The derivation for Nkt was modeled after assuming a low compressibility, homogenous elastic-perfectly plastic material following the Tresca Yield criterion. The derivation takes in to account the cone roughness, \(\alpha_c\), rigidity index, I_r, and initial stress anisotropy, \(\Delta\).

\[
Nkt \approx 3.4 + 1.6 \ln(I_r) - 1.9\Delta + 1.3\alpha_c \quad (\text{Eq. 3})
\]

The effect of the rigidity index is illustrated below in figure 4, with varying degrees of cone roughness and initial stress anisotropy over the typical range of Ir for clays of 50 to 500.
The plot shows an approximate 40% increase in the estimation of Nkt over the typical range of Ir values of clay. As stated earlier, this illustrates the significance of properly determining the rigidity index.

![Graph showing variation of Nkt with soil rigidity index (Lu et. al. 2004)](image)

**Figure 4: Variation of Nkt with soil rigidity index (Lu et. al. 2004)**

Therefore, the proper selection of rigidity index has been shown to be very significant in the determination of the theoretical cone factor and in determining the coefficient of consolidation correctly. The issue then becomes how to properly select the rigidity index based on the penetration of a cone. The remainder of the paper will describe what needs to be accounted for when properly selecting the rigidity index and review the methods available for estimating the rigidity index.

**Selection of Rigidity Index Parameters G and Su**

Methods involving the displacement of soil around an advancing cone have been shown to have a large sensitivity to the rigidity index. All methods developed using this term assume a soil that behaves as a linear elastic perfectly plastic soil. In reality, the elastic behavior is nonlinear and is a function of the strain path and amount of strain (Schnaid et. al. 1997, Yu et al 2000, Mayne 2001). However, using a constant rigidity index allowed for simplification of the methods that were developed. The question then becomes which single shear modulus (G) and reference strength (Su) represent the deformation of soil around an advancing cone the best.
Selection of Undrained Strength, Su

It is well established that the value of the undrained strength is unique to the test being performed and the most representative strength is typically used in engineering design. The question then becomes which strength test best represents an average strength being mobilized throughout the soil around the advancing cone. Table 2 below is a collection of recommended tests for determining the reference strength, Su.

<table>
<thead>
<tr>
<th>Author</th>
<th>Su test</th>
<th>Reasoning</th>
<th>Assumptions/Applicability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Keaveny (1985)</td>
<td>CkoU Triaxial Compression</td>
<td>Average Su mobilized around an advancing cone</td>
<td>Based on accurate predictions of Su using Vesic (1975,1977) bearing capacity Failure mode</td>
</tr>
<tr>
<td>Konrad &amp; Law (1987)</td>
<td>Pressuremeter</td>
<td>Produces results similar to CkoU Triaxial Compression</td>
<td></td>
</tr>
<tr>
<td>Schnaid et al. (1997)</td>
<td>CkoU</td>
<td>Not Specified</td>
<td>Strain Path Method &amp; Spherical Cavity Expansion</td>
</tr>
<tr>
<td>Yu et. al. (2000)</td>
<td>Undrained Triaxial Compression</td>
<td>Not Specified</td>
<td>Strain Path Method and Finite Element Analysis</td>
</tr>
</tbody>
</table>

Although earlier methods used different reference strengths, the use of anisotropic undrained triaxial compression appears to be the most representative strength around an advancing cone. Keaveny (1985) based the finding on accurate predictions of Su using the Vesic (1975,1977) bearing capacity approach and the failure zones around a cone first illustrated by Baligh (1984) in Figure 5 below. In Figure 6, Wroth (1984) illustrated that the CkoU Triaxial
test produces an average response as well. Later methods suggested the use of Undrained Triaxial Compression as the reference strength, but they were not specific as to whether it should be the anisotropic or isotropic test. Therefore, based on the findings of Keaveny (1985), it is recommended that CkoU Triaxial Compression be used as the reference strength for the rigidity index.

Figure 5: Predominant Failure Modes Around Advancing Probe (Baligh, 1984)

source: (Keaveny & Mitchell 1986)
Selection of Shear Modulus, $G$

The selection of the shear modulus that best represents a linear elastic perfectly plastic soil is the primary difficulty involved with the estimation of the rigidity index. The shear modulus is a function of the strain level, aging effects, and various other factors (Wroth et. al. 1979, Schnaid et. al. 1997). For undrained loading, where the poisson’s ratio is 0.5, the following relationship between the Youngs Modulus and Shear Modulus is applicable.

$$Gu = \frac{E_u}{2(1+\nu)} = \frac{E_u}{3} \quad \text{(Eq. 4)}$$

Figure 7 illustrates the different selections of the shear modulus based on a stress-strain response of a soil during testing. The Gmax, or initial tangent modulus, represents very low strain and therefore the largest shear modulus. As the strain level increases towards failure, the shear modulus decreases and is interpreted using a secant modulus. The reduction of shear modulus with strain is further illustrated in Figure 8 by Mayne (2001).
The rigidity index is used to describe the elastic response of the soil being affected by the advancement of the cone, so a value representative of this response is desired. Back calculated elastic moduli and shear moduli are not applicable in this case, since the strains are very small at depth and would suggest very high values of $G$. Table 3 is a summary of recommended $G$ values found in literature.
<table>
<thead>
<tr>
<th>Author</th>
<th><strong>G</strong> Recommended</th>
<th>Reasoning</th>
<th>Assumptions/Applicability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meyerhoff (1951)</td>
<td>Go Initial Tangent Modulus</td>
<td>(As referenced in Konrad &amp; Law 1987)</td>
<td>Spherical Cavity Expansion Theory</td>
</tr>
<tr>
<td>Skempton (1951)</td>
<td><strong>G</strong>ₐ₀ (Modulus at 50% deviatoric stress)</td>
<td>(As referenced in Konrad &amp; Law 1987)</td>
<td>Spherical Cavity Expansion Theory</td>
</tr>
<tr>
<td>Vesic (1972)</td>
<td>Go Initial Tangent Modulus</td>
<td>(As referenced in Konrad &amp; Law 1987)</td>
<td>Cavity Expansion Theory assuming soil outside of plastic boundary is elastically deforming at very small strain</td>
</tr>
<tr>
<td>Keaveny (1985)</td>
<td><strong>G</strong>ₐ₀ (Modulus at 50% deviatoric stress)</td>
<td>No Reasoning Provided</td>
<td></td>
</tr>
<tr>
<td>Konrad &amp; Law (1987)</td>
<td><strong>G</strong>ₐ₀ (Modulus at 50% deviatoric stress)</td>
<td>“The use of Eₐ₀ is considered more reasonable because the modulus mobilized outside the plastic zone ranges from Eₐ at the plastic-elastic interface to Eᵢ at infinity or a considerable distance away. The average mobilized value lies between these limiting values and is probably better approximated by Eₐ₀,” Pg 399</td>
<td>Cavity Expansion Theory</td>
</tr>
<tr>
<td>Schnaid et al (1997)</td>
<td>OC Clay: <strong>G</strong>ₐ₀ (Modulus at 50% deviatoric stress)</td>
<td>(Also shown by Baligh and Levadoux 1986 and Jamiolkowsky et al 1985) Provides a means of averaging out strain levels in plastic zone.</td>
<td>Based upon cₗ in field about equal to cₗ in lab</td>
</tr>
<tr>
<td></td>
<td>NC Clay: Gₓₓ≥Gₐ₀ (Modulus at 50% deviatoric stress)</td>
<td>When the Gₐ₀ secant modulus is used the ratio of lab to field ch is about 4 to 6. This ratio increases significantly below 50% stress. Pg 326</td>
<td></td>
</tr>
<tr>
<td>Yu, Mitchell (1998)</td>
<td><strong>G</strong>ₐ₀ (Modulus at 50% deviatoric stress)</td>
<td>No Reasoning Provided</td>
<td></td>
</tr>
</tbody>
</table>
The most common shear modulus selected for the rigidity index has been the secant modulus at 50% of the stress to strength. Earlier methods suggested the use of Gmax, but these assume that there are very small strains in the elastic region outside of the plastically deformed region of soil around the cone and were based on bearing capacity. In reality, the modulus at the boundary is the G at failure and increases to Gmax at some distance away. Therefore, taking a secant modulus at 50% stress between these values seems to provide an average response from the soil in that range (Konrad & Law 1987, Schnaid et. al. 1997). Schnaid et. al. (1997) used lab data to support the G50 assumption, and concluded it was more applicable to OC clays rather than NC clays. The data showed the ratio of chlab/chfield was approximately 1 for OC clays, while the ratio was approximately 4 to 6 for NC clays. They also determined that the ratio between lab and field ch values differed much more when a higher secant modulus (i.e. G25) was used to approximate the rigidity index. Assuming that the lab can approximate the same ch as a field test, this suggests that a shear modulus at a higher stress level may better approximate the relationship for NC clays. Robertson et. al. (1992) showed that CPTU vales of ch are slightly higher than the lab values of ch, but that they agree better with back calculated ch values from full-scale performance. Although this may illustrate that the assumption in Schnaid et. al. (1997) could be incorrect, the G50 assumption still appears to provide reasonable estimates of ch.

Therefore, it is apparent that a secant modulus at 50% stress to failure provides reasonable estimates for clays, but more research needs to be done to determine if a better approximation of G may provide better determination of the rigidity index within OC and NC clay soils. Since the soil response is simplified to choosing a single representative value of G, it should be expected that there could always possibly be some error in the analysis.

Another approach acknowledged by Keaveny (1985) and Yu et. al. (2000) to solve this issue suggests that a nonlinear approach could be used for selecting the shear modulus, such as the strain dependent modulus reduction curve. Seismic CPT could be used to define the Gmax on the modulus reduction curve as shown in equation 5.

\[
G_{max} = \rho \times V_s^2 \quad \text{(Eq 5)}
\]
The use of this type of relationship relies on the determination of the average amount of strain being induced on the soil around the cone however. More research could be performed to determine a relationship for the average amount of strain induced by the penetration of a cone. In cases where approximate estimates of the Rigidity index are desired, empirical and analytical methods may be utilized. These methods introduce much more error however in the estimation of the rigidity index.

**Empirical Methods for Determining Rigidity Index, \( I_r \)**

The rigidity index has been shown to be a function of sensitivity, OCR, PI, and organic content of a clay (Wroth et. al. 1979). Therefore estimation methods need to take these parameters into account. One commonly used empirical method for estimating the rigidity index was developed by Keaveny (1985) and suggested for use by Keaveny & Mitchell (1986) for estimating the undrained strength of clay. Figure 9 shows the relationship between the rigidity index, OCR, and PI based on Ko-consolidated triaxial compression, from which Mayne (2001) developed an equation to represent. Keaveny (1985) suggested that this plot produces estimates of the rigidity index within +/- 50%, but to verify the estimate, the original data will be reviewed.

\[
I_r \approx \frac{\exp\left(\frac{137-I_p}{23}\right)}{1+\ln\left[1+\left(\frac{OCR+1}{26}\right)^{3.2} \cdot 0.8\right]}
\]

(Mayne 2001)

![Figure 9: Estimation of Rigidity Index (Keaveny 1985) source: (Mayne 2001) - Keaveny & Mitchell (1986): CKo UC Triaxial Data](Mayne 2001)
Duncan & Buchignani (1976) proposed a relationship between the bulk modulus, OCR, and PI based on elastic modulus values back calculated from settlement performance of foundations at various sites. These elastic modulus values therefore are representative of the initial tangent modulus due to the low strains induced on the soil at depth. The reference strengths varied between each site used to develop their relationship, and were performed by tests that produce higher strengths than a CKoUTC test would predict (Keaveny 1985). In order to best fit the data from CKoTC data on this plot, Keaveny (1985) suggested that the following relationship between the bulk modulus and rigidity index was required.

\[
Ir = \frac{\frac{K}{2}}{2(1+\nu)} = \frac{K}{6} \quad (Eq. 7)
\]

This assumes that G50 can be approximated by dividing Gmax by 2. However, the relationship between the values of G depend on the stress-strain response and can vary dramatically as is illustrated in the next section. Figures 10 and 11 illustrate the correlations developed by Duncan and Buchignani (1976) and the data plotted by Keaveny vs K/2 respectively.

![Figure 10: E/Su vs OCR and PI (Duncan & Buchignani 1976)](image-url)
Figure 11 includes data plotted from 5 test sites, from which a significant amount of scatter can be seen when compared to the suggested curves. It should also be noted that there is a lack of data in the OC range as well. The scatter suggests that for an approximately NC soil with a PI of 30 the value of Ir can range from approximately 75 to 200, or larger than a factor of 2. Analytical methods have also been developed for estimating the rigidity index.

**Analytical Methods for Determining Rigidity Index, Ir**

Various analytical methods have also been developed for estimating the rigidity index. One such method illustrated by Mayne (2001) is based on the Cam Clay model. This method has a similar appearance to the method proposed by Keaveny (1985), with Ir being a function of OCR.
Caution should be used in the use of these analytical methods and other estimation methods as well, since natural variability in soils may result in dramatically different stress-strain results that cannot be predicted. An example of how variable a site may be over a very small area is illustrated by the tests performed by Schnaid et. al. (1997). Two tests were performed on samples obtained from a single hole at a depth of 6 and 8 meters. Due to the close proximity of the soils and similar OCRs, one would assume similar responses. However, the soil exhibited dramatically different results. The 6 meter sample exhibited a constant rigidity index over a range of strain, whereas the sample from 8 meters began softening immediately and therefore exhibited a decreasing rigidity index.
Now that some methods have been introduced for estimating the rigidity index, the methods will be compared to get an approximate idea of how accurate the assumptions for selecting Ir are.

**Comparison of Methods: Newbury Boston Blue Clay**

**Description of Site and Data Utilized**

In order to verify the assumptions used to estimate the rigidity index and to compare the various methods, data on Newbury Boston Blue Clay will be utilized. The following data was prepared by Landon (2007), from which all lab tests were performed on undisturbed block
samples and SCPTU were performed. The Newbury site consists of an overconsolidated crust overlying a soft silty clay as shown in figure 14 below.

![Soil Profile for Newbury, MA BBC (Landon 2007).](image)

The values of $G_{50}$ and $SU$ were determined from conventional anisotropic undrained triaxial compression tests (CAUC) and are compared to $G_{\text{max}}$ values calculated from the shear wave velocities determined from the Seismic CPTU. Table 4 includes the shear stress, indicated as $SU$, and $G$ values at failure and at 50% stress. The rigidity index values were then determined by dividing the $G$ by the maximum $SU$, or undrained shear strength.

**Discussion of Results**

The first thing to note is that the $G_{50}/G_{\text{max}}$ assumption for the samples at a depth of 7.6 and 9.6 m, agree closely with the assumption made by Keaveny (1985) for $G_{50}/G_{\text{max}} = 0.5$. The samples at depths of 5.6 and 6 m however produced $G_{50}/G_{\text{max}}$ values that are approximately 50% of the $G/G_{\text{max}}$ assumption made by Keaveny (1985). It should be noted that the $G/G_{\text{max}}$ is decreasing with an increase in the overconsolidation ratio, and therefore the rigidity index values are also decreasing with OCR. Although the deeper and less overconsolidated samples predict similar $G_{50}/G_{\text{max}}$ values of 0.5, the rigidity index values predicted by estimation methods is less
accurate than the more overconsolidated samples that exhibited smaller $G/G_{\text{max}}$ values. Therefore, the $G/G_{\text{max}} = 0.5$ assumption may not be the best assumption for the Newbury BBC data. The smaller $G/G_{\text{max}}$ for the less OC samples is most likely due to more strain occurring, which reduces $G_{50}$.

One key observation is that the methods predicted rigidity index well for the higher OC samples, while the estimation was much worse for the less OC samples. This may be illustrating the conclusion made by Schnaid et. al. (1997), in which the $G_{50}$ assumption holds well for OC soils, but may not be the best estimate for NC soils.

The analytical and empirical estimation methods used for the analysis predict similar values of the rigidity index and exhibit very similar responses as can be seen in the shape of the curve on Figure 15. One possible explanation as to why the methods are not able to estimate the rigidity index at this site, may be due to the cementation within the Newbury BBC site.

<table>
<thead>
<tr>
<th>Block Sample</th>
<th>Depth (m)</th>
<th>$G'_{vo}$ (kPa)</th>
<th>$\rho$ (Mg/m$^3$) from Table 8.1</th>
<th>Vs (SCPTu @ 5.6m)</th>
<th>Gmax $= \rho^*V_s^2$</th>
<th>Su (kPa)</th>
<th>$\varepsilon$ (axial) (%)</th>
<th>G (kPa)</th>
<th>I$_r$ = G/Su max</th>
<th>$G/G_{\text{max}}$</th>
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<tr>
<td>N2SBS2</td>
<td>5.6</td>
<td>66.7</td>
<td>1.81</td>
<td>133</td>
<td>32162</td>
<td>52.4</td>
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<td>33</td>
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<td>26.2</td>
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<td>6674</td>
<td>127</td>
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<td>46.5</td>
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</tbody>
</table>

Table 4: Calculation of $I_r = G_{50}/Su$ and $G/G_{max}$ for Newbury BBC
Table 5: Comparison of Ir = G50/Su Estimation Methods to CAUC Lab values for Newbury BBC

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>OCR (Table 4.5)</th>
<th>PI (Table 4.1)</th>
<th>eo (Table 4.4)</th>
<th>Cc [Determined from CRS plots]</th>
<th>Ir = G50/Su max (CAUC Lab Tests)</th>
<th>Keaveny (1985) Chart Solution</th>
<th>(Mayne 2001 eqn for Keaveny (1985) chart)</th>
<th>[Mayne 2001] Analytical Method based on Critical State Mechanics</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.6</td>
<td>4.32</td>
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<td>62.8</td>
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<td>150</td>
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<td>1.26</td>
<td>0.3</td>
<td>302.4</td>
<td>149</td>
<td>78.6</td>
<td>183.3</td>
</tr>
</tbody>
</table>

Figure 15: Comparison of Rigidity Index Estimation to Lab values vs OCR
Conclusion

A thorough literature review on the rigidity index, relating to cone penetration tests, has been performed in order to determine the proper selection of the shear modulus and the reference undrained strength. The findings suggest that Ko consolidated undrained triaxial compression tests produces the best average response to the deformation of soil around an advancing cone. The shear modulus at 50% stress, G50, has been suggested for use because it has been thought to provide an average response of the soil in the elastic region around the cone (Shnaid et. al. 1997, Konrad & Law 1987).

Current empirical correlations, e.g. Keaveny (1985) chart, relating the rigidity index to OCR and PI have been shown to have some errors in estimating the Rigidity Index for the Newbury BBC site for the less overconsolidated samples. The error may be due to cementation at the site, resulting in a non ‘ordinary’ clay response. The analytical critical state mechanics method illustrated in Mayne (2001) appears to have predicted the rigidity index values slightly better than the Keaveny (1985) chart.

Additional sites will need to be analyzed in order to verify if the G/Gmax = 0.5 and G50 assumptions are valid for OC and NC clays. Additional methods for estimating the stress-strain response of the soil should also be included in further research.

References


