Cambridge Self-Bore Pressuremeter

Session 3 – Strength of clays and sand

Strength of Clays and Sand

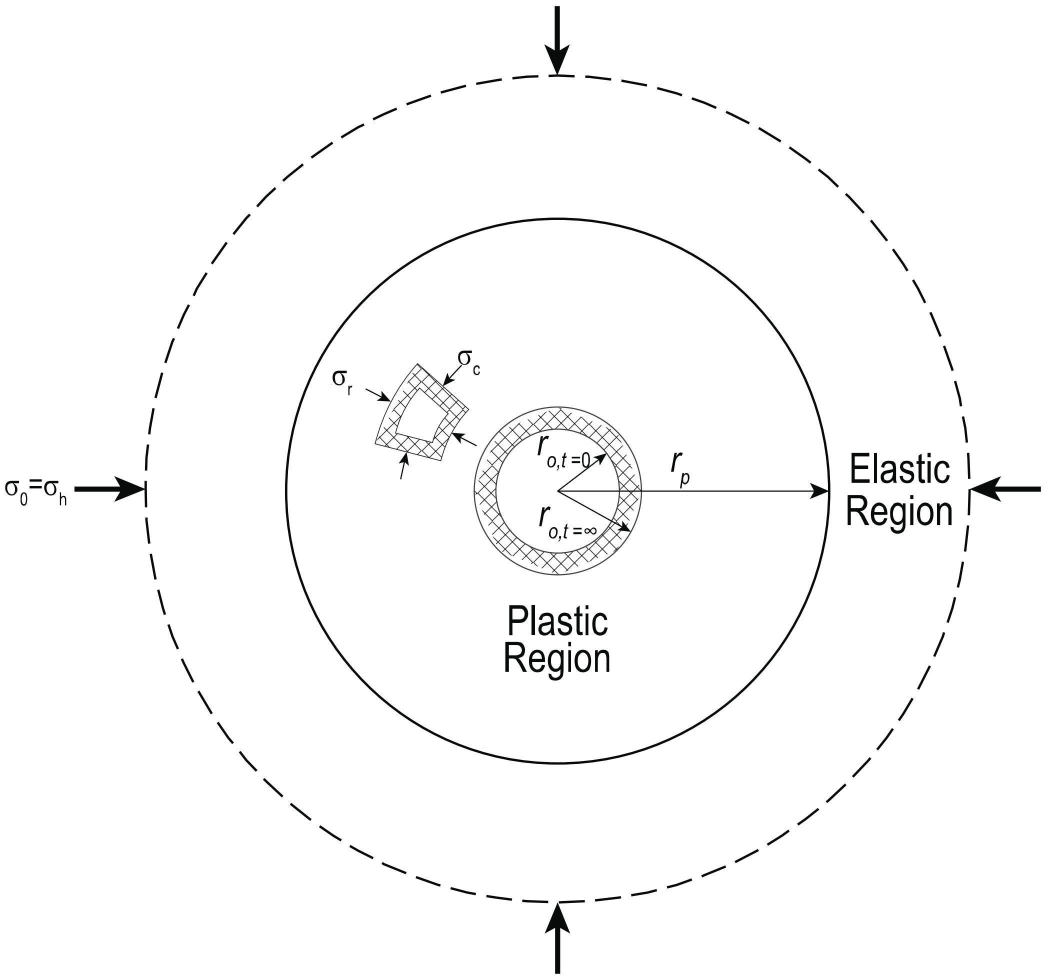
- In this Session, we will discuss the following:
  - Introduction
  - Gibson & Anderson (1961) on clays
  - Jefferies 1988 on contraction
  - Bolton & Whittle (1999)
  - Gibson & Anderson (1961) on sands
  - Hughes et al 1977 (expansion)
  - Houlsby et al (1986) contraction
  - Manassero (1989) numerical analysis
  - Carter et al (1986) c'- phi material
  - Yu & Houlsby (1990)
  - Connection to rocks

Strength of Clays and Sand

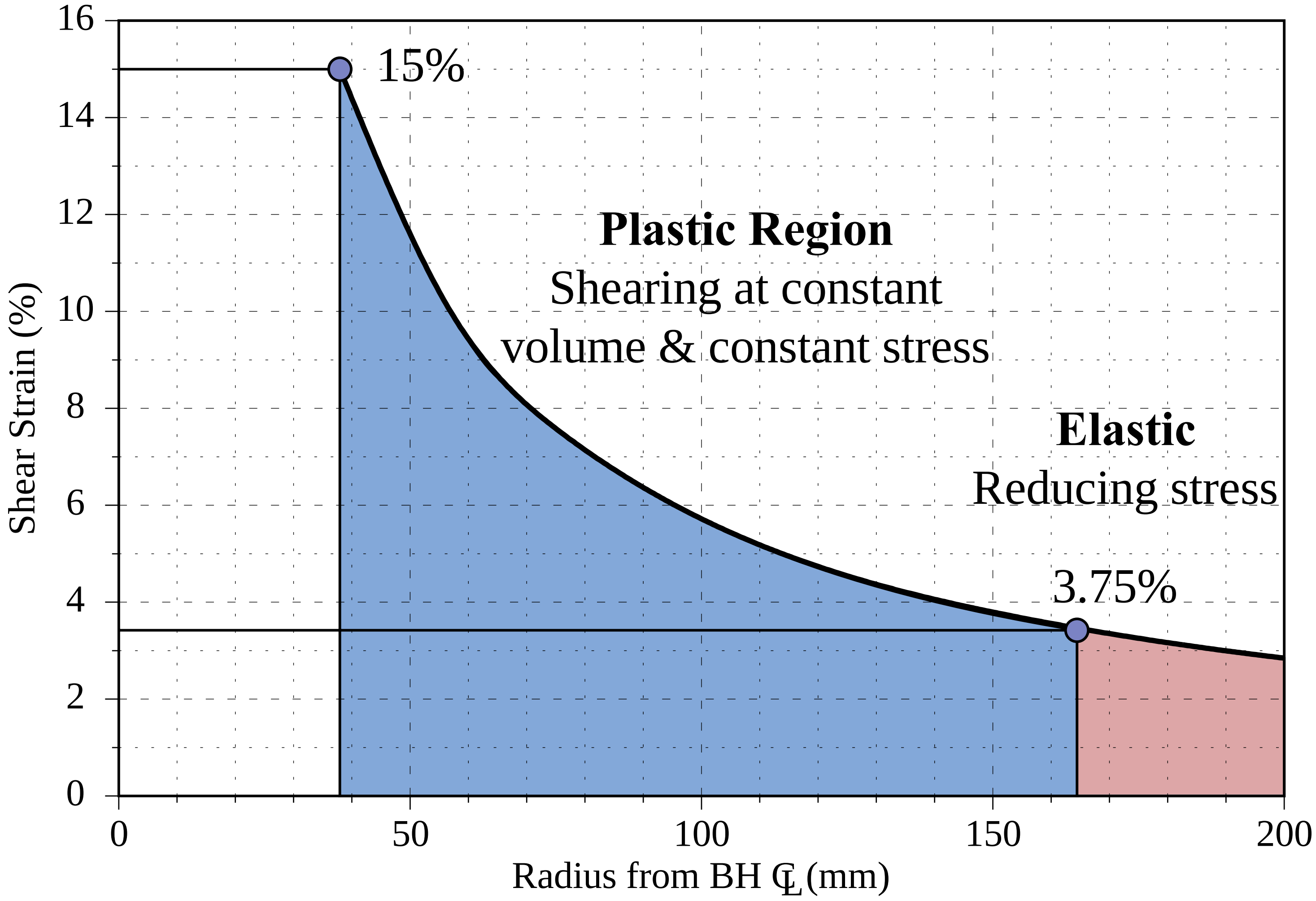
- Introduction
  - Strength of materials can be found using one of (or a combination of) several closed form solutions of a cylindrical cavity
  - Models exist that examine the material as either a linear elastic-perfectly plastic material or a non-linear elastic - perfectly plastic material
  - Most derived for clay as the assumption of zero volume change vastly facilitates the mathematics

Strength of Clays

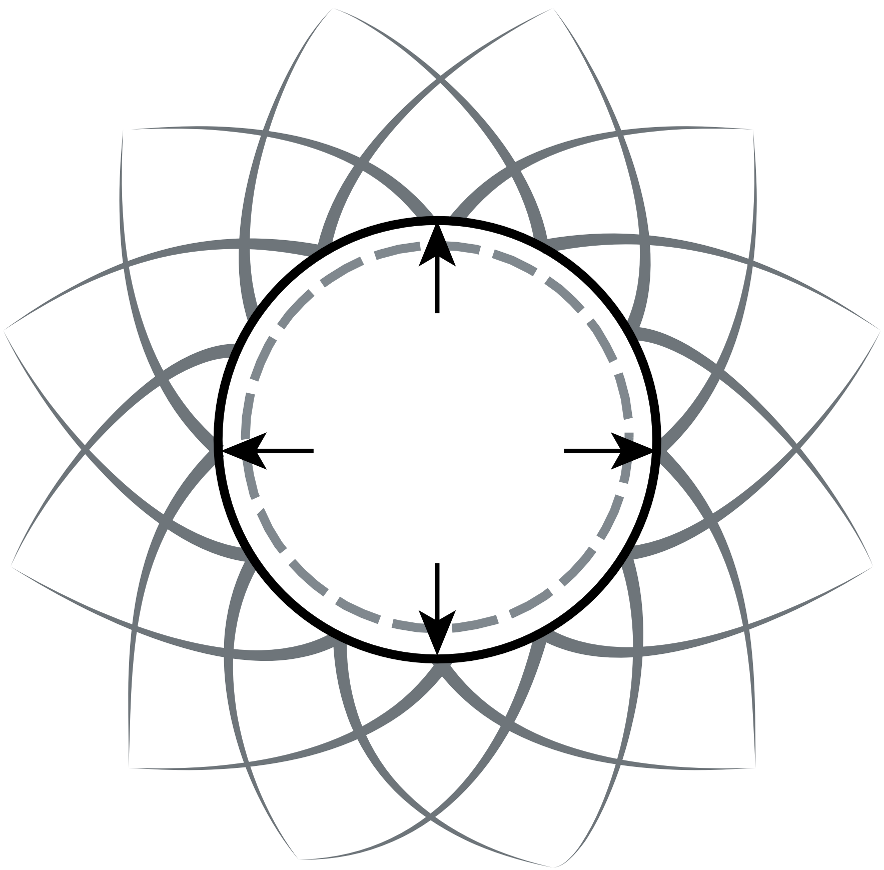
- Gibson and Anderson (1961) developed a closed form solution for an linear elastic – perfectly plastic undrained material
- The clay is characterized by the shear modulus, *G* and undrained shear strength, s_u



Strength of Clays



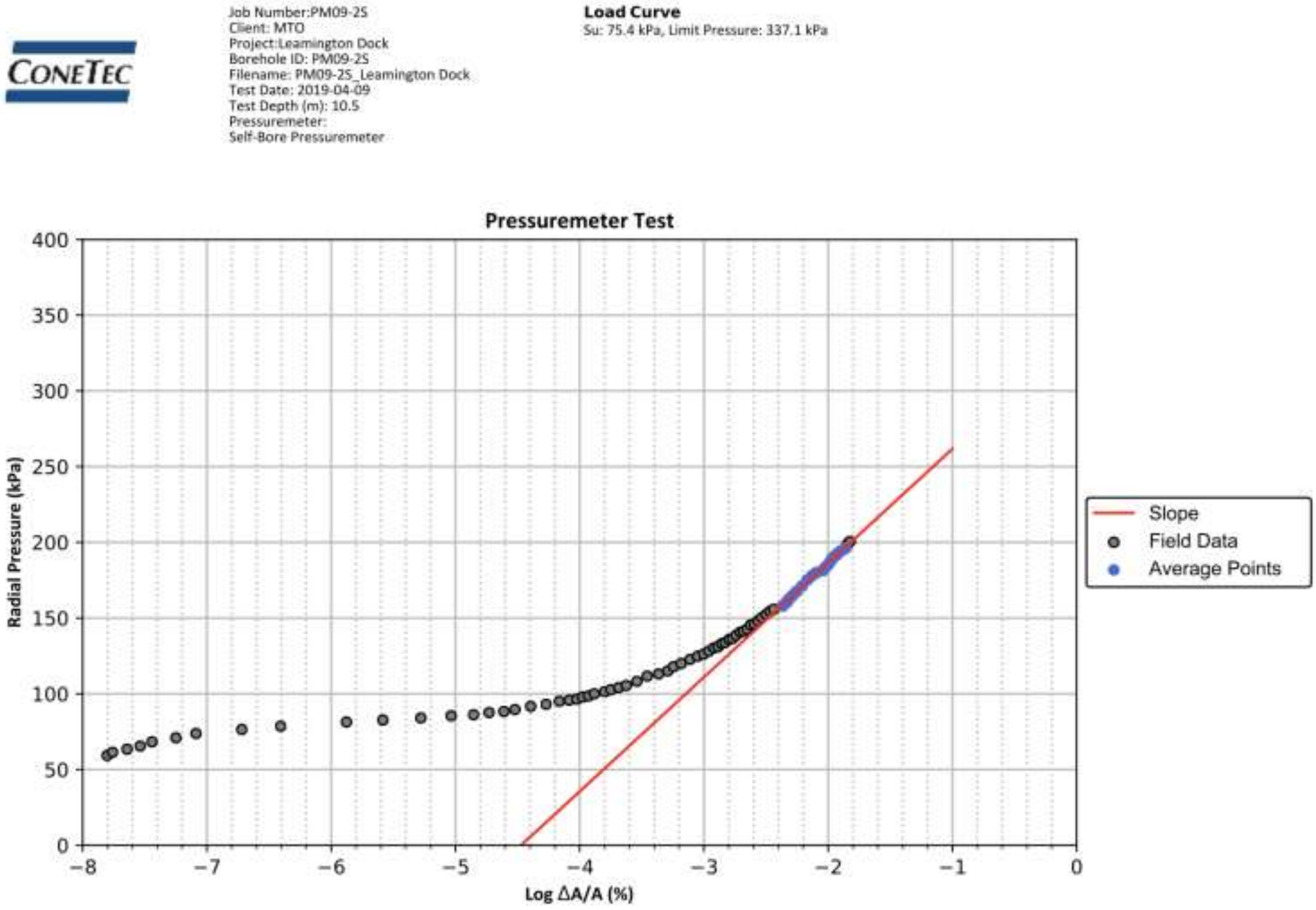
Pressuremeter Test



Strength of Clays

- The applied pressure, ψ is linearly related to the logarithm of the current volumetric strain $\Delta V/V$

- If natural logarithms are used, the gradient of the line will be equal to the undrained shear strength

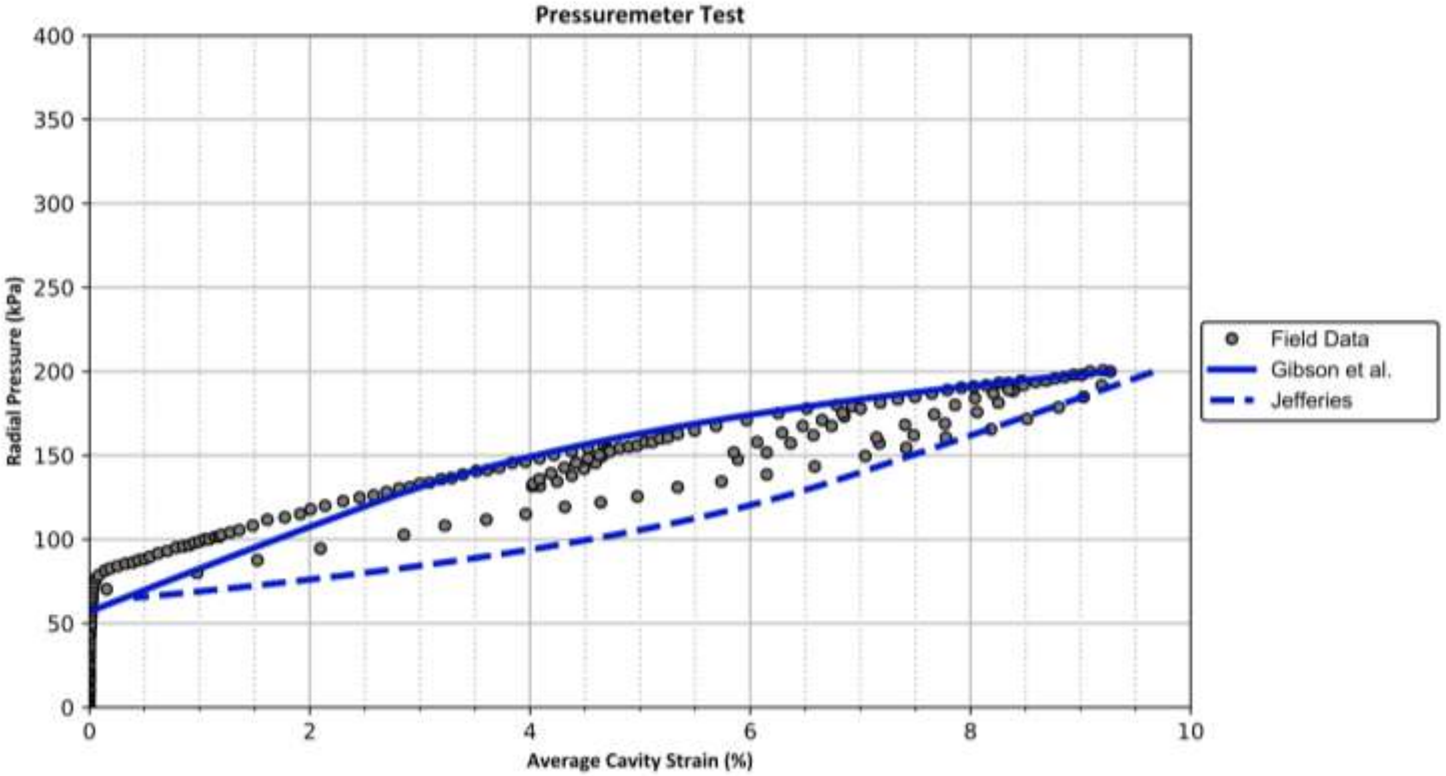


Strength of Clays

Job Number:PM09-2S
Client: MTO
Project:Leamington Dock
Borehole ID: PM09-2S
Filename: PM09-2S_Leamington Dock
Test Date: 2019-04-09
Test Depth (m): 10.5
Pressuremeter:
Self-Bore Pressuremeter

Load Curve Fit
Pre-bore shift (%): 0.0
Su Load (kPa): 66.2
G_{ur} (MPa): 1.3
σ'_{ho} (kPa): 57.0
K_o: 0.75
γ (kN/m^3): 16.0

Unload Curve Fit
Su Unload (kPa): 32.5
G_{unload} (MPa): 1.2
P_{max} (kPa): 200.6



$$p = \sigma_{ho} + s_u + s_u ln\left[\frac{G}{s_u}\left(1 - \left(\frac{a_0}{a}\right)^2\right)\right]$$

$$P = P_{max} - 2s_u - 2s_u ln\left[\frac{G}{2s_u}\left(\frac{a_{max}}{a} - \frac{a}{a_{max}}\right)\right]$$

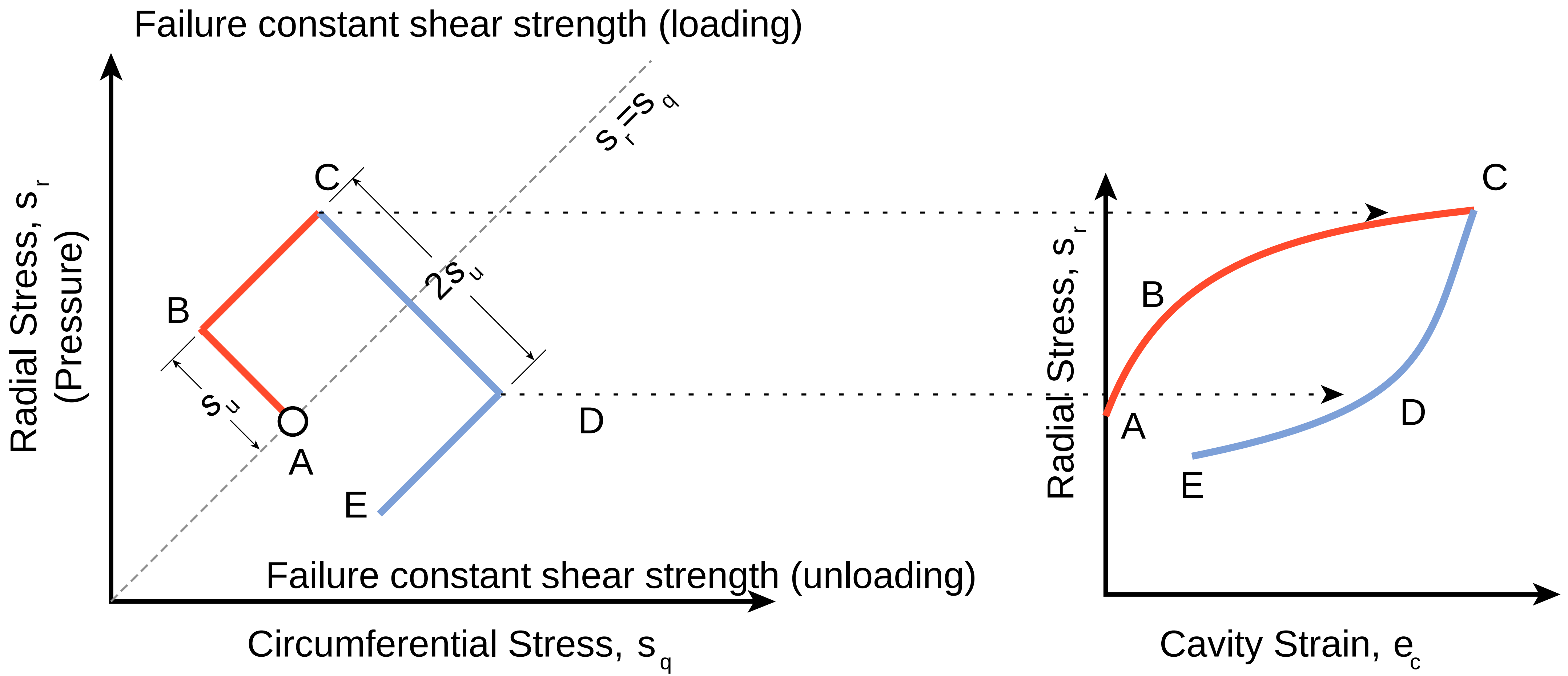
Strength of Clays

- Jefferies (1988) demonstrated that the undrained shear strength is best analyzed from the unloading curve

- Used the same analysis of Gibson and Anderson (1961), but instead of using the lateral earth pressure, the maximum applied cavity pressure was used

- This eliminated the potential damage to the borehole from installation

Strength of Clays

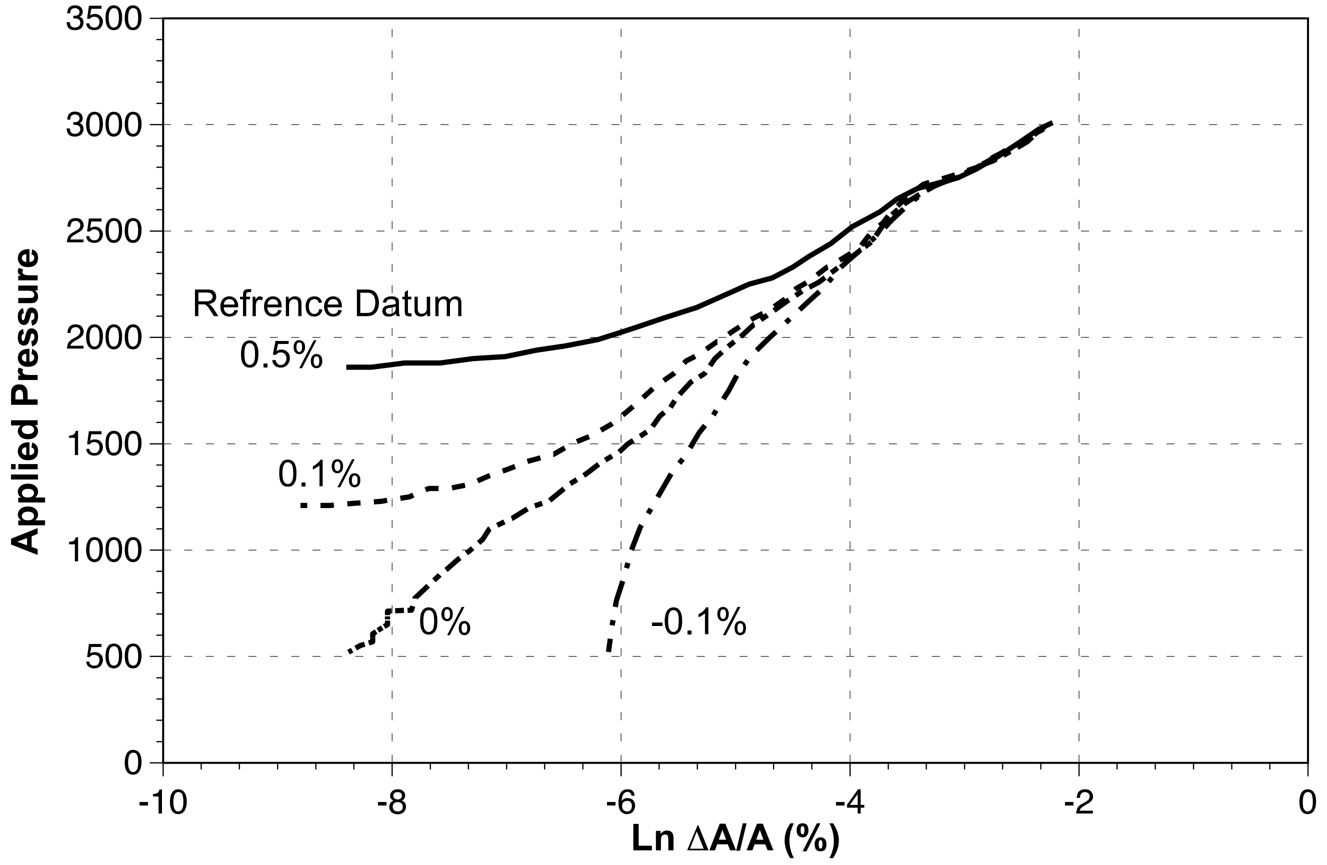


Stress Path

Ideal Pressuremeter Curve

Strength of Clays

- G&A and Jefferies took advantage that at the later stages of the test, that the shear strength near the borehole was fully mobilized and any damage due to drilling was eliminated



After Clarke, 1995

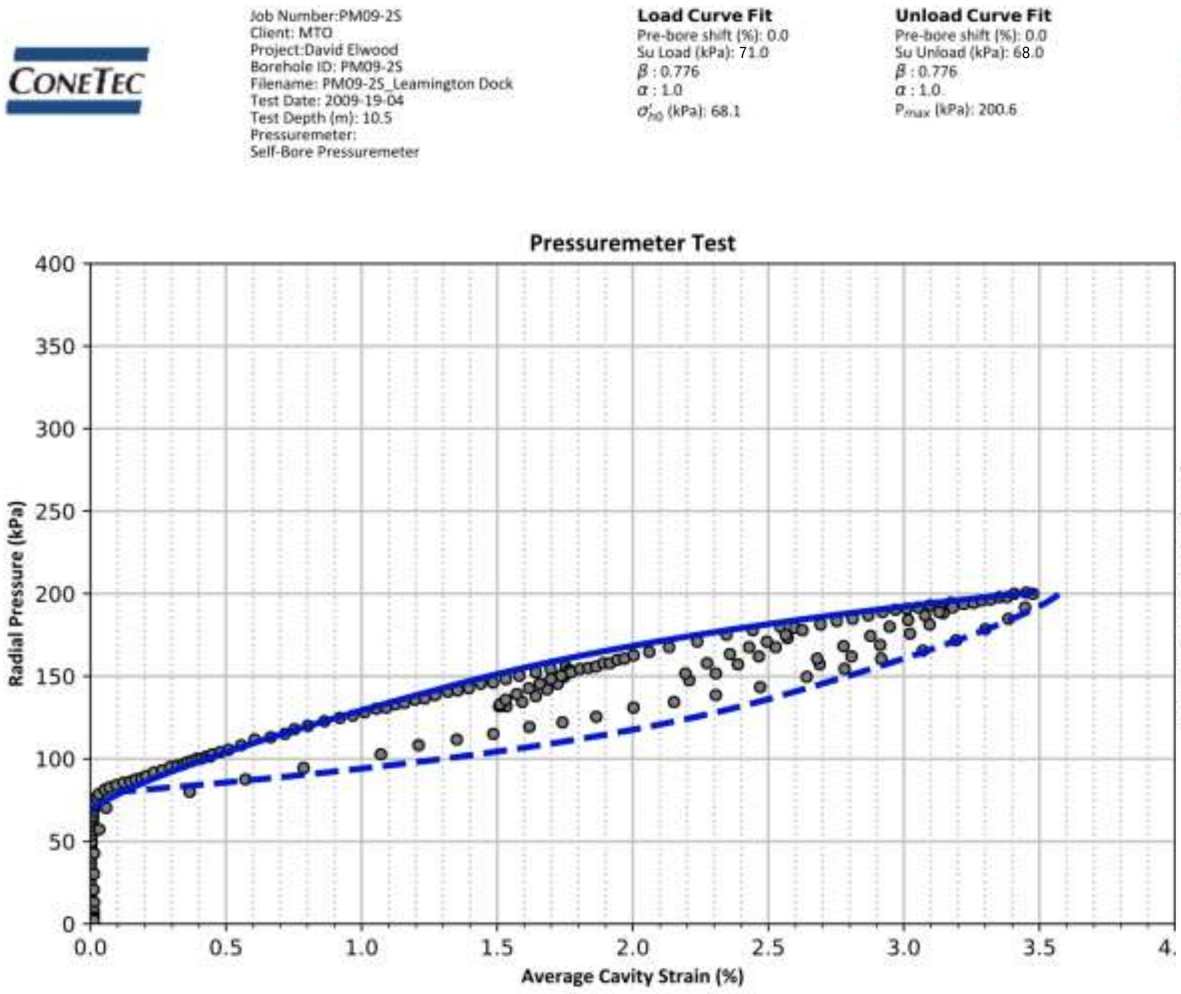
Strength of Clays

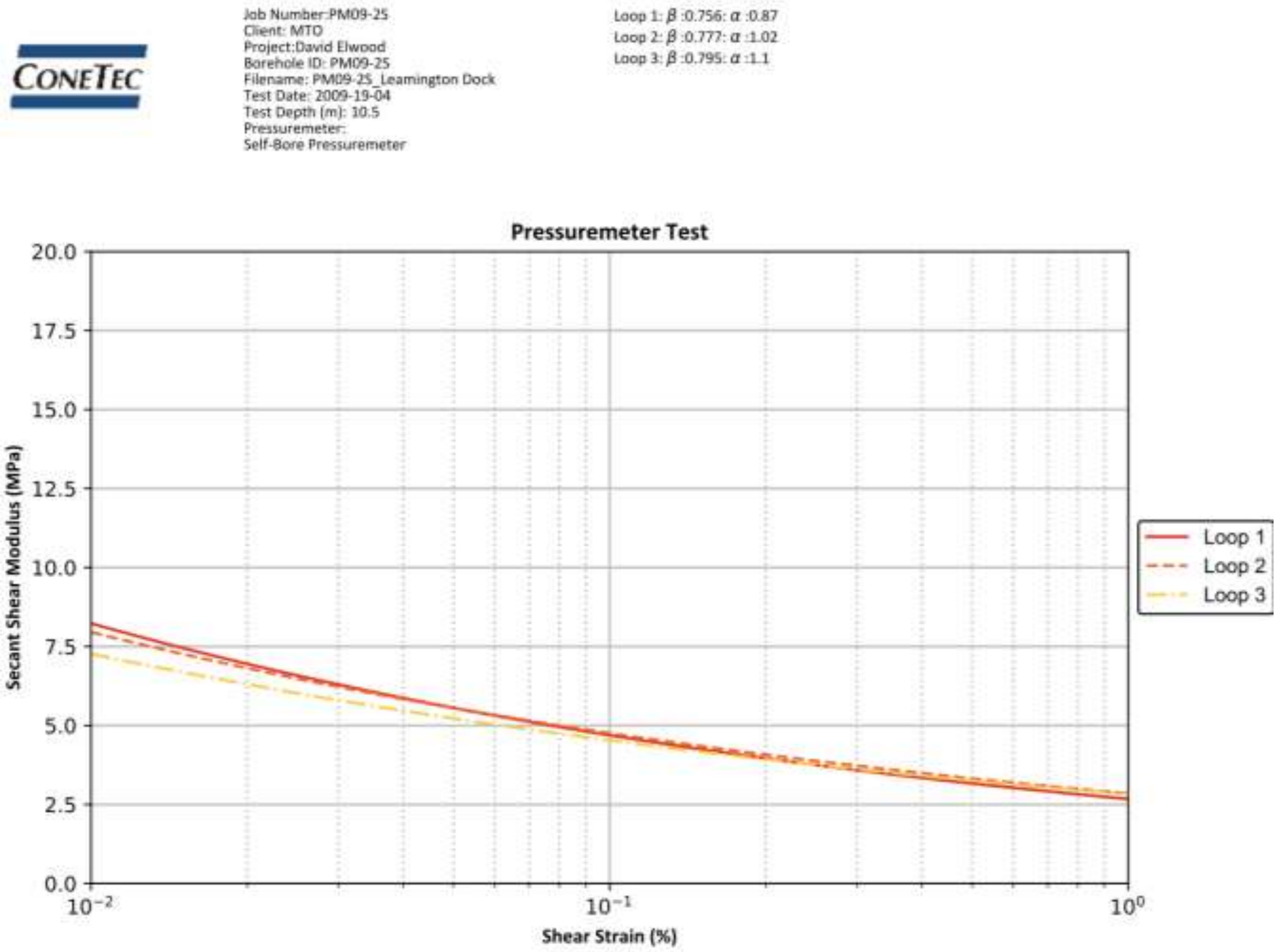
- Part of the elegance in the use of the G&A and Jefferies models is their simplicity
  - Both assume small-strain, linear elastic and large strain perfectly plastic models in their calculation
  - Very few assumptions needed and most values are measured directly from the SBPM test
  - Use of the latter portion of loading and unloading eliminate issues of borehole damage

Strength of Clays – Non-Linear Elastic

- Bolton and Whittle (1999) and Whittle (1999) recognized that a weakness of G&A and Jefferies laid in the assumption of linear elasticity
  - Use of the reload portion of u-r loops provided a basis for determining a power law fit for the secant and tangent moduli
  - The model assumes a non-linear elastic – perfectly plastic undrained model for clays
  - This non-linear model tends to fit the loading and unloading data better than the original models

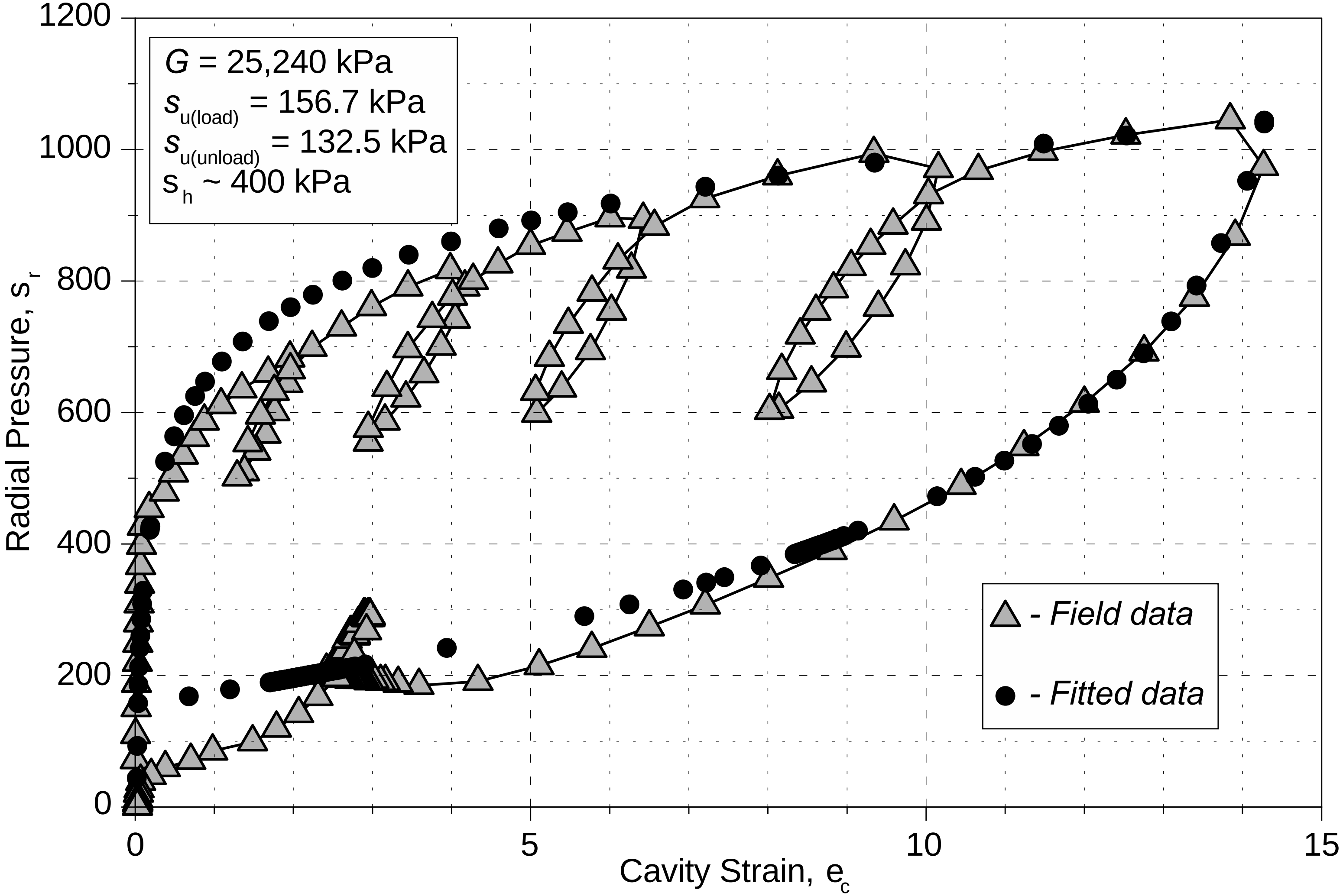
Strength of Clays – Non-Linear Elastic



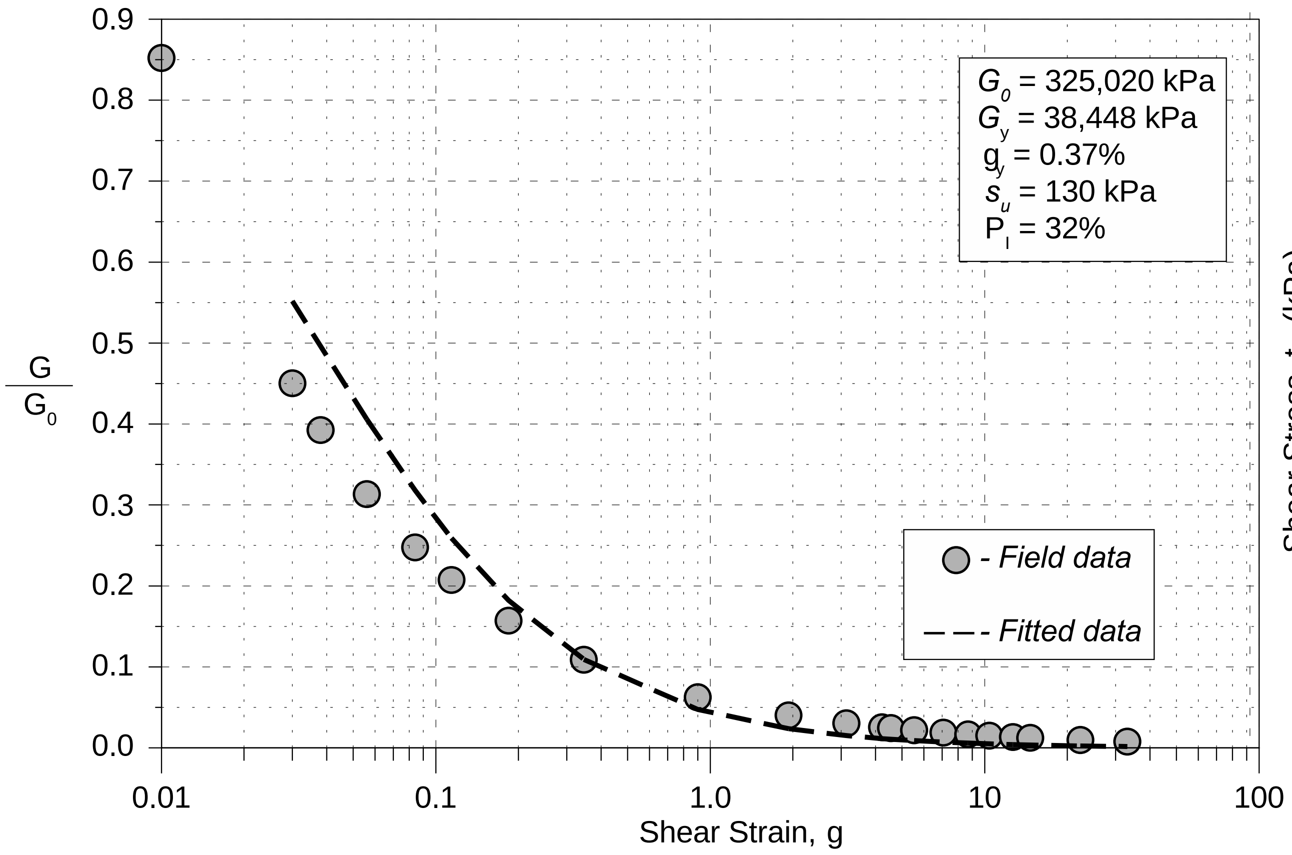


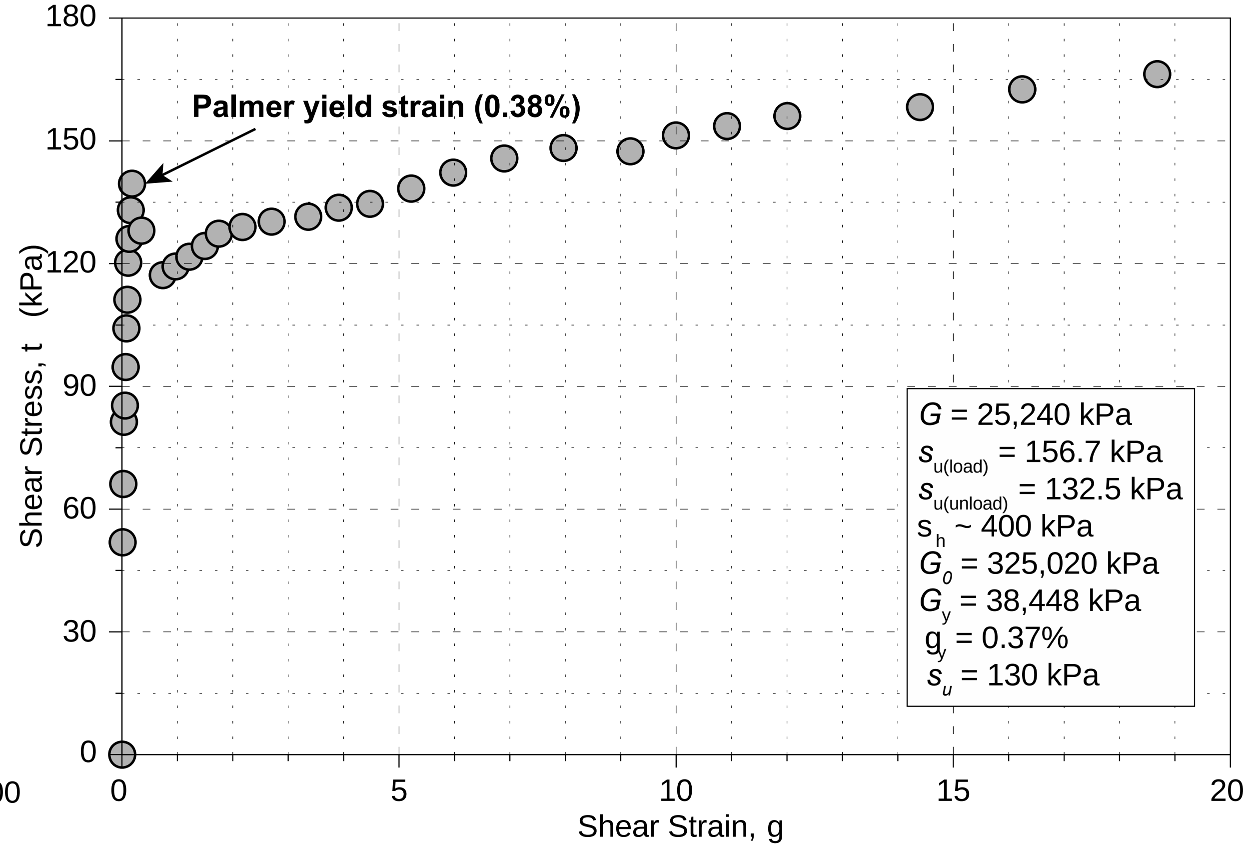
Strength of Clays – Non-Linear Elastic

- Data will also indicate the yield strain (γ_y) used for determining the expected ground response; Used with Palmer (1972) a complete τ-γ response can be provided



Strength of Clays – Non-Linear Elastic





Strength of Sands

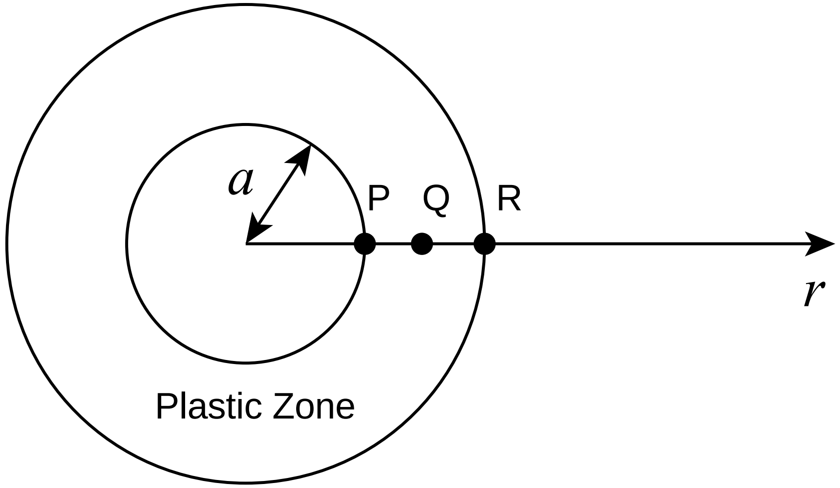
- The strength of sands is a more complex due to the volume change that occurs in the plastic region
  - Gibson and Anderson (1961) interpreted data of pressuremeter tests in sandy soils assuming that the sand behaved linear elastically until failure was reached
  - After failure, the sand continued to yield at constant ratio of effective stresses (as the stress level increased) and at constant volume.
  - By plotting log σ_r' vs log ($\Delta V/V$) and calculating the gradient of the resulting straight line which is equal to ½(1-N) or sin ϕ'/ (1 + sin ϕ')

Strength of Sands

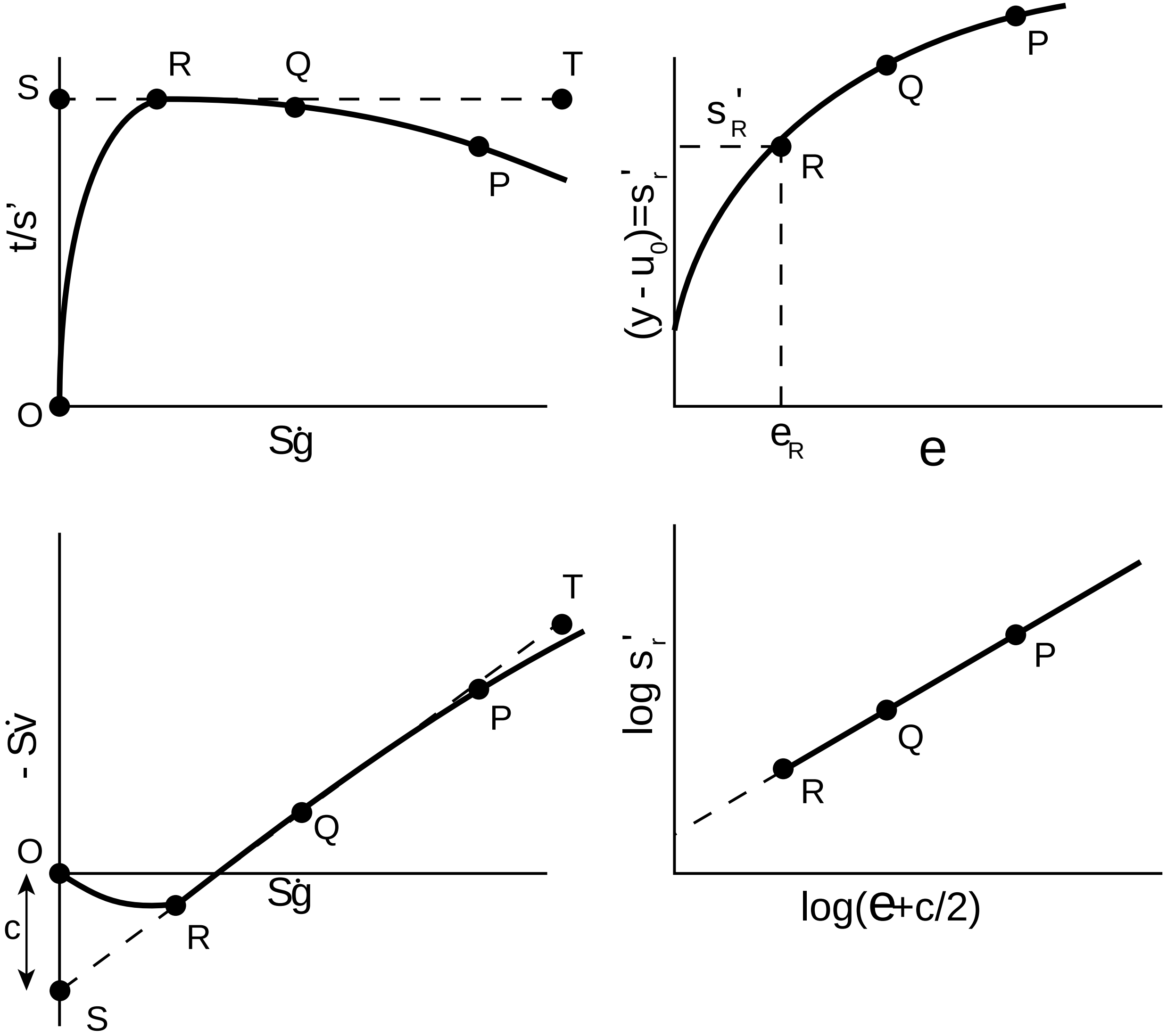
- Hughes et al. (1977) recognized that there was a major flaw in the G&A model and that was the assumption of shear at constant volume and that the exponent in the G&A solution was incorrect

  - The Hughes sand model utilized the findings of Stroud (1971) where the ratios of volumetric strain to shear strain do not vary significantly. These results agree well with Rowe's stress dilatancy theory
  - The sand fails at a constant ratio of effective stresses and at a constant rate of dilation.
  - Plotting the log effective radial stress (ψ-u_0) vs log (ε_c) and calculating the gradient of the resulting straight line, *s* if the dilation angle, υ is known then ϕ' may be calculated

Strength of Sands



$$s = \frac{\sin\phi' \left(1 + \sin\nu\right)}{1 + \sin\phi'}$$

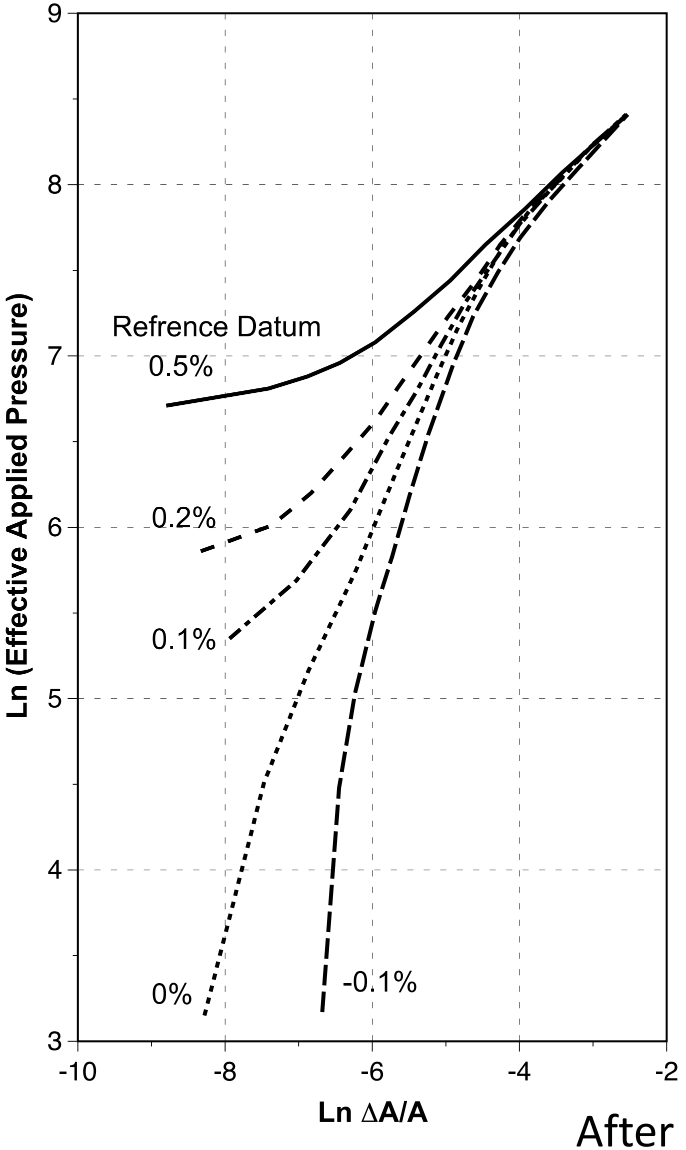


Strength of Sands

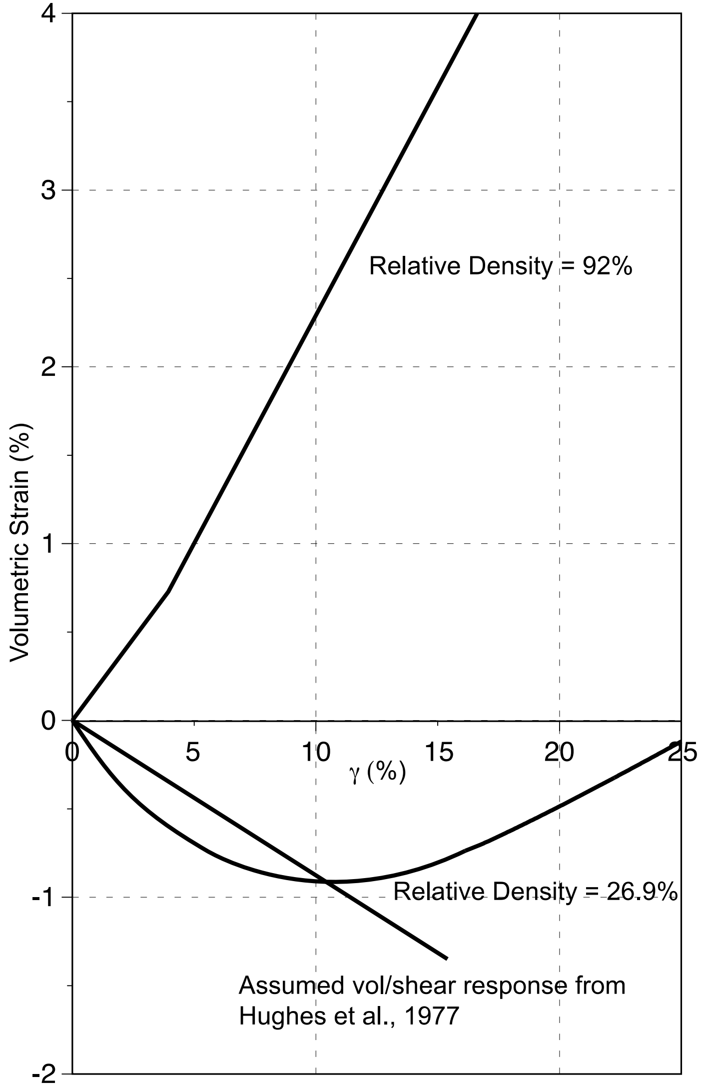
- Hughes et al. (1977) was limited in that it was particularly sensitive to the disturbance of the test pocket

- Hughes also was limited by the assumption that volumetric & shear strains are very near to linear and that the elastic strains are negligible. This is reasonable for dense sands, but not for loose sands

- Robertson and Hughes (1986(made an attempt to correct the Hughes model for loose sands



After Clarke, 1995



After Robertson and Hughes, 1986

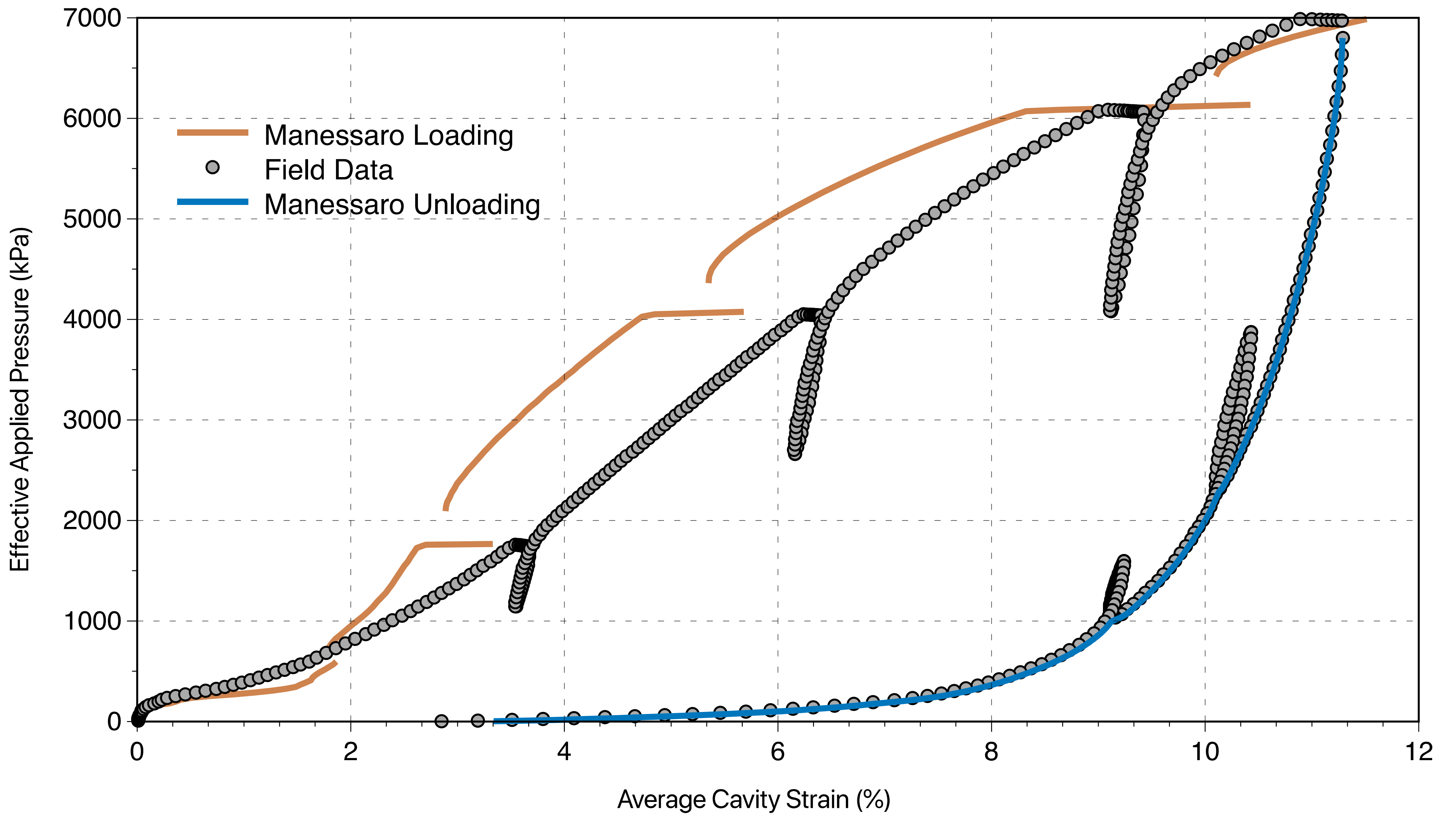
Strength of Sands

- In light of the sensitivity to the friction angle to disturbance, Houlsby et al. (1986) evaluated the friction angle on the unloading curve
  - It was clear that the method by Houlsby was not sufficient for determining a peak friction angle, but it proved useful for determining ϕ_{cv}' , though Hughes' model was not particularly sensitive to ϕ_{cv}'

Strength of Sands

- Manessaro (1989) developed a numerical solution assuming a backward finite difference, piecewise solution to fit curves
  - Though Manessaro admitted that considerable curve fitting (7^{th} to 9^{th} order polynomial fits) was required to determine the friction angle, the method was independent of test pocket disturbance
  - Whittle and Liu (2013) suggest that use of Manassaro for the unloading curve results in reasonable solutions for determining ϕ' without considerable data correction

Strength of Sands

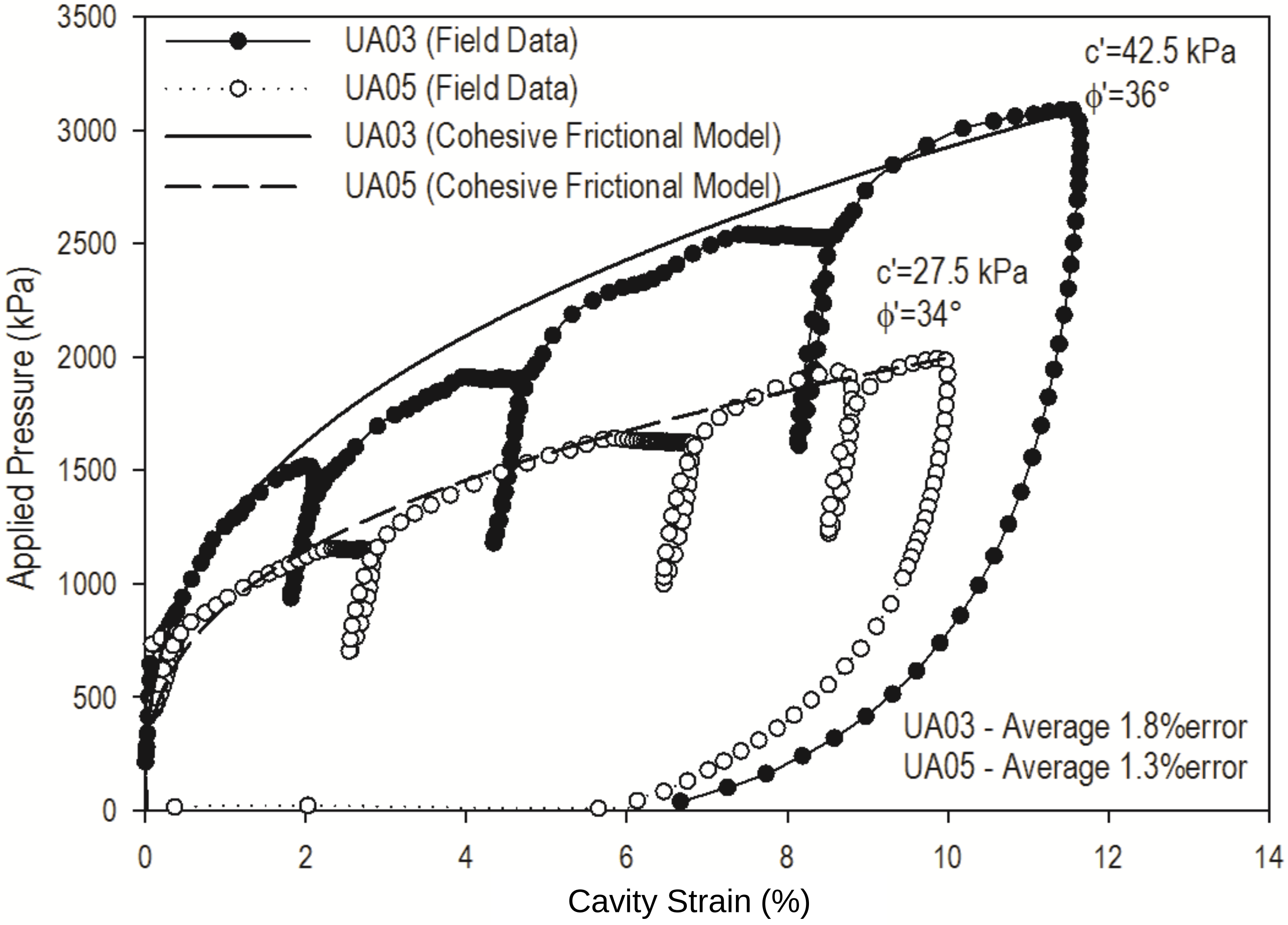


Cohesive – Frictional Soils

- Carter et al. (1986) developed a closed-form solution for cohesive – frictional soils considering small strains
- The model assumes linear elastic – perfectly plastic conditions with a constant rate of dilation (similar to Hughes' Sand Model)
  - The major difference between Carter's model is the incorporation of elastic strains within the plastic region and the incorporation of cohesion into the input
- Though introducing another variable, it is relatively simple to use and the results are quite applicable to most tills common to Canada which may be in an unsaturated state prior to borehole loading

Cohesive – Frictional Soils

- Evaluation of Carter's solution shows that if cohesion is set to zero, the answer is very similar to Hughes' model and if the friction angle is zero, then the solution is identical to G&A's model
- The constant rate of dilation can also accommodate either contractile or dilative soils



Cohesive – Frictional Soils

- Yu and Houlsby (1991) developed a closed-form solution for a non-associated, cohesive – frictional soil that is capable of infinite expansion
- The model assumes linear elastic – perfectly plastic conditions with a constant rate of dilation (similar to Carter's Model), but in this case, there is no restriction to the magnitude of displacements
  - As the applied stresses approach the limit pressure, then the strains become infinite
- Like Carter et al, this model reduces to the simpler models when aspects are eliminated. Ultimately, the use of this is overshadowed by the simpler to implement and less assumption prone models presented earlier

Final Thoughts…

- The strength values obtained from the PMT are extremely useful when the right models are applied
- Use of non-linear models in clays and sands can help understand the strain and stress dependent moduli and better inform numerical models
- It is important to understand the changes in stress and influences on pore-water pressures throughout the entire test including test pocket formation when applying a given curve fitting model