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Small Strain Stiffness assessments from in situ tests – revisited

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ABSTRACT: The increasing trend to model the behaviour of the ground requires more detailed soil properties that include small strain data. These data can be obtained from both field and laboratory tests to measure shear wave velocities from which the shear modulus at small strains can be calculated. However, such measurements are only specified for large projects and so smaller projects have to rely on correlations from other in situ tests. In 2004 Powell & Butcher presented and discussed some correlations between the small strain stiffness of the ground in different orientations measured by geophysical tests and parameters measured by the Cone Penetrometer (CPT) and the Marchetti Dilatometer (DMT). Since that time the database of information has grown and this paper reviews and updates the findings of the earlier paper. Anisotropy of stiffness is shown to be a major factor in the correlations especially in 'aged' clays.

1 INTRODUCTION

Whilst there has always been a need to not only design to prevent failure but to control movements, the latter item has often only been given cursory attention and often satisfied by a very generous factor of safety on the ultimate or failure state. Increasingly through the Euro and other codes we have to also show that we can prove that we satisfy serviceability (movement) limit state. As a result of this and also the need to supply strength and stiffness parameters for geotechnical modelling the need for realistic stiffness parameters has gained increased interest. It has been shown by Simpson et al (1996) that in order to successfully predict lateral and vertical movements around excavations and tunnels in overconsolidated soils the anisotropy of stiffness and stress are the most important properties to quantify.

The shear modulus is largest at very low strains and decreases with increasing shear strain. It has been found that the initial maximum shear modulus is constant for strains less than 10^{-3}% although this may vary with plasticity index I_P (Vucetic & Dobry (1991). This initial, small strain modulus is often denoted G_0.

Stiffness varies with stress level and is generally non-linear for most soils. It is now well accepted that the change or decay of stiffness follows an 'S' shaped curve with the stiffness at very small strain being the starting plateau maximum which decays to a lower plateau at large strains. Great interest has developed in the measurement of stiffness at very small strains and its extrapolation to the larger strains typical of other in situ testing devices.

Butcher & Powell (2001) published data from laboratory and field testing on a series of established and well-characterised testbed sites which highlighted the stiffness anisotropy present in many soils especially heavily overconsolidated 'aged' clays. Their work and that from many other authors has tried to explain what controls this anisotropy in terms of the basic soil properties encountered and the stresses present in the ground. Powell & Butcher (2004) published the stiffness data from their testbed sites and related it to measured and derived parameters from various in situ tests trying to establish simple correlations between the data. Others have tried similar approaches but, as will be discussed later, introduced various additional parameters in equations to improve 'fit'. Powell & Butcher (2004) suggested that a major influence in the derivation of correlations was 'anisotropy', from both stresses and inherent or fabric influences.

This paper tries to build on the 2004 paper by including additional data not only from the original sites but also from a variety of additional sites and databases that are now available. These comprise 10 European testbed sites with high quality information and used for a Brite Euram study on semi empirical foundations design procedures from in situ tests (Shields et al 1999, Powell et al 2001), 3 heavily overconsolidated clay sites investigated by Hosseini Kamel (2012) and Brosse (2012), the heavily overconsolidated Boom clay, Piriyakul (2006) and 11 working sites covering a range fine grained deposits including some additional London clay sites.

This paper uses selected data from these sites where geophysical seismic wave measurements have taken place as well as CPT, CPTU and DMT.

The purpose of the paper is to suggest that increased confidence can be given to previously suggested relationships related to anisotropy of stiffness and realistic correlations related to in situ test devices.

2 THE SITES

To include a table of the sites and their basic geotechnical properties would take up too much space and so the interested reader is referred to references mentioned above for full details. To summarise some information in a brief form for the new sites with 'aged' clays in Table 1.

Table 1. Aged Clays.

Clay	Approx. Age	W_P	W_L	I_P	e
	10^6 years	%	%	%	
Oxford Clay	161-156	34	66	32	0.6
Kimmeridge Clay	156-151	23	49	26	0.46
Gault Clay	122-995	28	74	46	0.67
London Clay*	56-49	29	66	37	0.82
Boom Clay**	35	26	72	46	0.73

* See other LC info in Powell& Butcher 2005.
** from Piriyakul (2006)

The rest of the clays in the study tend to be considerably younger, either of late glacial origin or estuarine or marine clays of Holocene age.

Void ratios vary from around 1.7 for the young estuarine and marine clays to as low as 0.45 for some of the glacial clays. For the 'aged' clays void ratios vary from 0.8 to 0.45.

3 GEOPHYSICAL TESTING

The interest in geophysical testing has grown considerably in the last 20 years as a way of establishing the small strain stiffness of a soil (the highest or maximum stiffness) from which point the now stiffness against strain decay curve can be deduced, often referred to as the 'S' shaped decay curve. The value derived is often referred to as G_{max} or G_0 which can be misleading. It is strongly suggested that, when a value for small strain stiffness from geophysical testing is reported, then it should be quite obvious as to how and in what orientation it was derived. The various methods of geophysical testing tend to test the ground in different orientations and hence can be used to look at stiffness anisotropy.

The original field measurements of shear wave velocities were made using BRE equipment and

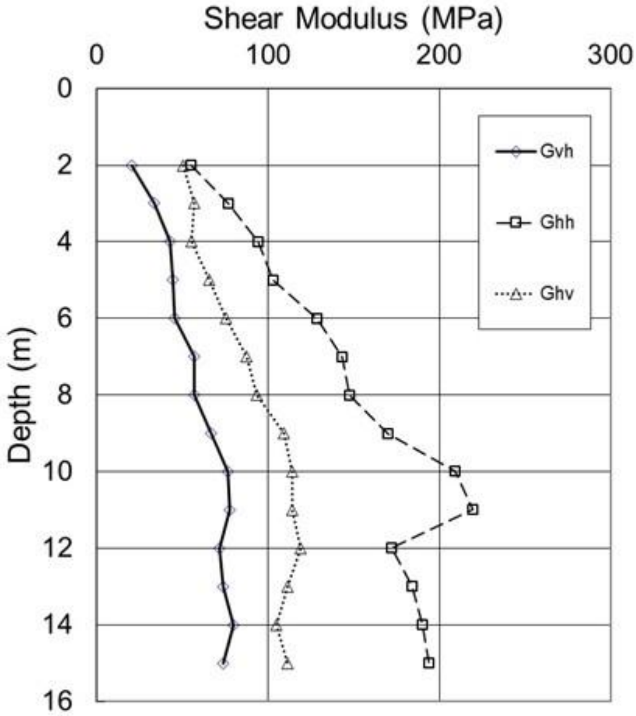

Figure 1. Typical G_{vh}, G_{hv}, and G_{hh} profiles for London Clay

techniques (Butcher & Powell, 1995a). Figure 1 shows a typical result from a London clay site.

For convenience in the following, subscripts will be added to G which will be related to the direction of polarization and propagation of the shear waves. For example, G_{vh} will denote the stiffness derived from vertically propagating, horizontally polarized (down- hole/seismic cone) shear waves.

It is generally to be expected that for a continuum medium $G_{vh} = G_{hv}$ and this has been shown by Stokoe et al (1991) and Bellotti et al (1996) for sands and Lo Presti et al (1999) and Pennington et al (1997) for reconstituted clays. However, Butcher & Powell (1995b) and Pennington et al (1997) showed that this may not be true for very stiff overconsolidated or layered clays and in fact differences can exist between G_{vh} and G_{hv} even in soft soils, but this may relate more to differences in testing methods in the two orientations, and others have found similar behaviour. In terms of stress anisotropy then it may be expected that G_{hh} is either close to or less than G_{vh} and G_{hv} in normally consolidated soils and greater in heavily overconsolidated soils. The situation is further complicated in the 'aged' clays or those showing significant fabric as there will be both stress and inherent anisotropy.

It must be remembered that there is a significant scale difference between field and laboratory shear waves used in the measurements. It is likely that fabric will affect field measurements more than laboratory measurements hence contributing to the differences between G_{vh} and G_{hv}.

Hardin (1978) suggested that for clays, the small strain shear modulus, G_0, depends on the applied stresses, void ratio and overconsolidation ratio

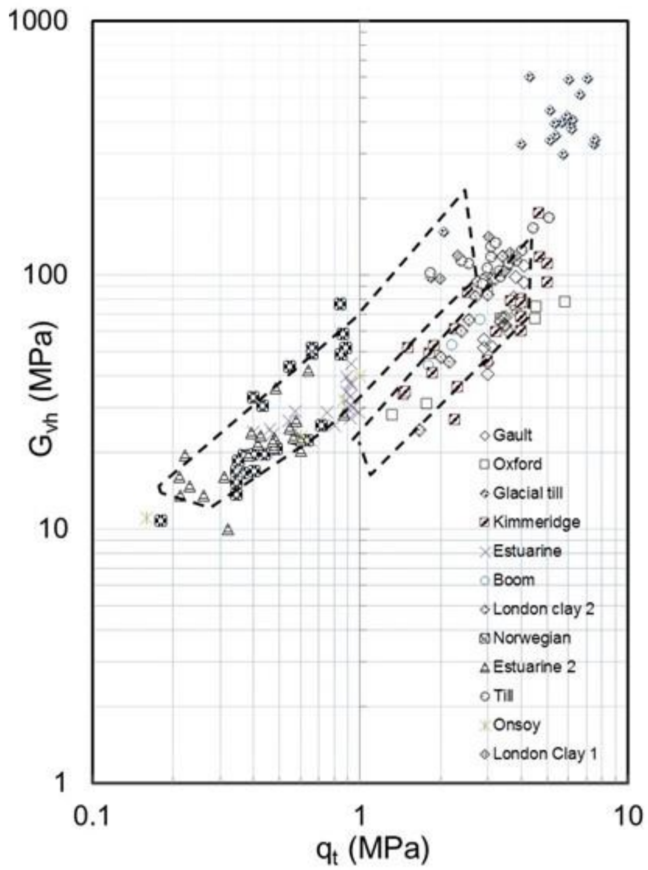

Figure 2a. Shear Modulus G_{vh} against q_t

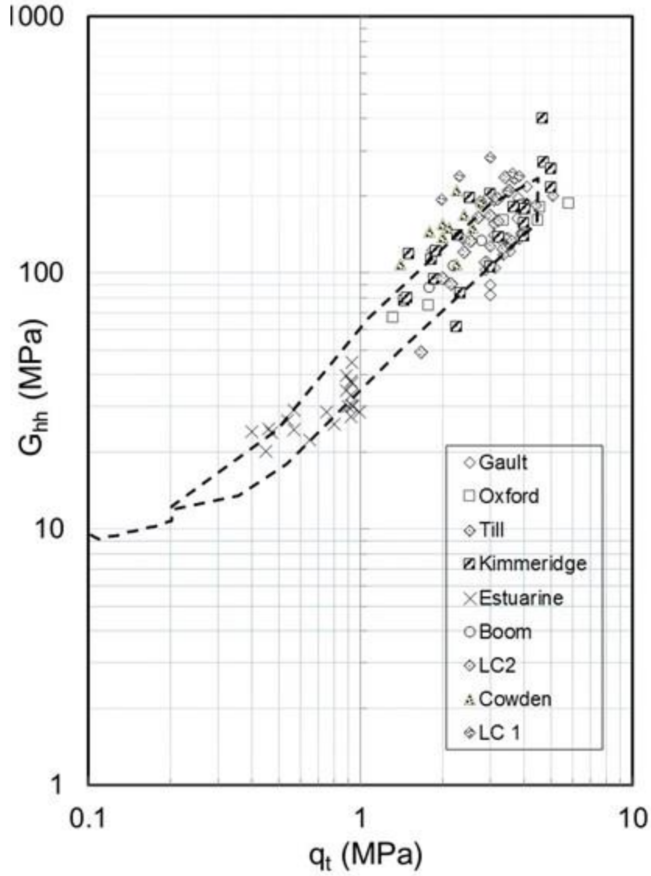

Figure 2b. Shear Modulus G_{hh} against q_t

(OCR). It has however been shown that the effect of OCR is, to a large extent, taken into account by the effect of void ratio and could be neglected. Butcher and Powell (1995b) had some success in relating shear modulus to the in situ stress anisotropy and its influence on the propagation and polarisation directions of the shear waves.

4 SMALL STRAIN STIFFNESS FROM CPT

Many authors have tried to correlate parameters from CPT and CPTu tests with small strain stiffness.

Various authors have tried to correlate q_c or q_t with G_0 with varying success. One of the first to do this, Mayne and Rix (1993), used an extensive database to try to correlate G_0 with measured cone resistance, q_c, with some success but also with some considerable scatter. They later suggested (Mayne and Rix, 1995), based on the original premise of Hardin mentioned above, that it might be better to relate the small strain shear modulus with a combination of void ratio (e) and cone penetration resistance q_c and suggested that it was valid for a wide range of clays. Variations of their equations can be found in the literature.

Simonini and Cola (2000) suggested that the pore pressure ratio from the piezocone (B_q) could be used as an additional parameter in the correlation to replace void ratio. They showed that when considering relatively lightly overconsolidated mixed deposits in Venice, then a better correlation between q_t and G_0 was obtained. Long & Donohue (2010) have tried this route for Norwegian soils and came up with a variation on the Simonini and Cola (2000) equations. Many authors seem to have some degree of success finding fits to their own data sets, but when used by others these often seem to fail (see Long and Donohue 2010 for some examples).

In Figure 2a the G_{vh} shear moduli derived from either standard downhole or SCPT testing, on the collection of sites mentioned earlier, are plotted against the corrected cone resistance q_t (on a log-log plot). For clarity only the boundary of the original data from Powell & Butcher (2004) is shown, simply by areas bounded by dotted lines. Also included are results from Norwegian clays presented by Long & Donohue (2010), but only for those sites with SCPT data (in their paper other sites used MASW results and these certainly increased the scatter of their data). It can be seen that all the new data fall either within or as extensions to the earlier data, with a reassuring result. The 'aged' clays (Table 1 + additional London and Gault clay sites) fall in a lower group and the younger clays tending to the upper group.

Because of a lack of availability of equipment, or a lack of understanding of its usefulness, field G_{hh} testing is seldom done. An alternative approach has been used to try and extend the earlier study, namely scaling the G_{vh} data by the use of laboratory assess

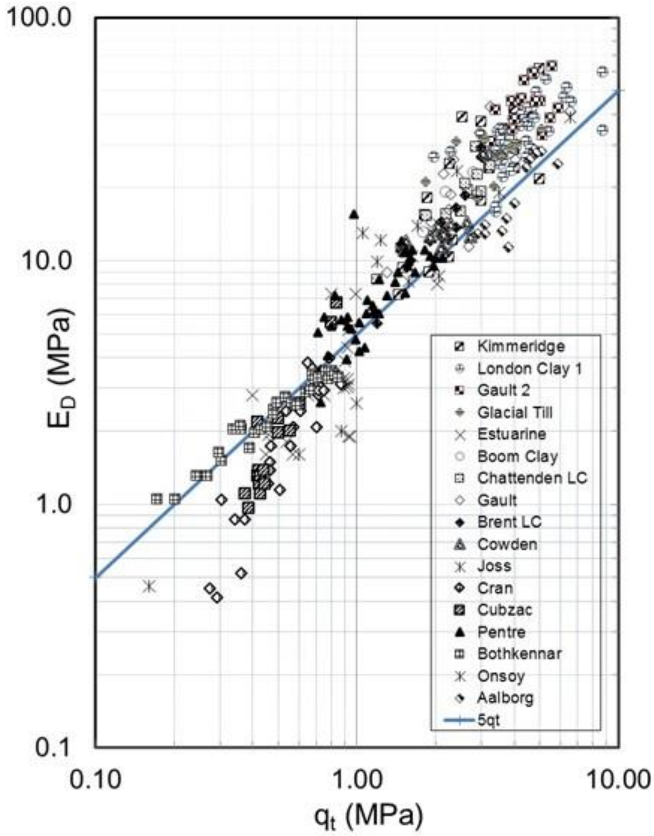

Figure 3. Dilatometer Modulus E_D against q_t

ments of the G_{hh}/G_{vh} ratio on high quality samples. In the earlier field work Powell & Butcher showed that typically for London clay G_{hh}/G_{vh} was between 2 and 2.5 and in the Gault clay around 2 (for the normally consolidated slightly cemented Bothkennar clay the ratio was 0.8 - 0.85; for Pentre 0.9 – 1.05). From Laboratory work Hosseini Kamel (2012) reports ratios based on G_{hh}/G_{hv} (note G_{hv} used here) of 1.8 for London clay (Gasparre 2005), 1.9 for Gault Clay, 1.7 for Kimmeridge Clay and 2.3 for the Oxford clay; he puts the higher values in the Oxford clay down to the prominent clay particle orientation and bedding. Piriyakul (2006) suggests a ratio of just over 2 for the Boom clay. In advanced commercial laboratory testing at Geolabs similar ratios to those above have been found for London and Gault clays.

Whilst in the field it is often found that G_{vh} and G_{hv} are not equal these differences are often not so marked in laboratory testing, given that G_{hv} is generally larger than G_{vh} this may explain the slightly lower values of G_{hh}/G_{hv} from Hosseini Kamel (2012) work compared to field test data. Piriyakul (2006) in fact does show differences and examines the potential role of the different stresses acting in the different planes. He investigates the effects of stress anisotropy as well as inherent anisotropy and this is an area mentioned by many including Butcher and Powell (1995b) when they tried the same approach. In Figure 2b the G_{vh} results for the stiff aged clays have been plotted as G_{hh} using the scaling factors above. A constant ratio has been used for each clay type and this may explain the larger scatter seen using this approach as compared to actual field measurements (see Butcher & Powell 2001) to see that ratios change with depth most probably related to changing inherent anisotropy). It is clearly seen in Figure 2b that the strong relationship suggested by Powell and Butcher (2004) between q_t and G_{hh} is further confirmed by these additional 'aged' soils as well as the stiff clay tills (Cowden field G_{hh} data is now also included).

In their earlier paper G_{hv} was also considered and showed an improvement over G_{vh} but only to a limited extent.

5 CORRELATIONS WITH DMT

The Marchetti Dilatometer is now widely used in many countries and its derived parameters are correlated with many soil properties (Marchetti 2015). Figure 3 shows data from many of the sites in the present and past studies plotted as E_D, the dilatometer modulus against q_t.

A very strong trend linking the two parameters can immediately be seen. Most recently Robertson (2009) showed this type of relationship with the use of normalized parameters. He suggested that the equation $E_D = 5q_n$ would fit most of the data where q_n is the net cone resistance (q_t minus the vertical total stress) but that 'site specific' values between 2 and 10 would be even better. It can be seen in Figure 3 that there could well be 'site specific' values available here especially at low values of q_t. Using net cone resistance in Figure 3 would generally simply tend to move the points with lower q_t values slightly to the left in the plot when tests are shallow. In the simple form shown here there is still a striking link between the two parameters which would not fully fit with Robertson's mean equation though.

In Figures 4a & b we see plots of E_D and small strain modulus G_{vh} and $G_{hh;}$ again with the previous data ranges from Powell and Butcher (2004) shown as boundaries marked with dotted lines. Once again we can see the data falling into two distinct groups in Figure 4a (young upper, 'aged' lower)' but in Figure 4b we see a strong potential single group correlation between E_D and G_{hh} rather than G_{vh}. It implies, as one might expect, that E_D is very much related to a horizontal stiffness.

Marchetti (1980) uses E_D to derive a constrained modulus from the test. This is done using

$$M_{DMT} = R_M . E_D$$

where $R_M = 0.14+2.36 \log K_D$; for clays

Good success has been reported using this approach in predicting settlements of foundations

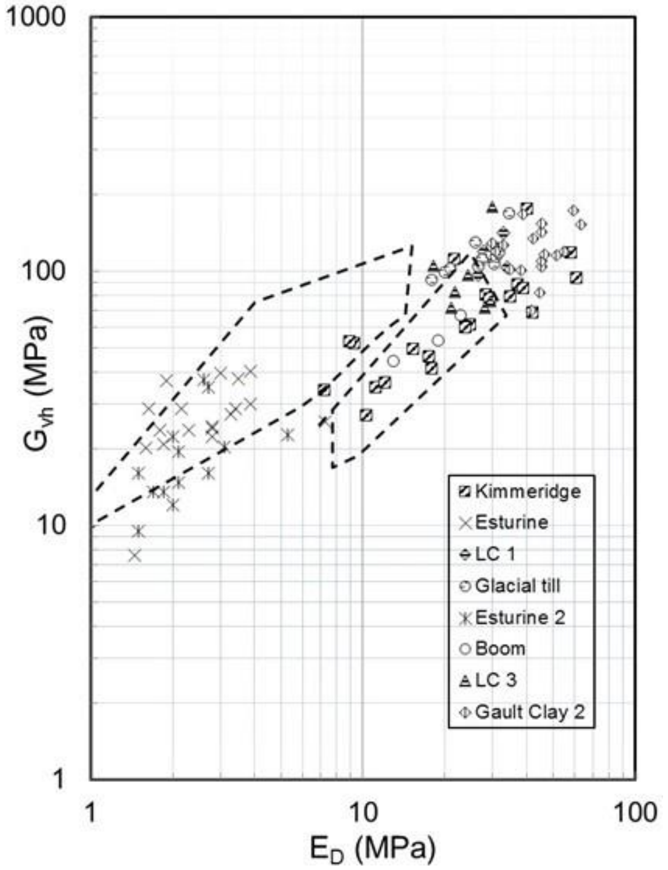

Figure 4a. Shear Modulus G_{vh} against E_D

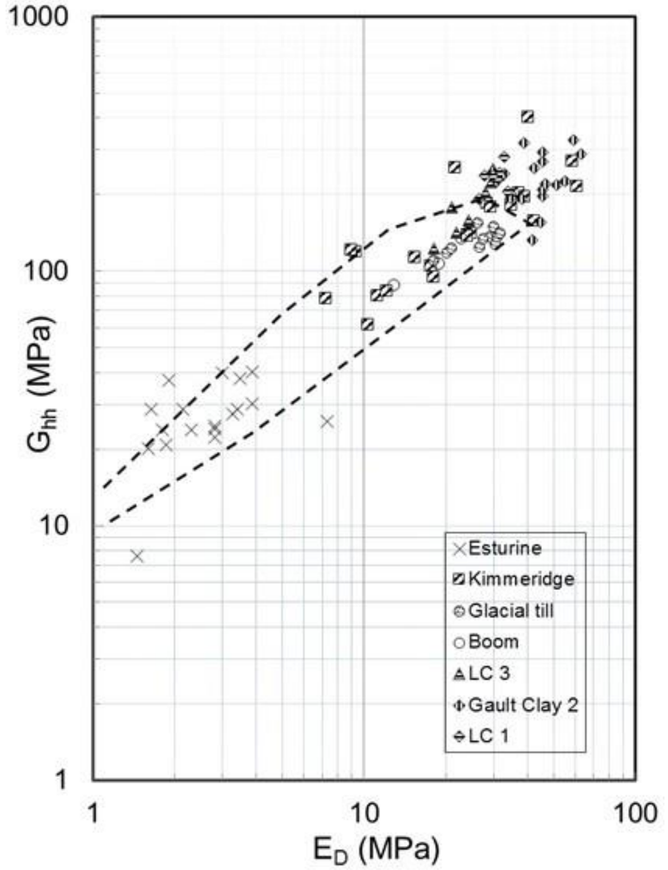

Figure 4b. Shear Modulus G_{hh} against E_D

however, the above seemingly strong correlation between E_D and G_{hh} opens up a question. The link between R_M and E_D. Marchetti (1980) shows K_D is linked to K_0 and OCR and an increasing K_D shows and increasing K_0 and OCR, however and increasing K_D also gives and increasing R_M in the above equation. This raises a question, should R_M really increase with increasing K_D? Surely if E_D is related to horizontal stiffness then, as K_0 increases the scaling horizontal to vertical should reduce. Using the above equation, R_M rises from 0.87 to typically 3 or so as K_D increases. Powell and Uglow (1988) suggested that R_M was around 0.5 for the 'aged' clays.

Monaco et al (2009) looked at links between G_0 and E_D and other dilatometer parameters. Consideration of the above matters may well influence their conclusions.

It is suggested that the topic of anisotropy of stiffness within the DMT framework needs further consideration and review.

6 DISCUSSION AND CONCLUSION

Data from a range of different soils have been presented in this paper that strongly supports the earlier work of Powell & Butcher (2004). This will give increased confidence to designers who rely on the correlations to get small strain behavior from in situ tests. It shows that the corrected cone resistance from the CPTu, q_t, and the Dilatometer Modulus, E_D, are strongly influenced by the horizontal stiffness and stresses in the ground. Both parameters correlate well with the in situ horizontal shear modulus G_{hh} and these correlations are very much stronger than with either G_{vh} or G_{hv}. To what extent the inherent anisotropy of very old 'aged' clays plays a role within this cannot be determined at this stage. It is only with the marked differences in directional stiffness in these aged clays that the picture becomes clearer.

Most of the correlations developed to date have tended to concentrate on data from younger clays and have often not been very successful when transferred to different soils. The presentation of the data in log-log is not ideal but does seem to indicate strong trends.

It is suggested that the correlation of constrained modulus (M_{DMT}) from E_D of the DMT test needs to be better understood.

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