

Small Strain Stiffness of Soils from Field Measurements

Technical Abstract

Predicting soil behaviour in an earthquake is difficult because we lack sufficient knowledge of in-situ soil dynamic properties. The two key parameters that characterise soil dynamic behaviour are the shear modulus and the material damping ratio. At small strain levels (typically less than 0.001%), the soil dynamic response is linear, where the two parameters remain constant. Once the strain level exceeds the elastic threshold limit, the soil exhibits non-linear response. Then the shear modulus decreases with strain while the material damping ratio increases.

In-situ determination of the linear properties of soil has been extensively studied, with the development of the widely accepted small strain tests. The non-linear characteristics measured from laboratory tests differ from the field test results due to sampling disturbance. Field tests attempted in previous studies are generally difficult to control and lack adequate flexibility.

The field test for this study was conducted at two locations, the National Geotechnical Experimentation Sites (NGES) and the Fitzpatrick Ranch site. Vibroseis trucks were used to exert a vertical static load and a range of horizontal dynamic loads into the soil via a circular footing. An array of horizontal geophones were installed under the footing to detect the shear waves and the signals were analysed to measure the shear modulus and shear strain. Cyclic horizontal loads were applied in steady-state dynamic tests to evaluate the non-linear characteristics of soil. Downhole and crosshole tests were carried out in conjunction with the steady-state dynamic tests to evaluate the linear stiffness of soil.

Geophone outputs from steady-state dynamic tests are supposed to be approximately sinusoidal under a sinusoidal dynamic load input. The time lag between two geophone outputs is measured to calculate the shear modulus. The shear strain is estimated by either comparing the relative displacement when the upper geophone is at peak displacement or by taking the average relative displacement across the steady-state range. Three analysis methods have been applied to analyse the steady-state test data, namely the sinusoidal curve fitting method, the by-eye method and cross correlation. The sinusoidal curve fitting method simplifies the analysis by fitting

sinusoidal curves to the geophone outputs using least square error method. It is not feasible as it has been shown to have a large percentage error ($>10\%$) even when the geophone output is close to a sinusoidal curve. The by-eye method measures the average time lag and relative displacement in the steady-state cycles directly from the geophone outputs. The results produced have a large scatter mainly due to poor quality of data. A proposed modification would be measuring the time lag from a specific feature in the geophone outputs to improve the consistency of results. Finally, cross correlation has produced the most reliable results but verification is necessary to show that its measurements are accurate to the true soil behaviour.

The characteristics of the results appear to depend on the vertical static load. At a low static load, the results are coarser as the geophone outputs generally have a poorer quality, possibly due to poor contact at the soil-footing interface and poor compaction of soil for adequate transmission of shear wave. At a mid-level static load, the results generally show more notable linear and non-linear soil characteristics. At a high static load, the results exhibit less of the linear characteristic. Instead, the shear modulus tends to become larger at very small strain levels. This could possibly be due to the breakage of cementation at small strain levels. The high stress differential in soil due to the high static load could also be the cause for this anomaly.

Furthermore, the results show that the frequency of dynamic loading has an influence on the shear modulus characteristics. Tests carried out with 100Hz dynamic load have produced anomalous results, possibly due to irregular dynamic load input to the footing. Otherwise, the dependency on frequency can be explained by the possibility of rocking. Vertical geophone signals have shown that rocking occurred during the tests. The dynamic response of soil to rocking is sensitive to frequency, unlike horizontal shearing. The pressure waves generated from rocking would also have induced extra strains in the soil. Consequently, the assumption for the analysis that only horizontal shear was experienced by the soil is no longer valid.

Despite the inconsistency in the results, this study has led to ideas for future studies. In particular, the effect of rocking and uneven cementation should be investigated to clarify questions raised by this study. Moreover, the determination of the characteristic shear strain under cyclic shear waves and possibly some pressure waves would require further verification.

Small Strain Stiffness of Soils from Field
Measurements

by

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Fourth-Year Undergraduate Project

In Group D, 2014/2015

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I hereby declare that, except where specifically indicated, the work submitted herein is my own original work.

Signature: _____

Date: _____

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1. Introduction

The analysis of soil dynamic response has always been a challenge in geotechnical earthquake engineering. A more thorough characterisation of soil dynamic properties will help to reduce the related uncertainties, leading to more optimised foundation design of structures. Research studies have been conducted for decades to study soil dynamic properties and have achieved significant developments. There are two key parameters that characterise the dynamic behaviour of soil, namely the damping ratio (D) and the shear modulus (G). Laboratory testing has shown that soils respond to dynamic excitation in linear or non-linear behaviour, which depends on the shear strain level. Over the past few decades, progress has been made in the in-situ determination of linear shear modulus. On the other hand, an acceptable method to determine the in-situ non-linear dynamic properties has yet to be developed.

This report focuses on the data analysis of raw data from the field tests that were developed and performed in 2006 and 2007 to determine the in-situ soil dynamic properties. The weaknesses and potential refinement of analysis methods will be investigated. The results produced by these analysis methods will be evaluated to interpret the soil dynamic behaviour under different loading and soil conditions. They will also be used to identify necessary improvements of the in-situ tests and to highlight future studies required to improve the interpretation of results.

1.1. Theory

Figure 1.1.1 shows the stress-strain relationship in shear of a soil element under normal shearing (Monotonic Backbone Curve) and cyclic shearing (Typical Cyclic Curve). The material damping ratio is related to the area of the hysteresis. It remains zero in the linear region and increases with shear strain level in the non-linear region. The linear shear modulus, G_0 or $G_{\max}$, is given by the tangential stiffness of the backbone curve at a very small shear strain level. When the strain is small, the soil particles are locked in position due to the stiffness of the intergranular contacts. The linear shear modulus comes from the contact stiffness between the soil particles. It is independent of the strain level but is influenced by factors such as confining pressure, soil density, soil type, cementation, etc. The shear modulus remains constant until the shear strain exceeds a certain level, known as the elastic threshold shear strain, γ_t^e , which is typically not more than 0.001% but depends on factors such as soil uniformity, density and the confining pressure, etc.

Once the shear strain exceeds the elastic threshold limit, the shear modulus curve becomes non-linear and decreases with strain. The soil particles begin to break free from the intergranular contacts and displace. The sliding of soil particles results in reduction in intergranular contacts causing a loss in shear modulus. Material damping increases due to energy dissipation during the sliding. The typical linear and non-linear characteristics of shear modulus and material damping ratio are shown in Figure 1.1.2.

The linear shear modulus depends on the confinement pressure. As the pressure in the macro environment increases, the intense pressure in the intergranular contacts causes the contacts to flatten. The stiffness of the soil increases as the grains come into better contact. According to Wroth and Housby (1985), the relationship between the linear shear modulus (G_0) and confining pressure (σ_0) is given by:

$$\frac{G_0}{P_r} = A \times \left(\frac{\sigma_0}{P_r} \right)^{0.5} \quad (1.1)$$

Where, P_r is a reference pressure

A is a factor dependent on the choice of reference pressure

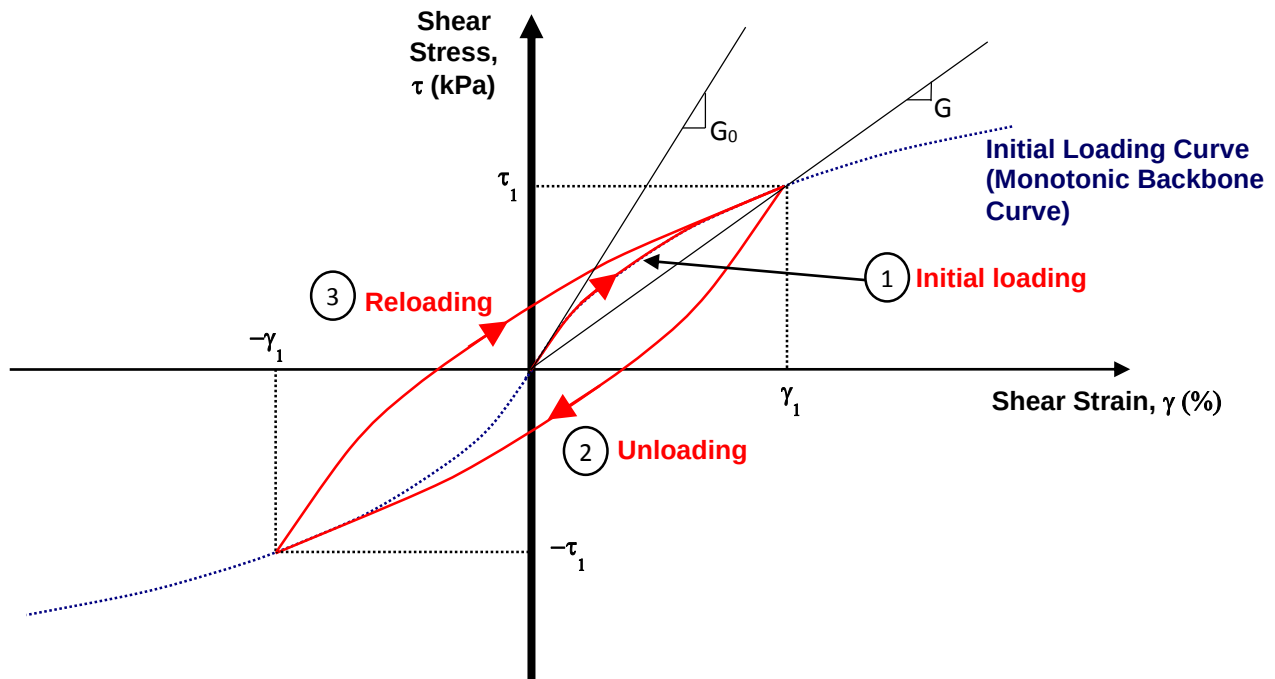


Figure 1.1.1 - Shear stress-strain relationship in one cycle of dynamic shearing

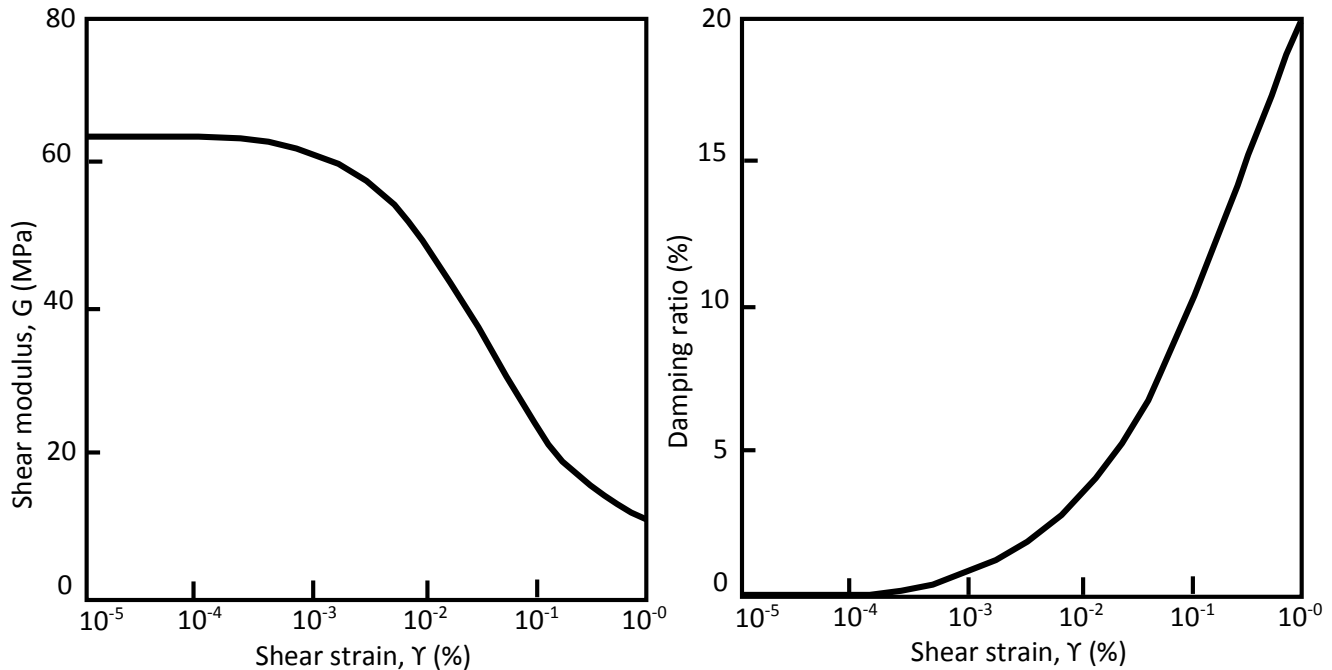


Figure 1.1.2– Typical variation of shear modulus (G) and material damping ratio (D) with shear strain

1.2. Background

Measurement of linear and non-linear properties has been made possible through laboratory tests such as resonant column tests and torsional shear tests. However, the soil dynamic properties obtained by these laboratory tests often have a significant discrepancy with the in-situ properties, as the soil samples are always subject to sample disturbance. One of the major challenges with the in-situ determination of nonlinear properties is to create an active wave source of excitation to induce sufficient shear strain for the non-linear behaviour of soil. In previous studies, active wave sources were provided in two ways: a) using earthquakes; b) developing testing devices to apply dynamic loading.

- a) Using earthquakes as the active wave source has the advantage of providing us a more direct understanding of the soil dynamic behaviour in a real seismic event. It does not require the effort to create shear waves in the soil to simulate seismic shearing. However, it is limited in many ways due to unpredictable nature of the magnitude, location and time of an earthquake. As it often takes many years for an earthquake to occur, the accuracy of the embedded sensors become questionable over time. Regular inspections and calibration of the sensors

are required. The unpredictable nature of earthquakes would also mean that such tests lack sufficient standardisation and flexibility. The choice of locations would be limited and it is not possible to achieve a full range of shear strains to obtain complete characteristics of the non-linear dynamic properties.

- b) A number of test devices and techniques have been developed to attain more standardisation and control. Previous studies have produced results that widened our understanding of the in-situ soil dynamic properties, but can be improved to acquire better and more complete results. The data produced in previous studies have been insufficient to evaluate the validity of the methods and the soil dynamic properties. More studies with improvements are necessary to confirm that the methods and results are valid. The devices used have limited control over the dynamic loading, making it difficult to achieve the desired range of shear strain. For instance, in the study by Salgado et al. (1997), a large-strain seismic crosshole test (LSCT), in which the large-strain range was induced by dropping hammers of various weights from various heights, were conducted. It is difficult to control precisely the magnitude and the duration of loading to cause the shear wave in the soil. Finally, some of the field tests are not applicable to all types of soil, as the preparation of such tests would cause too much disturbance to certain types of soil, such as saturated loose sand and cemented soils.

More recent field tests by Kurtulus (2006) and Stokoe et al (2006) allow more control over the dynamic load through the use of vibroseis trucks. The excitation was transmitted into the soil via a drilled shaft casted in place and the signals were picked up by the sensors around the shaft. In the field tests conducted for this study, instead of using a shaft, the excitation was applied via a circular footing to minimise the amount of drilling into the soil. The main goal is to improve the field testing techniques and analysis methods for more robust and consistent measurements. The field tests took place at two locations, the National Geotechnical Experimentation Sites (NGES) at Texas A&M University (College Station, Texas, USA) and the Fitzpatrick Ranch site (Austin, Texas, USA).

1.2 Objectives

Primarily, the field tests should be capable of evaluating the linear and non-linear shear modulus across a range of shear strains. However, there is a potential to use the field tests to evaluate other parameters such as non-linear Young's modulus and material damping ratio.

The major objective of this study is to develop a generalised field testing technique that is practical in use. It should be both repeatable and flexible in evaluating various types of soil, especially the hard-to-sample soils, such as gravelly soil, cemented soil and sensitive soil. The use of vibroseis trucks provides the opportunity to develop a more standardised field testing technique by allowing easy control of dynamic loading and static loading. Finally, the accuracy of laboratory testing has always been affected by issues such as sampling disturbance, difficulty in replicating the field conditions, equipment compliance and time effects. It is hoped that the field testing technique can eliminate these problems through evaluating the soil dynamic properties directly with minimal disturbances to the soil.

Another objective of this study is to explore different approaches to analyse the raw data and subsequently develop an analysis method that is robust and capable of producing the most accurate results. The results produced can also reveal the weaknesses of the field testing, giving us clues of possible improvements and future studies required.

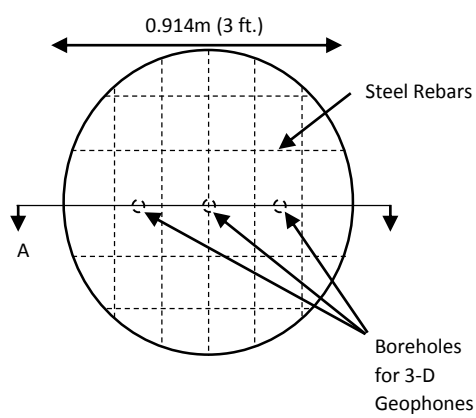
2. Apparatus and experimental techniques

The general idea behind the field test is to create a simple horizontal shear wave that propagates vertically down into the soil, so that the analysis of the signals from the embedded sensors could be simplified. The sensor signals will be used to measure the shear modulus and shear strain. Any disturbance to the soil should be minimised to ensure a true measurement of the in-situ properties.

2.1. Test Set-up

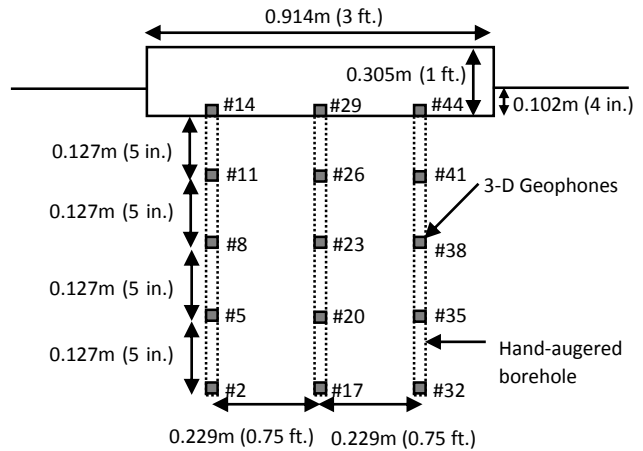
The general test set-ups for the NGES and the Fitzpatrick Ranch site are presented in Figure 2.1.1(a) and (b) respectively. To minimise the extent of any movement other than horizontal shearing and to ensure an adequate transmission of shear wave at the footing-soil interface, there should be no gap between the footing and the soil, and the footing should be horizontal.

Before casting the footing, hand-augered boreholes are dug for the installation of geophones. The geophones should be installed at the exact locations and orientations to avoid picking up undesired signals from movements in other directions. Caution should be taken to minimise disturbance to soil during drilling, such as keeping the drilled soil in sealable plastic bags to prevent moisture loss and backfilling back to the original depth. Nevertheless, some disturbance due to shearing and loss of water content is inevitable.

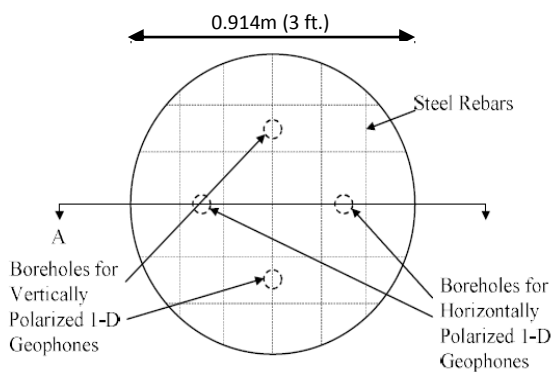


(a) Plan view

Figure 2.1.1(a) - Test set-up at the NGES

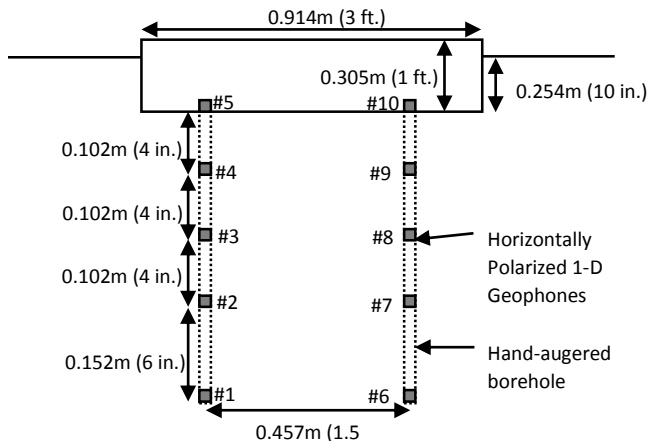


(b) Cross-section A-A



(a) Plan view

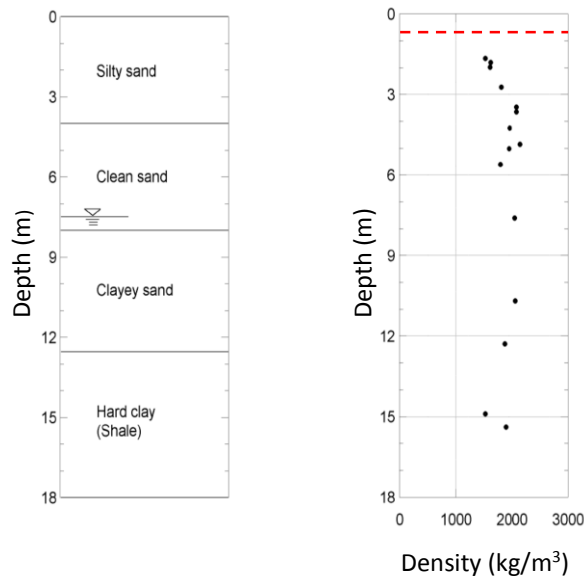
Figure 2.1.1(b) - Test set-up at the Fitzpatrick Site



(b) Cross-section A-A

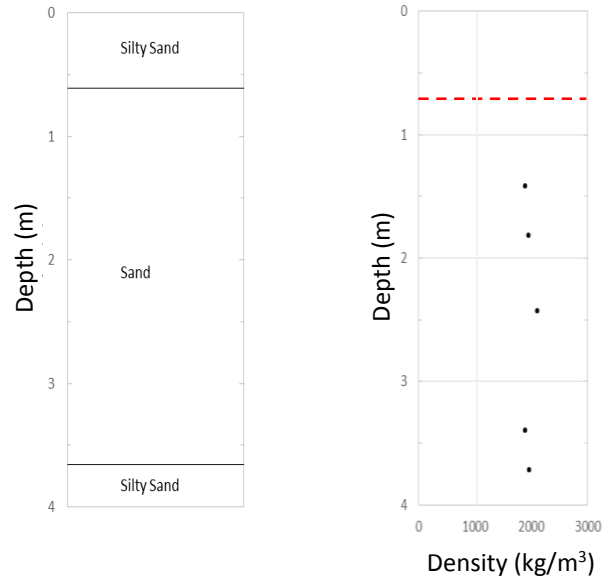
2.2. Soil Properties

The soil density for shear modulus calculation is assumed to be uniformly 1800kg/m^3 for both sites. The red dashed lines in the Figure 2.2.1 (a) and (b) indicate the approximate depth up to which geophones were installed. The groundwater level was well below the test instruments.



a) Stratigraphy (Briaud, 1997) b) Density (Briaud, 1997)

Figure 2.2.1(a) - Stratigraphy, soil density profile at the NGES



a) Stratigraphy

b) Density

Figure 2.2.1(b) - Stratigraphy, soil density profile at the Fitzpatrick site

2.3. Test Instruments

2.3.1. Concrete footing

The wave is transmitted into the soil through a circular footing of diameter 0.914m (3ft) and thickness 0.305m (1ft). The shape, dimension and the material for the footing are important factors to consider to ensure a good quality of the test results. A circular shape allows the calculation for confining pressure to be analysed as an axisymmetric case and minimises the potential error caused by misalignment in the direction of horizontal excitation. The thickness is chosen in such a way that it is thick enough for the footing to be considered as rigid, but thin enough to prevent excessive rocking. Finally, the concrete is cast in-situ to allow a better contact with the soil for better wave transmission. Any irregularities around the footing can interrupt the wave transmission, and consequently affect the quality of the results.

2.3.2. Geophones

1D geophones and 3D geophones were used at the Fitzpatrick Ranch site and the NGES respectively to pick up the shear wave. The quality of the geophone output depends on many factors such as soil homogeneity and alignment of geophones. The geophone output is linearly proportional to its own velocity caused by the horizontal shearing of the surrounding soil. The multiplication factor depends on the frequency of the wave source, as shown in Figure 2.3.1 (a) and (b).

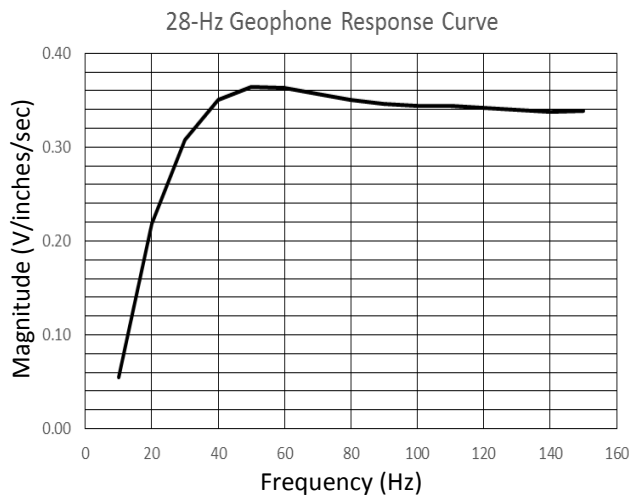


Figure 2.3.1(a) - Typical geophone response curve for the NGES

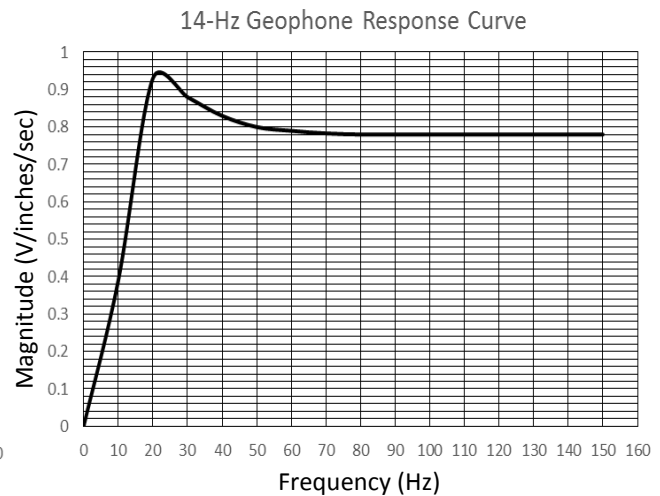


Figure 2.3.1(b) - Typical geophone response curve for Fitzpatrick Ranch site

2.3.3. Mobile shakers

Two types of mobile shakers were used, namely 'Thumper' and 'T-Rex', as shown in Figure 2.3.2(a) and (b). 'Thumper' has a peak dynamic loading output 27kN (~6000lb), in both the vertical and horizontal directions, between 17Hz and 225Hz. On the other hand, 'T-Rex' can produce a larger dynamic loading output at a lower frequency, 267kN (~60,000lb) between 12Hz and 180Hz for vertical loading and 134kN (~30,000lb) between 5Hz and 180Hz for horizontal loading.



(a) Small-capacity, three-axis vibroseis truck, Thumper



(b) Large-capacity, three-axis vibroseis truck, T-Rex

Figure 2.3.2 – Pictures of mobile shakers, Thumper and T-Rex (Stokoe et al., 2004)

2.3.4. Other apparatus

This includes load cells used to measure the static load applied onto the footing, a dynamic signal analyser, a DC power supply to power the instruments and a function generator. The dynamic signal analyser samples the signals from the geophones simultaneously at a sampling rate of 51.2 kHz, allowing frequency components up to 25.6 kHz to be recovered without any distortion based on sampling theorem. The function generator produces a sinusoidal wave signal to control the magnitude and the frequency of the output dynamic force.

2.4. Steady-State Dynamic Test

The aim of this test is to obtain data for the non-linear relationship between the shear modulus and shear strain. A vertical static load and a sinusoidal horizontal dynamic force are applied onto the footing. This creates a horizontally polarized shear wave propagating vertically downwards through the geophone array. The geophones detect the shear wave and produce voltage outputs that represent the velocity time histories of the geophones.

2.5. Small Strain Tests

The small strain tests are namely crosshole (CH) and downhole (DH) tests. Figure 2.5.1 shows the general set-up for the two tests. The purpose is to measure the linear shear modulus and to ensure that the soil structure remains intact after each stage of steady-state dynamic loading. In these tests, a small-magnitude transient impact is applied to create a very small strain in the soil and is detected by the embedded geophones. The geophone outputs will be used to evaluate the variation of linear shear modulus with depth as well as with confining pressure.

In a crosshole test, a borehole is drilled at a distance from the footing and a PVC tube is installed into the borehole. The tube is divided into segments with lengths corresponding to the geophone spacing. The segments are connected by flexible membranes. A transient impact is applied to individual segment, creating a vertically or horizontally polarized shear wave (S_{hv} and S_{hh}) that travels horizontally through the geophones at the corresponding depth.

In a downhole test, a transient impact is applied to the side of the footing, creating a horizontally polarized shear wave propagating vertically downwards. The subsequent geophone output is then analysed. For this project, only data from the downhole tests have been analysed as the crosshole test data have not been retrieved.

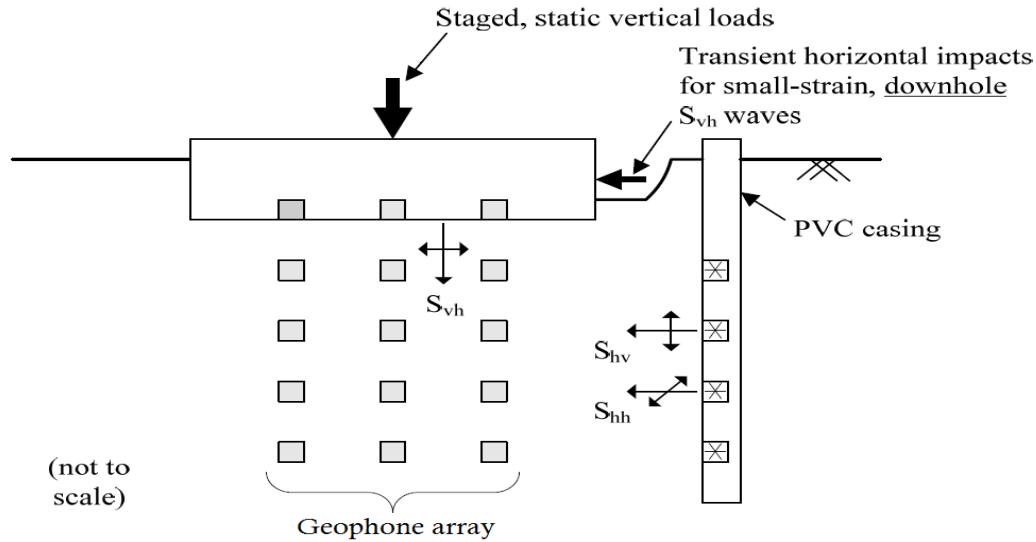


Figure 2.5.1 - Typical field set-up for crosshole and downhole tests (modified from Park, 2007)

2.6 Test Sequence

The loading is applied in stages. The static load increases after each stage. In each stage, small-strain tests are performed before and after the steady-state dynamic tests to assess whether the soil is still intact after the series of steady-state dynamic tests have been performed. The magnitude of the steady-state dynamic load in each stage is increased from small to large, in order to cause a range of shear strains in the soil.

Figure 2.6.1 shows the general sequence of loading and tests. Stages 1, 5 and 7 are static load-settlement tests. They are not discussed here as they are beyond the primary scope of this study. For the NGES, only data from tests carried out at a static load of 35.6kN (8kips) have been analysed. The frequencies of dynamic loading applied are 50Hz and 100Hz. Data for other static loads have not yet been retrieved. As a result, comparison with results from other stages is not available. However, the results can be used to investigate the possible sources of error and to give clue of future improvements. For the Fitzpatrick Ranch site, data from tests carried out at static loads of 13.3kN, 26.7kN and 53.4kN (3, 6 and 12 kips) have been analysed. The frequencies of dynamic loading applied are 40Hz, 50Hz, 70Hz and 100Hz. The static loads and dynamic loading frequencies at the two locations are summarised in Table 2.6.1

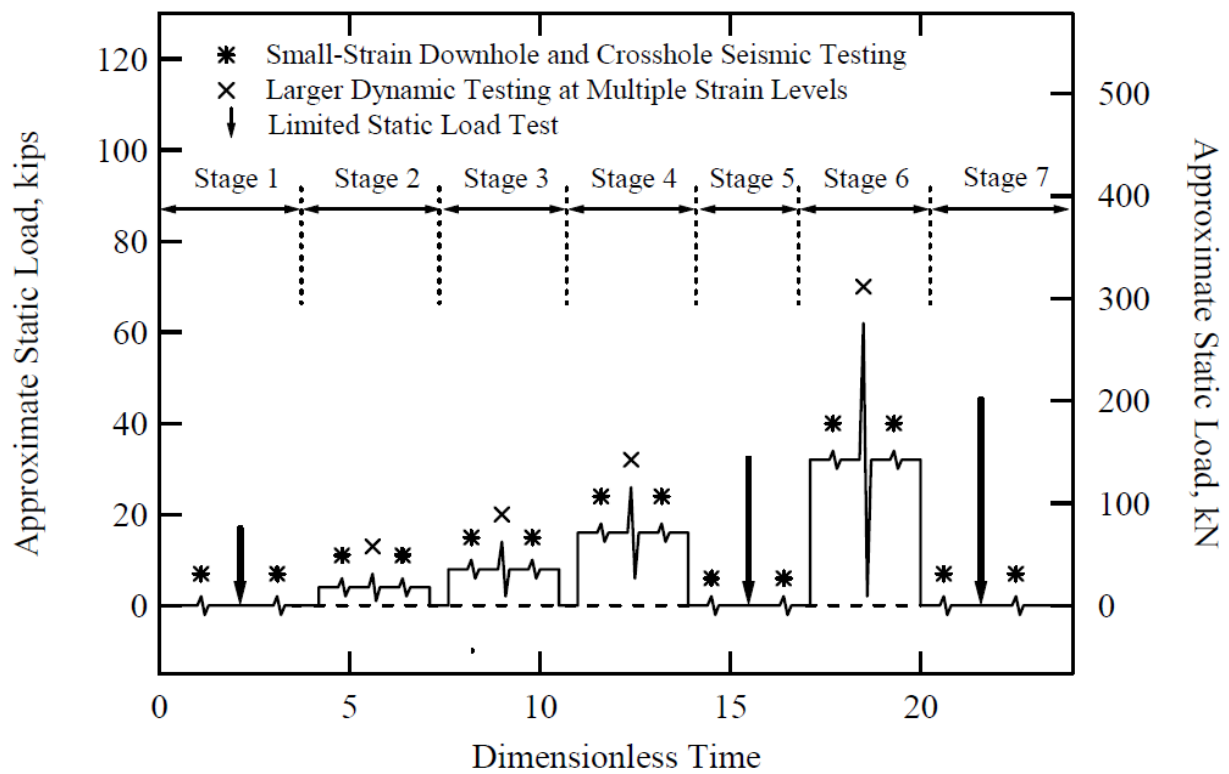


Figure 2.6.1 - Generalized loading sequence (modified from Stokoe et al., 2006)

Location	NGES	Fitzpatrick Ranch site
Static load - steady-state dynamic tests (kN)	35.6 (8kips)	13.3, 26.7, 53.4 (3, 6, 12kips)
Dynamic load frequency (Hz)	50, 100	40, 50, 70, 100
Static load - downhole tests (kN)	0, 4.4, 8.9, 17.8, 35.6 (0, 1, 2, 4, 8kips)	13.3, 26.7, 53.4 (3, 6, 12kips)

Table 2.6.1 – Summary of static loads and dynamic load frequencies for both sites

3. Data Analysis Approaches

3.1. Steady-State Dynamic Test

3.1.1. Raw Data

Figures 3.1.1 and 3.1.2 show the typical geophone outputs from steady-state dynamic tests at both sites. As the dynamic loading is sinusoidal, the geophone outputs are expected to be sinusoidal too with a phase lag due to the time it takes for the shear wave to travel down the soil. The amplitude of the shear wave decreases with depth as the shear wave spreads out and energy is dissipated in cyclic shearing. The time lag between the geophone outputs is the time for the wave to travel between the geophones.

The wave travelling speed is primarily governed by soil properties such as density, void ratio, confining pressure, etc. But the quality of geophone outputs directly influences the measurement of the wave travelling speed. The geophone outputs have experienced various levels of distortion. Some of the poor quality geophone outputs have multiple minor peaks and more noise. An example of such is given in Figure 3.1.3. In that example, the dynamic response of the footing measured by the geophone installed in the footing somewhat does not have a regular sinusoidal shape, indicating that there might be issues with the dynamic load input from the mobile shaker. The outputs from the lower geophones are usually more irregular. As the shear wave travels through the soil, the wave becomes more distorted due to inhomogeneity of the soil. Other factors impairing the quality of data include local irregularities around geophones, inaccuracy in geophone location and orientation, rocking of concrete footing, etc.

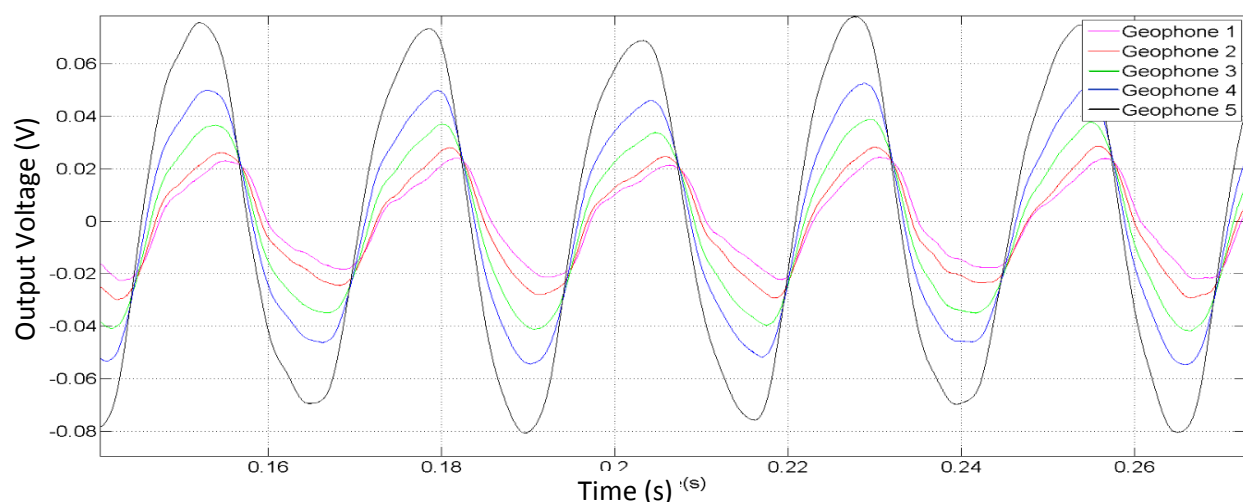


Figure 3.1.1 – Example of Geophone output voltage against time under 26.7kN (6kips) static load and 40Hz dynamic load at the Fitzpatrick Ranch Site

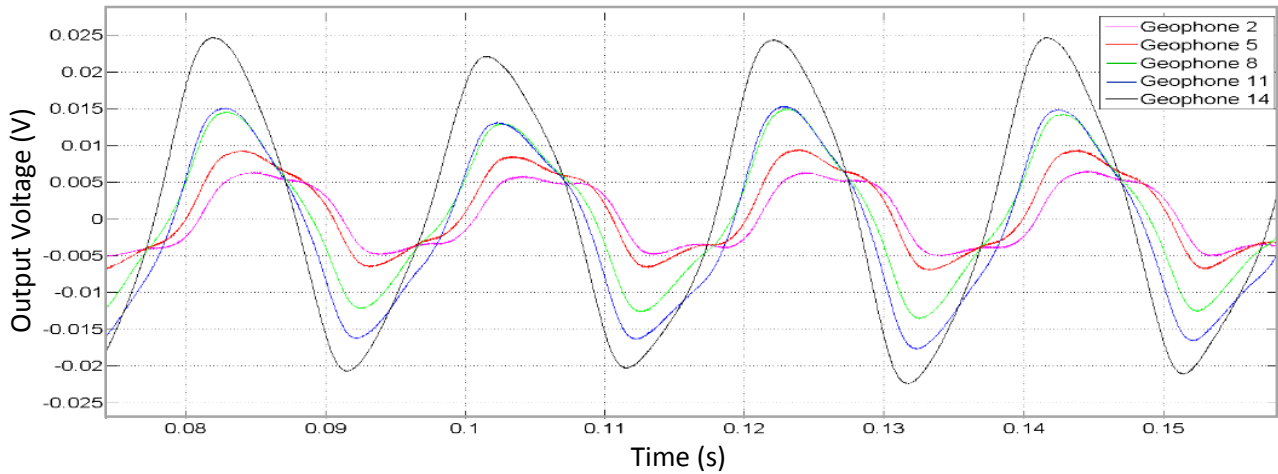


Figure 3.1.2 – Example of geophone output voltage against time (35.8kN (8kips) static load and 50Hz dynamic load at the NGES)

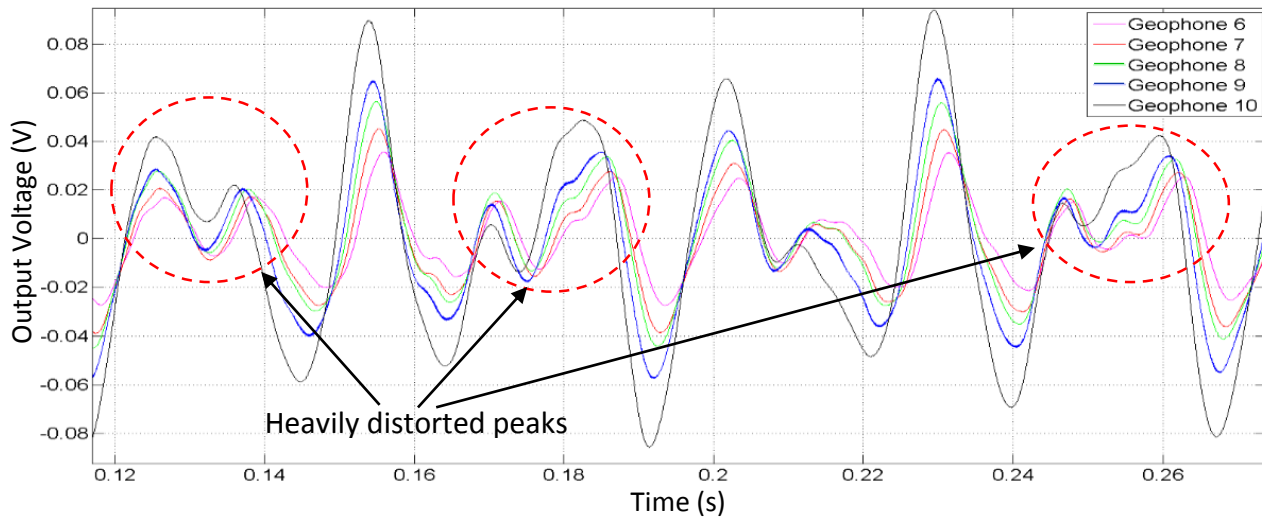


Figure 3.1.3 - Example of poor quality data (13.3kN (3kips) static load and 40Hz dynamic load at the Fitzpatrick Ranch Site)

3.1.2. Evaluation of Shear Modulus and Shear Strain

The major focus is to find a consistent method that is robust to poor quality of data to evaluate the shear modulus. The shear modulus is calculated by the following equation:

$$G = \rho V_s \quad (4.1)$$

where, G is the dynamic shear modulus

ρ is the soil density

V_s is the shear wave velocity

The soil density is assumed to be uniformly 1800kg/m^3 as stated in Section 2.1. The horizontally polarised shear wave arrives at the geophones at different times depending on the geophone depth. The shear wave velocity can be calculated by dividing the distance between two geophones in the same borehole by the time lag between the signals from the two geophones. As the time lag is very small, it is very sensitive to the quality of data. Three methods, the sinusoidal curve fitting method, the by-eye method and cross-correlation, have been used to measure the time lag. The method that yields the most consistent results will be chosen and the results from the other two methods used for verification.

3.1.2.1. Sinusoidal Curve Fitting Method

This method is taken from the study by K. Park (2010). By finding a best fitting sinusoidal curve, the evaluation of shear modulus and shear strain is simplified by comparing the two sinusoidal curves. The best fitting curve is determined by optimising the amplitude, offset, phase and period. Geophone output offset, calculated by averaging the output across the entire time domain, arises from influencing factors such as interference, geophone calibration, etc. The geophone outputs must be adjusted to set the offset to zero. The amplitude is determined by averaging all the peak magnitudes of the cycles within the steady-state range. The period and phase are optimised through the least square error method. Figure 3.1.4 shows the comparison between a geophone output and its best fitting sinusoidal curve.

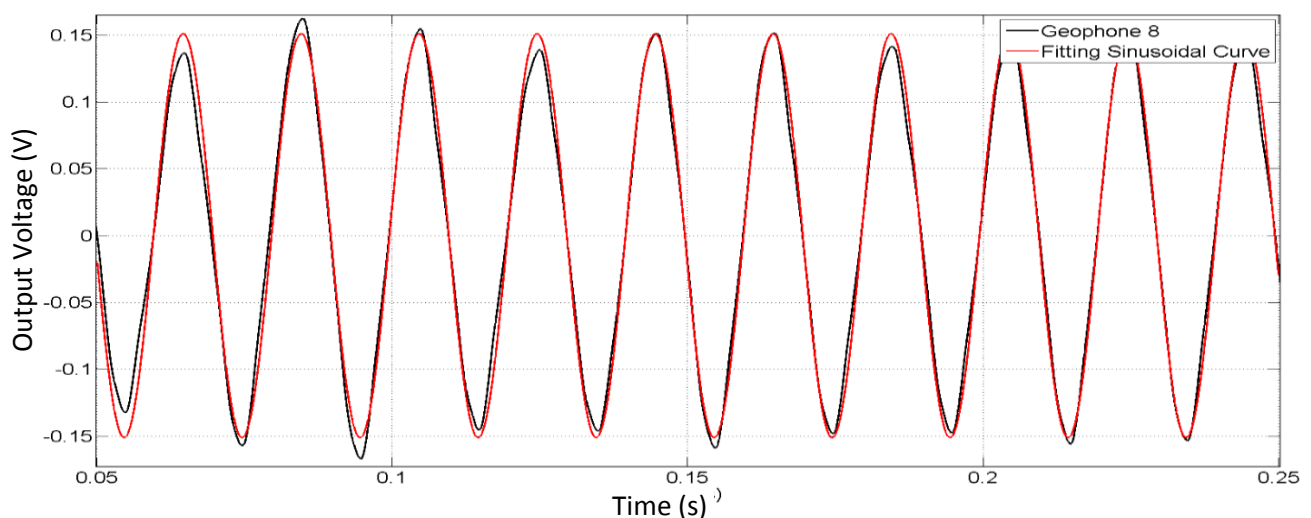


Figure 3.1.4 - Geophone 8 output and its fitting sinusoidal curve (26.7kN (6kips) static load and 50Hz dynamic load at the Fitzpatrick Ranch Site)

The time lag between the two fitting sinusoidal curves for the corresponding two geophone outputs is then measured, shown as Δt in Figure 3.1.5. The distance between the geophones is divided by Δt to estimate the shear wave velocity, V_s . Having determined V_s , the shear modulus, G , can be calculated using Equation 3.1. A time lag of 0.0015s is measured in the example from Figure 3.1.5. The distance between Geophones 8 and 10 is 0.203m (8 in.). Hence, V_s is 140.01m/s. By taking the soil density as 1800kg/m³, the shear modulus would be 35.3MPa.

To evaluate the shear strain, first the velocity time history is derived using the geophone response curves, as shown in Figure 2.3.1, on the geophone output. Then the displacement time history is derived by integrating the velocity time history. The measurement of the relative displacement between two geophones, Δu , is shown in Figure 3.1.6. The physical illustration of the relative displacement between two geophones is given in Figure 3.1.7. The shear strain, γ , is then calculated by:

$$\gamma = \frac{\Delta u}{d} \times 100\% \quad (3.2)$$

where, Δu is the relative displacement when the upper geophone is at peak displacement
 d is the distance between two geophones

This method simplifies the evaluation and minimises the error by optimising the fitting curve. However, if the geophone output differs too much from a sinusoidal curve, it becomes much less reliable and valid.

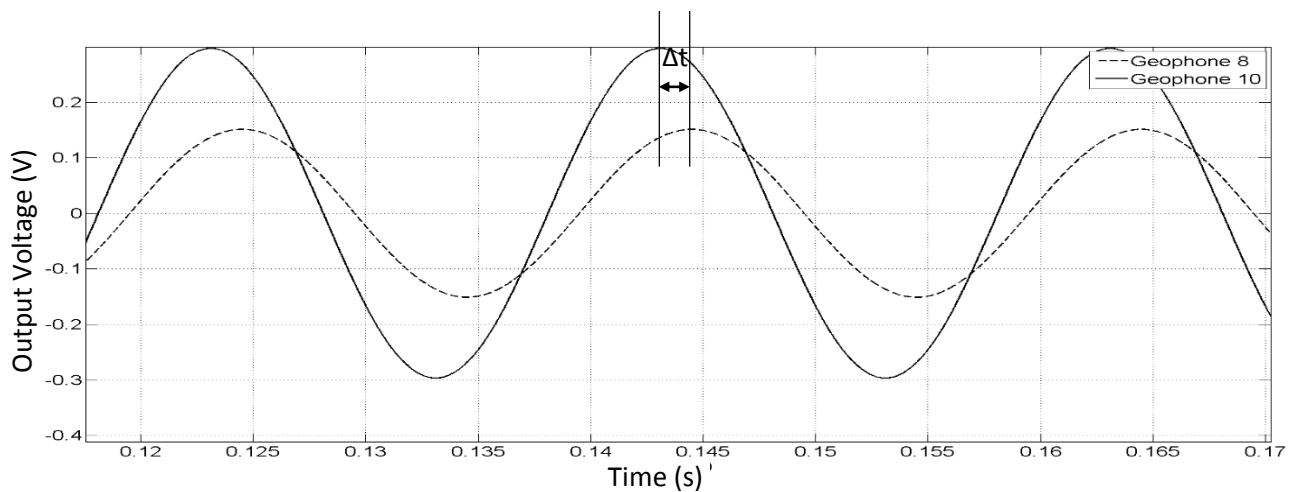


Figure 3.1.5 - Fitting sinusoidal curves for Geophones 8 and 10 output (26.7kN (6kips) static load and 50Hz dynamic load at the Fitzpatrick Ranch Site)

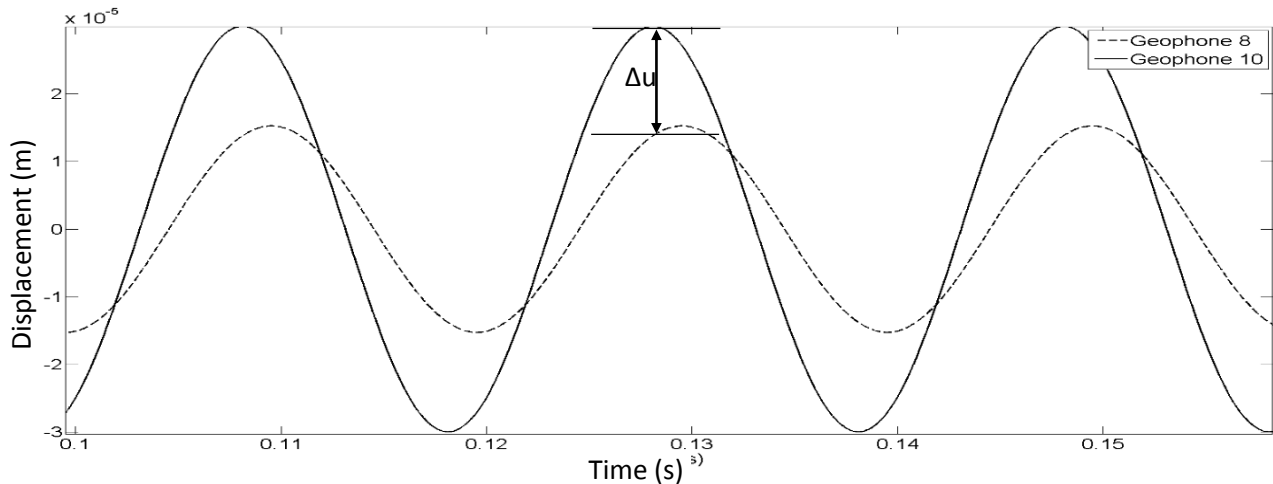


Figure 3.1.6 - Fitting sinusoidal curves for Geophones 8 and 10 displacement time history (26.7kN (6kips) static load and 50Hz dynamic load at the Fitzpatrick Ranch Site)

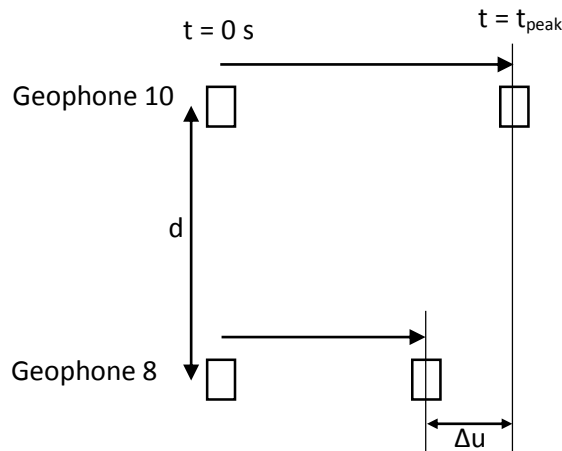


Figure 3.1.7 – Illustration of relative displacement between two geophones

3.1.2.2. By-Eye Method

This is an alternative method that directly evaluates the shear modulus and shear strain from the actual data regardless of the quality of the data. It has the advantage of keeping the data unaltered. Thus, given that the data quality is adequate, it should produce more accurate results. If the data quality is poor, a larger scatter is expected as the peaks might be difficult to identify. Distortion to the geophone outputs could mean that the time lag measured by this method might not be the actual shear wave travelling time. Therefore, it would be beneficial to study the data quality to verify the results by this method.

A number of cycles within the steady-state range of the geophone output data are chosen to evaluate the peak-peak time lag of each cycles ($\Delta t_1, \Delta t_2, \Delta t_3$, etc.). An average of the peak-peak time lags of all the chosen cycles is taken to be the estimated wave travelling time between the geophones. Then, the shear wave speed and shear modulus can be calculated in the same way as the sinusoidal curve fitting method using Equation 3.1. In Figure 3.1.8, 8 cycles out of 11 cycles in the steady-state range are evaluated. The first and last few cycles are not included as they are normally in a transition between static condition and steady-state dynamic shearing.

The shear strain is measured in a similar fashion as the sinusoidal curve fitting method, except that the data is analysed directly. A displacement time history is obtained by applying the trapezoidal integration method to the velocity time history derived from the geophone output. Figure 3.1.9 shows an example of a displacement time history produced by applying trapezoidal integration. A sampling rate of 51.2 kHz is high enough that the error arising from trapezoidal integration is negligible. The relative displacement ($\Delta u_1, \Delta u_2, \Delta u_3$, etc.) is measured at each cycle. The final relative displacement, Δu , is taken as the average of the relative displacements for all the chosen cycles. Finally, the shear strain is calculated using Equation 3.2.

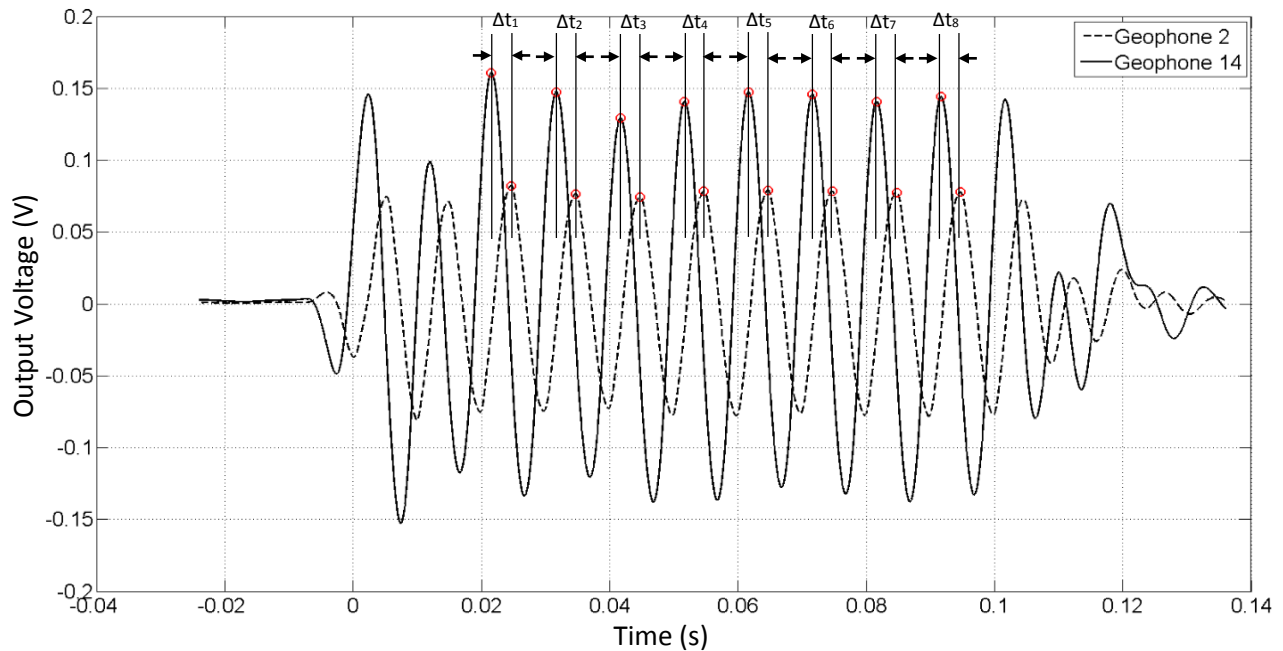


Figure 3.1.8 – Measurement of wave travelling times from the 3rd to the 10th peak in an 11-cycle dynamic loading (35.6kN (8kips) static load and 100Hz dynamic load at the NGES)

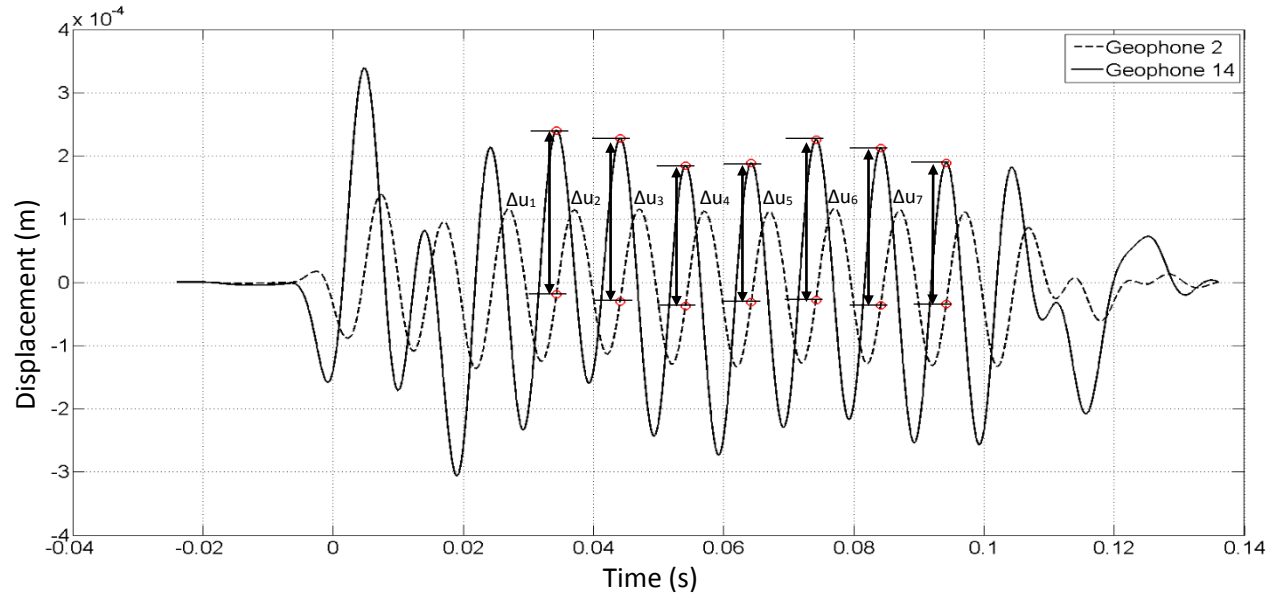


Figure 3.1.9 – Measurement of relative displacements between Geophone 2 and Geophone 4 from the 4th to the 10th peak in an 11-cycle dynamic loading (35.6kN (8kips) static load and 100Hz dynamic load at the NGES)

3.1.2.3. Cross Correlation

Figure 3.1.10 shows that when the geophone output is too distorted, it becomes deviated from a sinusoidal curve. Therefore, the results produced by this method will not be consistent. Figure 3.1.11 shows that the by-eye method can still yield reliable measurement of peak-peak time lag as the common characteristics, such as the peaks, are still identifiable in the two geophone outputs. However, the assumption that estimates the shear strain to be the relative displacement when the upper geophone is at peak displacement is no longer feasible as the displacement time histories of the geophones are rather irregular.

Therefore, a more robust method is adopted to measure the time lag and the shear strain. The time lag is measured by cross correlation which finds the required time shift of one geophone output to give the best match with the other. Cross correlation is subject to systematic error depending on the similarity of the geophone outputs, but it is able to obtain the results with an adequate level of consistency. Another issue of this method is that the precision is limited to the sample time which is 1.95×10^{-5} seconds ($1/51.2 \times 10^3$ kHz). The magnitude of time lag has an order of 0.001s. Therefore, the percentage error due to sample time is normally within a few percent. To compute the shear strain, instead of measuring the relative displacement at a specific time,

the shear strain is estimated to be the average shear strain within the steady-state range in the displacement plot derived by trapezoidal integration.

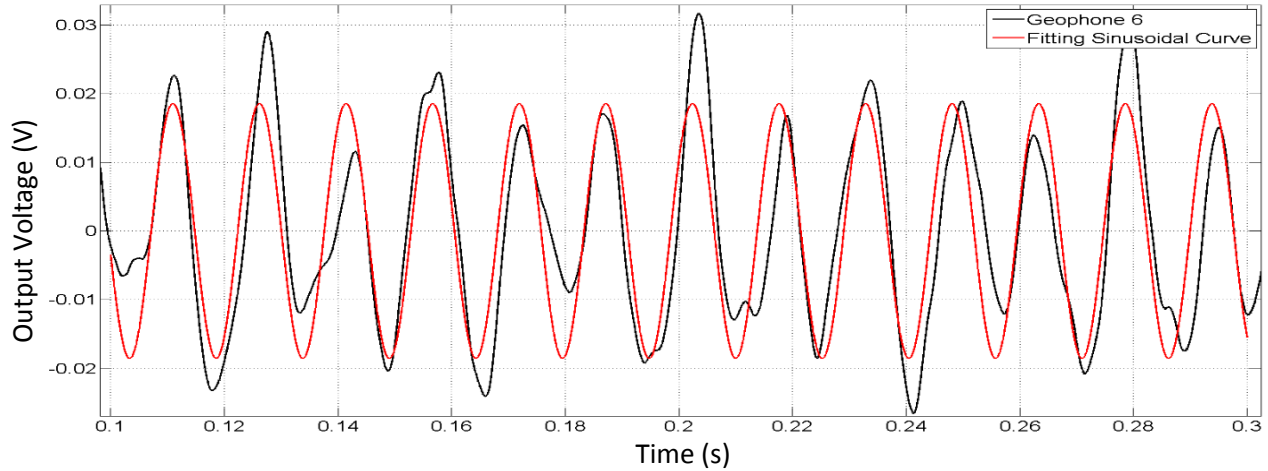


Figure 3.1.10 – Application of sinusoidal curve fitting method on a poor quality data (13.3kN (3kips) static load and 40Hz dynamic load at the Fitzpatrick Ranch Site)

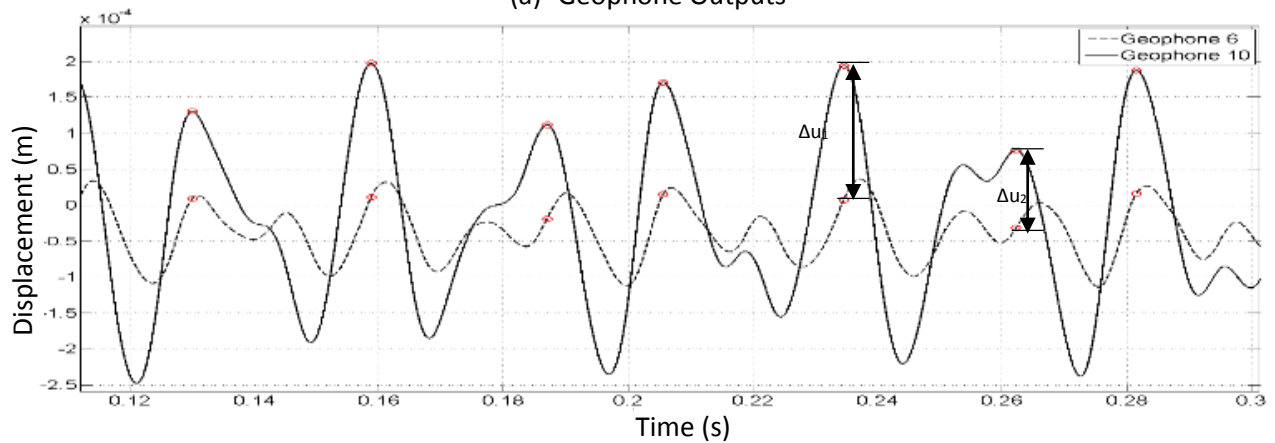
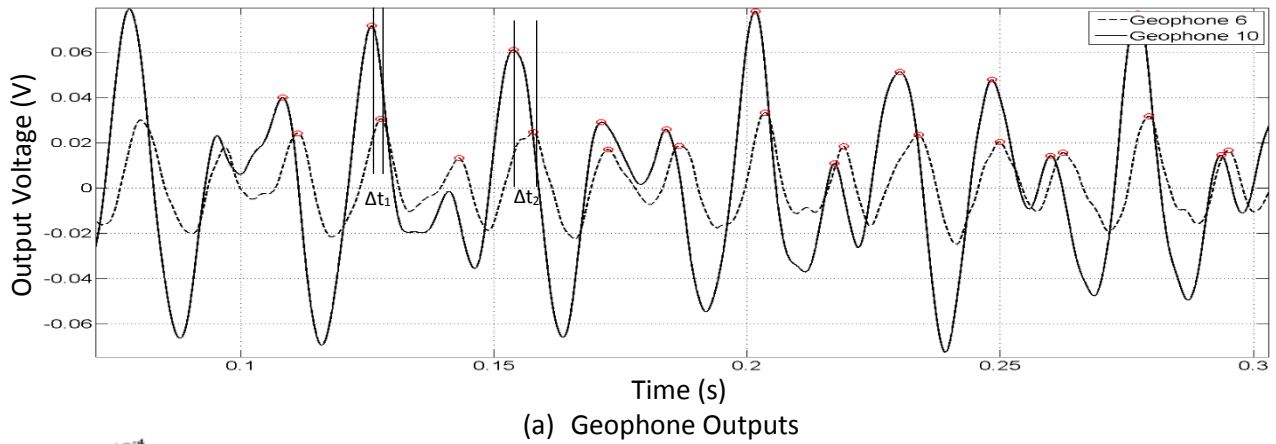


Figure 3.1.11 – Application of the by-eye method on a poor quality data (13.3kN (3kips) static load and 40Hz dynamic load at the Fitzpatrick Ranch Site)

3.2. Small Strain Tests

3.2.1. Raw Data

Only downhole test data have been analysed as the crosshole test data are not available. Downhole test results would be more relevant to the steady-state dynamic test results as the transient impact is applied onto the concrete footing, creating a vertically propagating horizontal shear wave like the steady-state dynamic tests. This allows comparison of linear shear moduli obtained from the downhole tests and steady-state dynamic tests. On the other hand, the crosshole test results would be useful in understanding more about the soil conditions under the footing. They are also expected to be more accurate than downhole tests, as they are less affected by problems such as soil inhomogeneity caused by backfilling of boreholes, rocking of footing, etc. Examples of geophone outputs from downhole tests at the Fitzpatrick Ranch site and the NGES are shown in Figure 3.2.1 and 3.2.2 respectively.

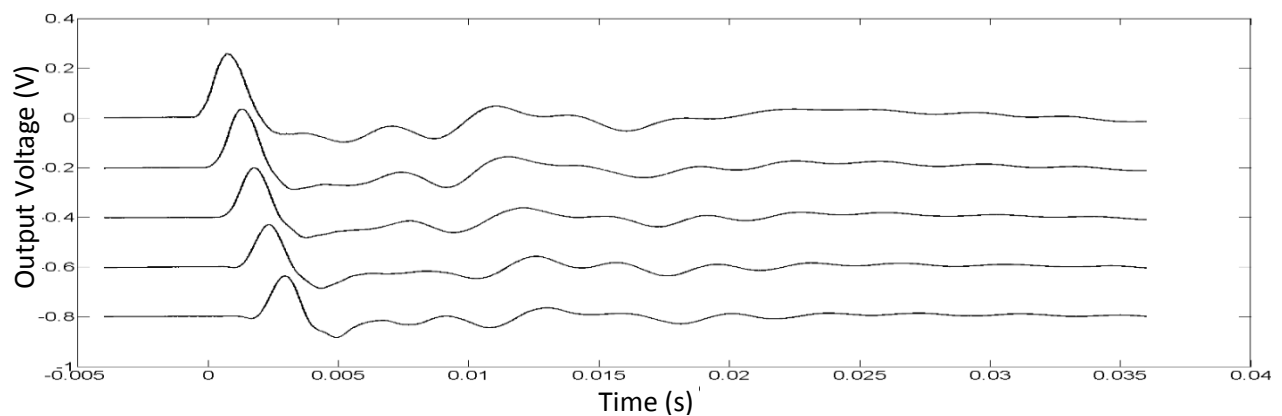


Figure 3.2.1 - Geophone Outputs from downhole tests before steady-state dynamic tests (26.7kN (6kips) static load at Fitzpatrick Ranch Site)

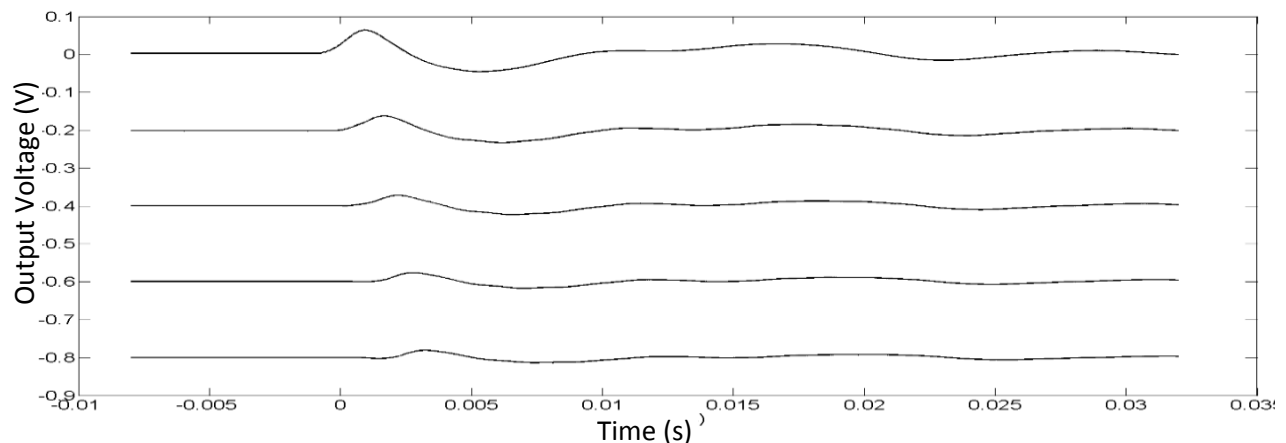


Figure 3.2.2 - Geophone Outputs from downhole tests before steady-state dynamic tests (35.6kN (8kips) static load at the NGES)

3.2.2. Evaluation of Shear Modulus

The linear shear modulus is evaluated in the same way as analysing steady-state dynamic tests, by measuring the wave travelling time between the geophones. The shear wave velocity is calculated from the wave travelling time and then used to calculate the linear shear modulus using Equation 3.1. The challenge with the determination of the wave travelling time is the selection of a specific geophone output characteristic that allows consistent measurement.

Figure 3.2.3 shows the typical geophone output from a downhole test. In some of the geophone outputs, there is a small trough (A-B-C) before the sharp peak. This arises from the arrival of pressure wave possibly due to rocking of footing, as illustrated in Figure 3.2.4. The pressure wave normally travels faster than the shear wave. Points A, B, C and D are compared between two geophone outputs. If the small trough due to the pressure wave does not exist in either or both of the geophone outputs, points A and B are disregarded. The times of the corresponding points are then compared to measure the time lag. It is expected that point D is the most suitable for measuring time lag. However, other points are still taken into consideration to verify the results.

Cross correlation has also been applied to verify the results produced by comparing peaks. It is not used to determine the linear shear modulus directly as the accuracy is limited by dissimilarity of geophone outputs.

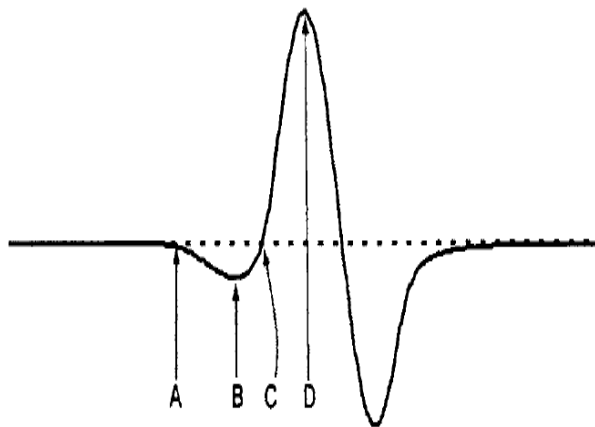


Figure 3.2.3 – Typical S-wave signal due to transient impact (Lee & Santamarina, 2005)

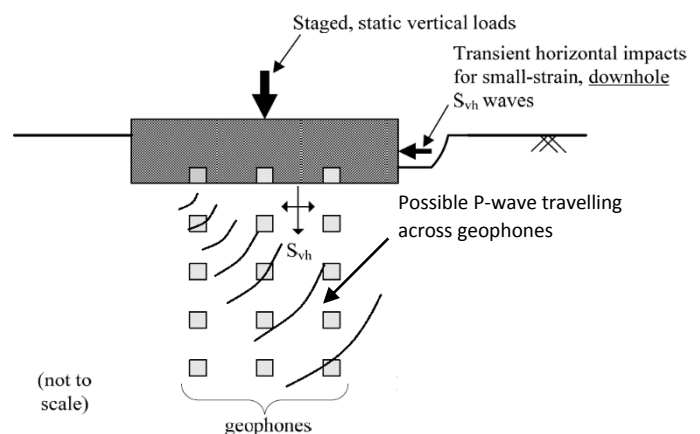


Figure 3.2.4 – Illustration of a P-wave travelling in a downhole test possibly due to rocking

Not only that the linear shear modulus evaluated from the downhole tests can be used to compare with the steady state dynamic test results, they can be used to assess the variation of stiffness in the soil and the effect of confining pressure on linear shear modulus. The confining pressure within the soil is calculated by estimating the vertical and horizontal effective stresses in the soil. The vertical effective stress, σ_v' , is assumed to be the superposition of stress due to static load on a circular footing (based on Boussinesq stress distribution) and stress due to self-weight of soil. Since the lateral pressure at rest (K_0) is unknown, the horizontal effective stress, σ_h' , is assumed to be equal to one-half of the vertical effective stress, i.e. $K_0 = 0.5$. The confining pressure, σ_0 , is then calculated using the estimated stresses:

$$\sigma_0 = \frac{1}{3}(\sigma_v' + 2\sigma_h') \quad (3.3)$$

3.3. Attempts to Improve Data Quality

Other measures have been attempted to improve the data quality, by moving average smoothing and Gaussian smoothing. Average smoothing is able to filter away small noises in the data which improves the reliability of the results. Gaussian smoothing involves modification of data by convolution with a Gaussian function, removing some minor characteristics of the data. An example of a Gaussian smoothed curve is shown Figure 3.3.1. If Gaussian smoothing is applied, the shear modulus and shear strain will be evaluated using the by-eye method. To a certain extent, this proposed method changes the shape of the output, but it does not simplify the evaluation as much as the sinusoidal fitting method. It affects the accuracy of the results produced by the by-eye method and also increases the computational time required to analyse the data. Therefore, Gaussian smoothing has not been applied in the analysis.

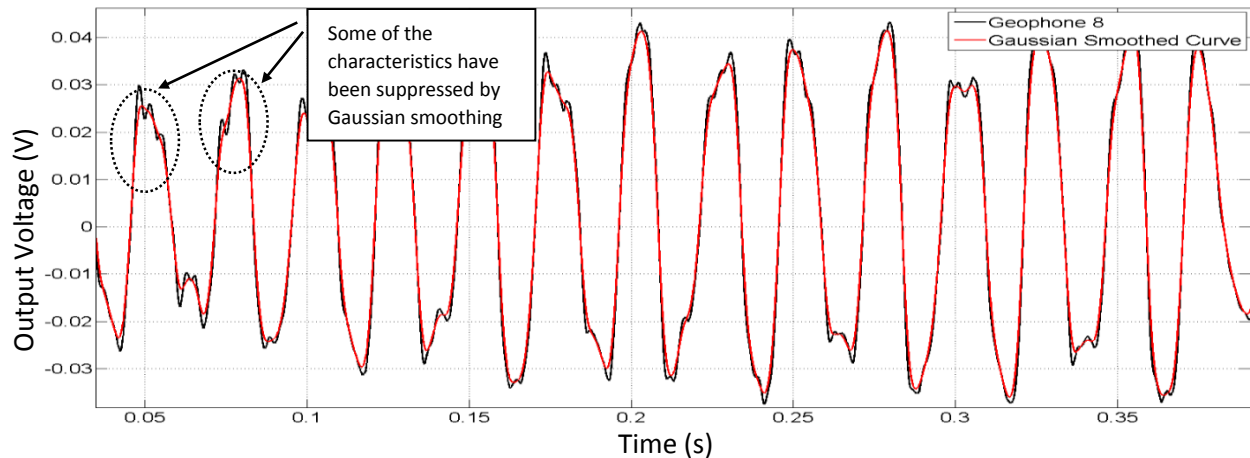


Figure 3.3.1 – Gaussian smoothed curve (53.4kN (12kips) static load and 40Hz dynamic load at the Fitzpatrick Ranch Site)

4. Results

4.1. Steady-State Dynamic Tests

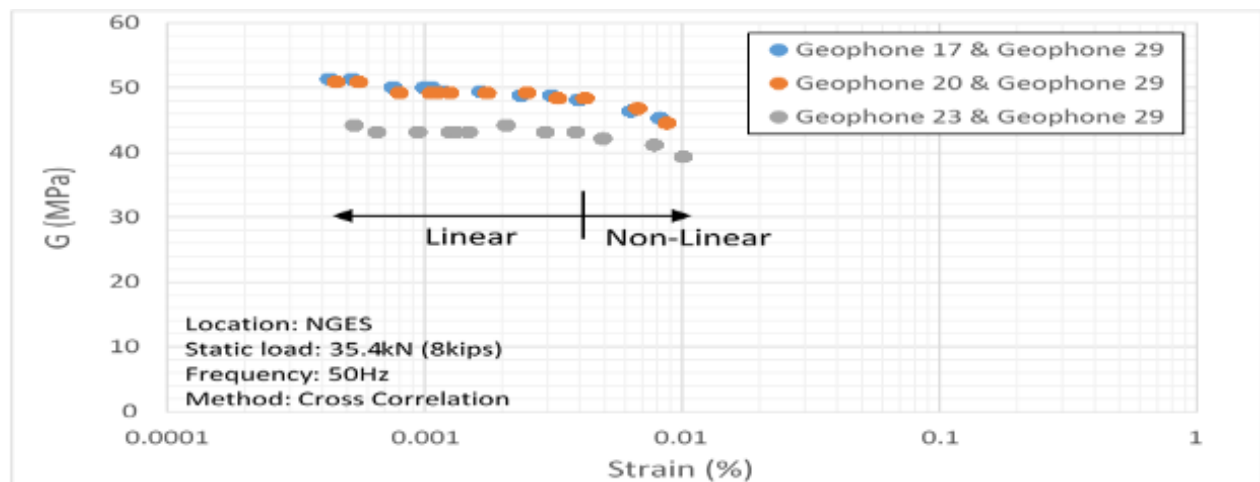
The steady-state dynamic test results for both sites show that the general pattern depends on the static load, the frequency of dynamic loading and the depths of geophones being compared. In each borehole, outputs from all the geophones in the same borehole, except the one nearest to the footing, have been compared with that from the geophone installed in the concrete footing. The output from the geophone installed in the concrete footing measures directly the vibration of the footing and is therefore subject to minimal distortion. It is ideal to be used for comparison with other geophone outputs. The geophone closest to the footing has not been analysed as the distance is too short to measure the time lag without having a large percentage error. The results produced by the sinusoidal fitting curve test generally have an unusually large scatter with no clear pattern to be seen. Therefore, they are not presented here.

4.1.1. NGES

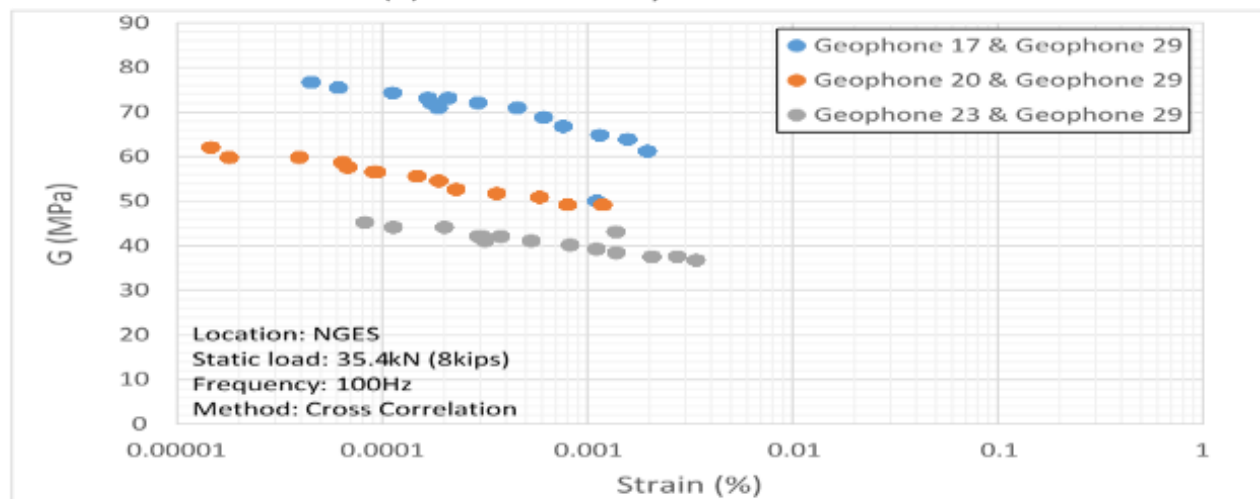
Results produced by cross correlation and by the by-eye method are presented for the middle borehole in Figures 4.1.1 and 4.1.2 respectively for comparison. The by-eye method has produced results with a larger scatter, while cross correlation has produced much neater results. Results produced by cross correlation for other boreholes are presented in Figure A.1 in Appendix A.

The results presented in Figure 4.1.1 and 4.1.2 show that although the by-eye method manages to produce results that show a decreasing trend of shear modulus as the strain increases, the

quality is not high enough to observe much similarity with the theoretical shear modulus-shear strain relationship as shown in Figure 1.1.2. On the other hand, a clear linear region and a small part of the non-linear region can be observed in the results from 50Hz tests produced by cross correlation. However, the linear and non-linear regions are more difficult to distinguish in the results from 100Hz tests produced by cross correlation. The possible explanations for this issue will be clarified in Section 6. Figure 4.1.3 shows the normalised results from 50Hz tests compared with the normalised shear modulus curve proposed in Seed & Idriss (1970). The results just lie outside the upper bound of the curve. It is likely to be due to difference methods adopted to estimate the shear strain or different soil types tested.

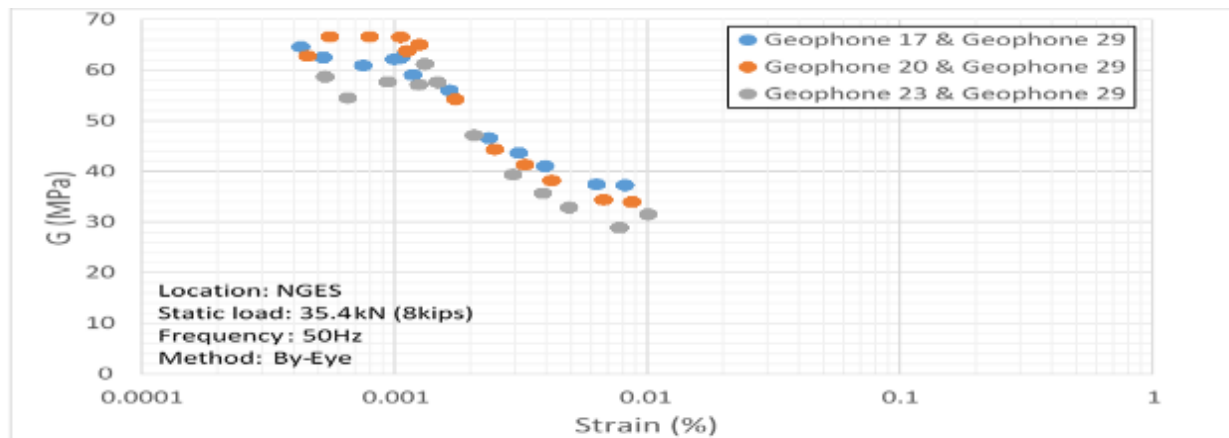


(a) Under 50Hz dynamic excitation

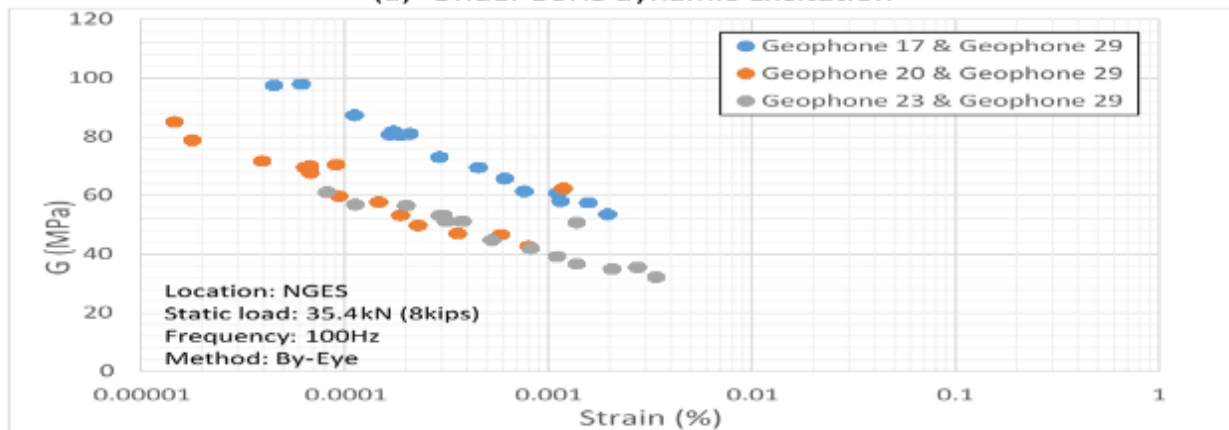


(b) Under 100Hz dynamic excitation

Figure 4.1.1 – Results for the steady-state dynamic tests produced by cross correlation (35.6kN (8kips) static load at the NGES



(a) Under 50Hz dynamic excitation



(b) Under 100Hz dynamic excitation

Figure 4.1.2 – Results for steady-state dynamic tests produced by the by-eye method (35.6kN (8kips) static load at the NGES)

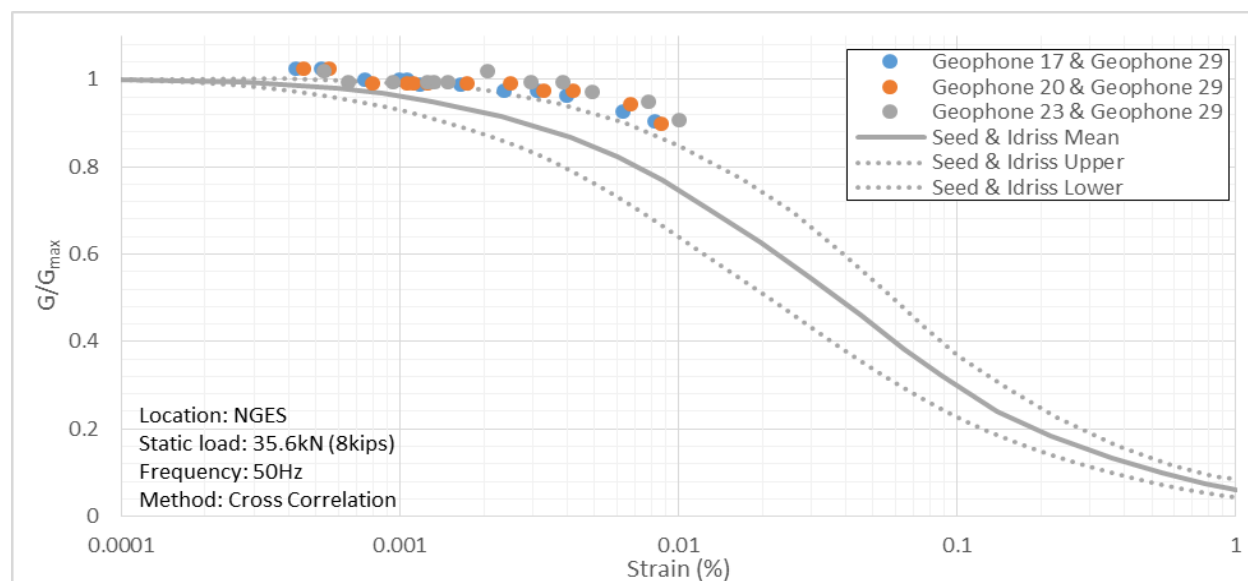


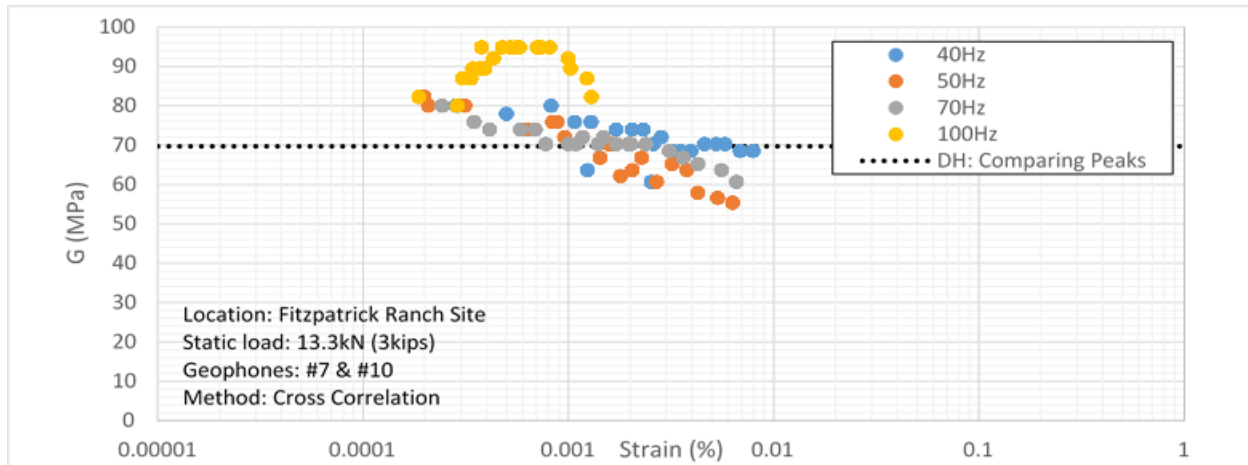
Figure 4.1.3 – Normalised results compared to Seed & Idriss curve (35.6kN (8kips) static load at the NGES)

4.1.2. Fitzpatrick Ranch Site

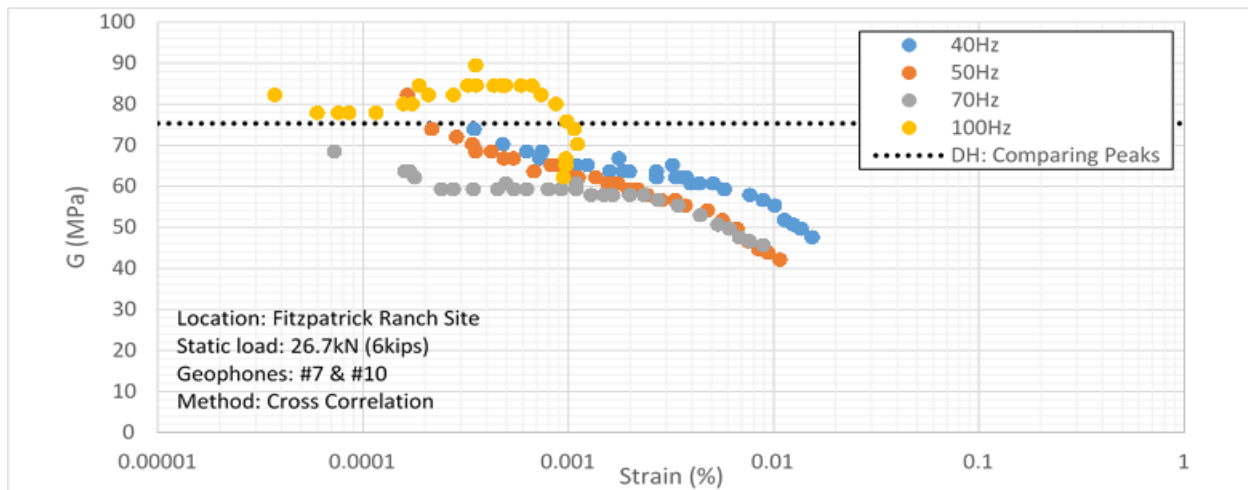
Only results between the second lowest geophone (Geophone 7) and the geophone that was installed in the footing (Geophone 10) are presented in Figure 4.1.4 as the general characteristics of the results for other geophones in the same borehole are essentially similar. The results appear to have a dependency on the static load and the frequency of the dynamic load applied. Results between Geophone 2 and Geophone 5 in the west borehole are presents in Figure A.2 in Appendix A.

The results produced by the sinusoidal curve fitting method or the by-eye method are not presented here as they are too coarse to show any observable trends. The problems with both methods and recommendations for improvement will be further discussed in Section 6. The results produced by cross correlation are presented in Figure 4.1.4. The tests carried out at 13.3kN (3kips) static load generally give coarser results. Results under 53.4kN (12kips) do not show much of the predicted linear response at small strain levels, but shows a decreasing trend of shear modulus as the shear strain increases. Out of the three levels of static loads, the tests under 26.7kN (6kips) have produced results that agree the most with the predicted characteristics. The possible explanations on why the characteristics of the results are affected by static loads are discussed in Section 6. The results under 100Hz dynamic load differ from the rest of the results. It is possible that the geophone signal is disturbed by other mechanisms in the soil, causing this abnormal characteristics of results. The assumption to take the shear strain from the relative displacement over the steady-state cycles might not have applied well on the 100Hz data. This could be deduced from the “hook” shape in the curve for 100Hz tests in Figure 4.1.4.

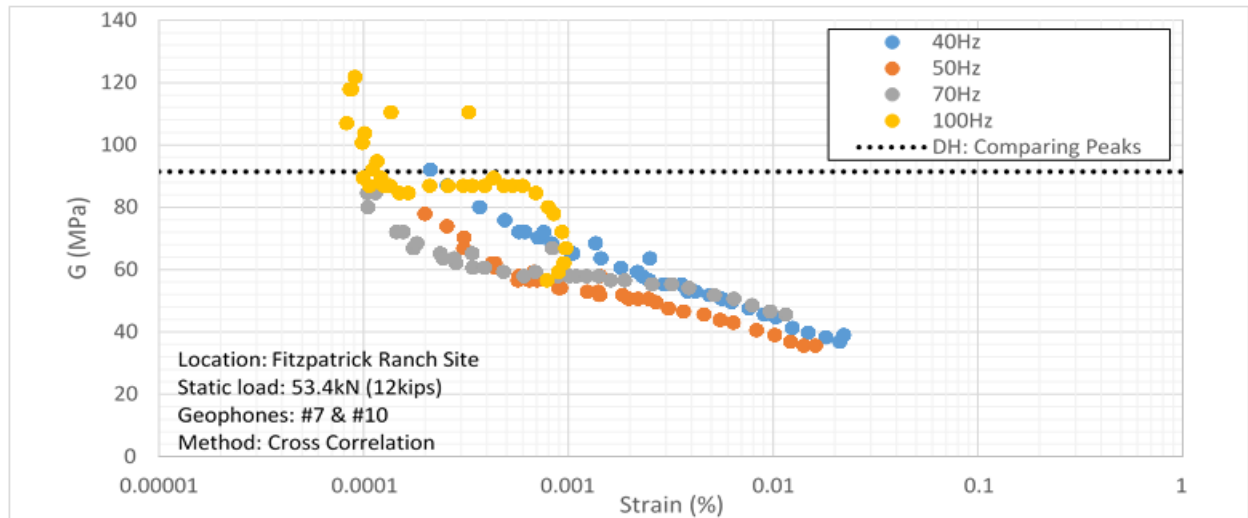
The normalised shear modulus under 26.7kN (6kips) for both boreholes are presented in Figure 4.1.5. The results from 100Hz tests are not included. The figure shows that there is a greater scatter in the linear region. But in the non-linear region, the results mostly agree with the Seed & Idriss curve and lie within the upper and lower bounds.



(a) Under 13.3kN (3 kips) static load



(b) Under 26.7kN (6 kips) static load



(c) Under 53.4kN (12 kips) static load

Figure 4.1.4 Results for steady state dynamic tests produced by cross correlation (Static Load: (a) -13.3kN (3kips); (b) - 26.7kN (6kips); (c) - 53.4kN (12kips); at the Fitzpatrick Ranch Site)

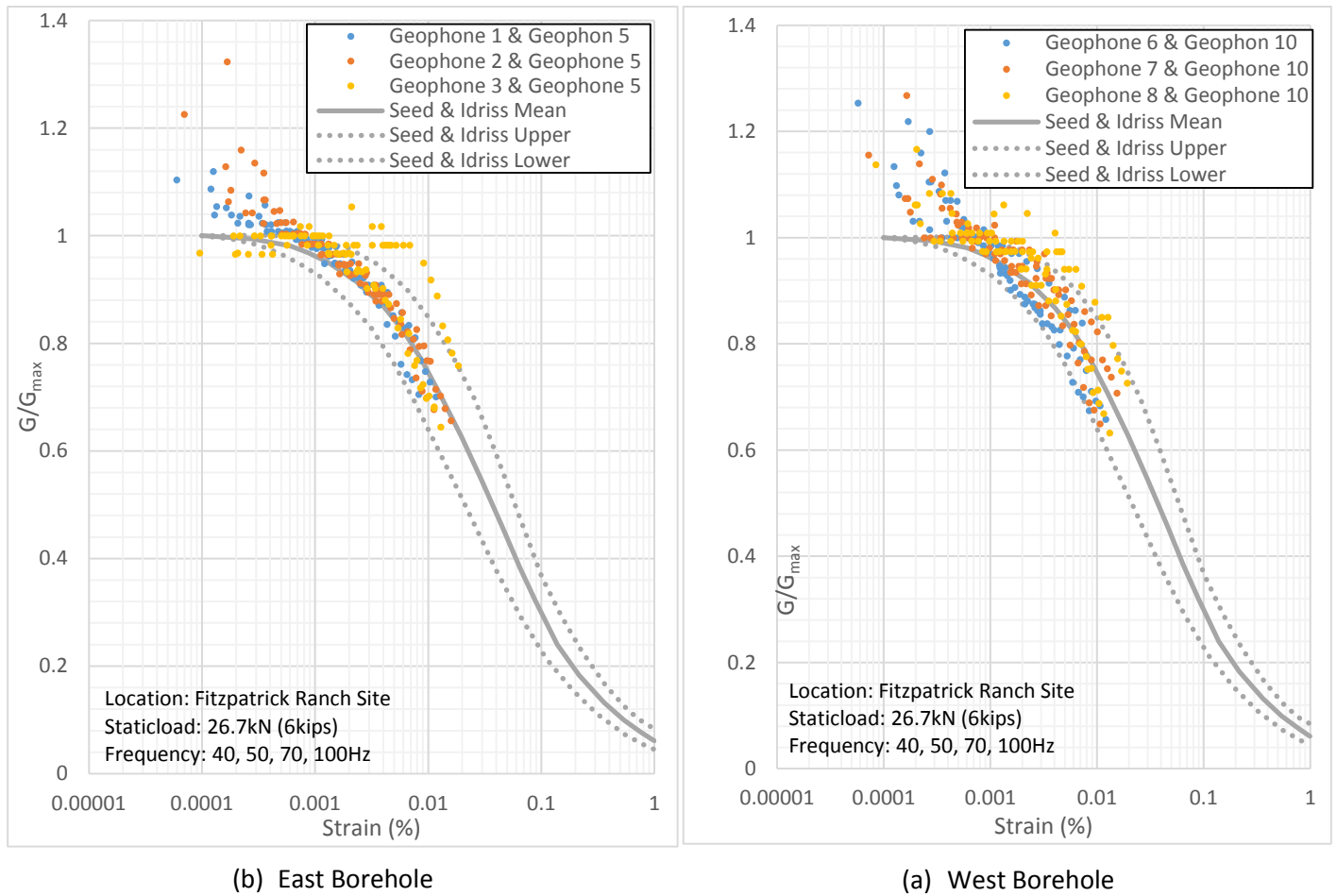


Figure 4.1.5 Normalised results compared to Seed & Idriss curve (26.7kN (6kips) static load at the Fitzpatrick Ranch Site)

4.2. Downhole Tests

4.2.1. NGES

The linear shear modulus estimated from the downhole tests is consistently larger than the shear modulus calculated from the steady-state dynamic tests. This could be explained by error in the tests or in the analysis or by the possibility that the soil experienced pressure waves from rocking in the steady-state dynamic tests. This will be further discussed in Section 6. Figure 4.2.1 shows the shear wave velocity profiles (V_s) at the east borehole calculated from the downhole tests. The shear wave velocity profile under each static load is estimated by taking the average of the shear wave velocities measured from all the tests carried out under the static load. The profiles show that the soil stiffness is not uniform, probably due to soil inhomogeneity and non-uniform soil

density. The soil is stiffer at around 0.2m depth and becomes less stiff at around 0.3m depth. The shear wave velocity generally increases with static load, but there are exceptions. This could be attributed to measurement resolution, as a considerable scatter is observed in the shear wave velocities measured under the same static load. The shear wave velocity profiles at other boreholes are presented in Figure B.1 in Appendix B.

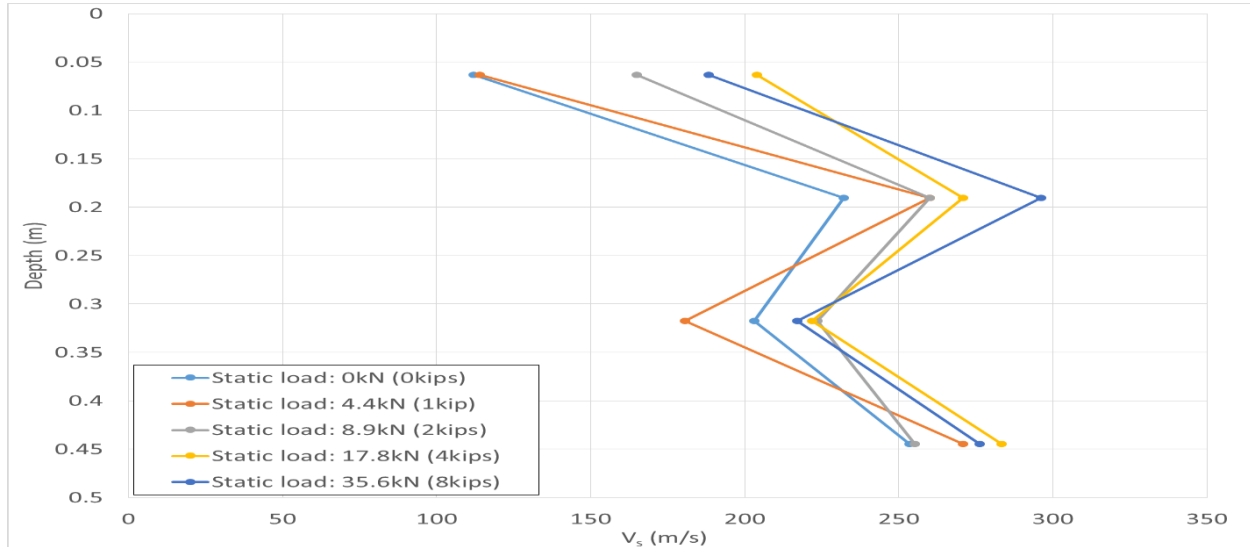


Figure 4.2.1 – Shear wave velocity, V_s , profiles for east borehole from downhole tests at the NGES

Taking $\log_{10}$ of both sides and substituting G with ρV_s^2 in Equation 1.1 indicate that the value for the coefficient of $\log \sigma_0$ should be 0.25 in theory. In Figure 4.2.2, the results for east borehole show that the shear wave velocity increases with confining pressure. The coefficient of $\log \sigma_0$ is closest to the theoretical value at shallow average depths. The lines for other depths have smaller gradients, which means that the shear wave velocity is less sensitive to confining pressure. The reduction in sensitivity to confining pressure can be attributed to factors such as presence of cementation or water menisci. However, presence of cementation is unlikely at the NGES. Presence of water menisci in soil is possibly the cause and has the same effect as cementation, which adhere the soil particles together through surface tension. From the west borehole results, it is shown that at average depths of 0.191m and 0.445m, the shear wave velocity even decreases with confining pressure. The anomaly is likely to be caused by analysis inaccuracies. Nevertheless, it shows that at those depths in the west borehole, the soil is relatively insensitive to confining pressure.

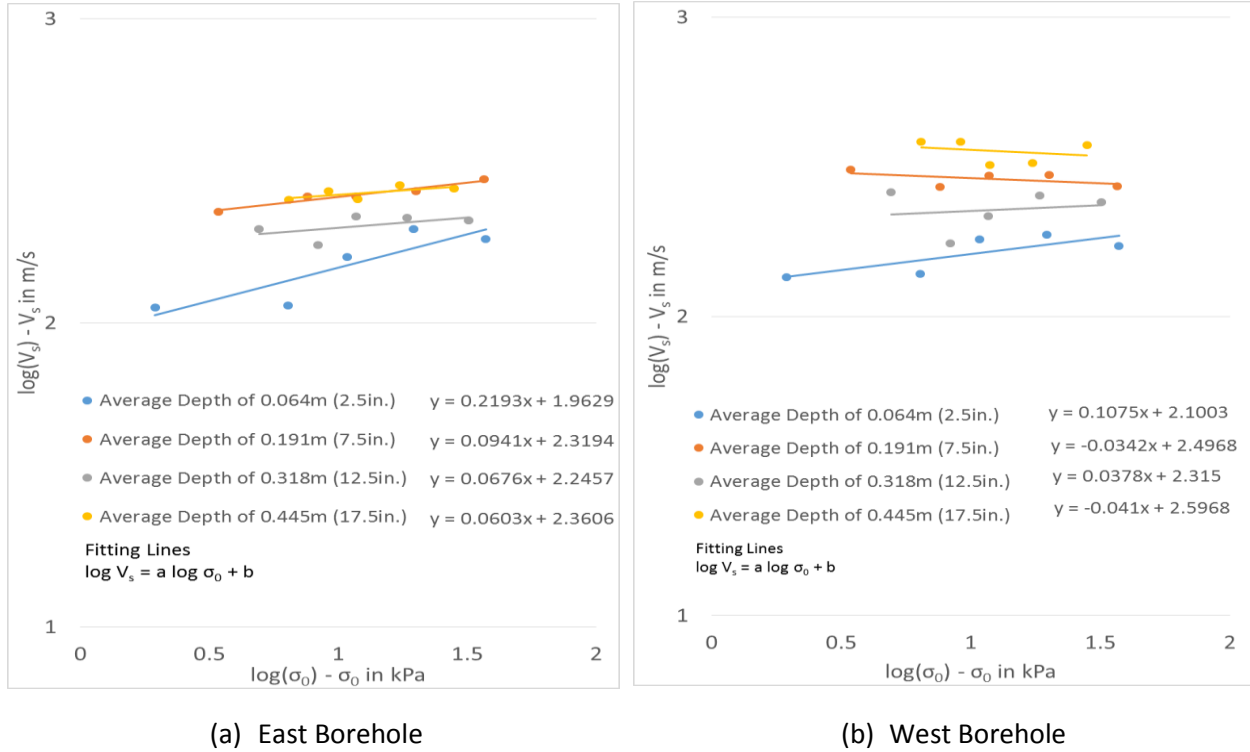


Figure 4.2.2 – $\log V_s$ against $\log \sigma_0$ plots from downhole tests at the NGES

4.2.2. Fitzpatrick Ranch Site

Figure 4.2.3 shows the shear wave velocity profiles at the east borehole from downhole tests carried out at the Fitzpatrick Ranch site. The profiles are coincidentally similar to those for the NGES, which also indicate a non-uniform stiffness in the soil. The soil is stiffer at around 0.15m and 0.6m depths but is less stiff near the footing and at around 0.25m depth. The profiles after experiencing large strain (dotted lines) have some minor differences from the profiles before experiencing large strain, which can be attributed to measurement resolution. Hence, it can be assumed that the steady state dynamic tests did not cause too much damage to the soil structure. The shear wave velocity profile at the west borehole is presented in Figure B.2 in Appendix B.

Figure 4.2.4 shows the variation of $\log V_s$ with $\log \sigma_0$ evaluated from in-situ tests and laboratory tests (red dashed line). The results show that the shear wave velocity increases with confining pressure, but the influence of confining pressure depends on the amount of cementation and disturbance the soil experienced. The line for laboratory results represents the variation of shear wave velocity of a disturbed soil sample. On both boreholes, the lines for shear wave velocity at 0.051m depth have gradients close to that of the line from laboratory testing. This shows that

some disturbance likely occurred to the soil near the footing. For other depths, the lines are less steep, indicating that possible cementation in the soil below 0.051m depth was present.

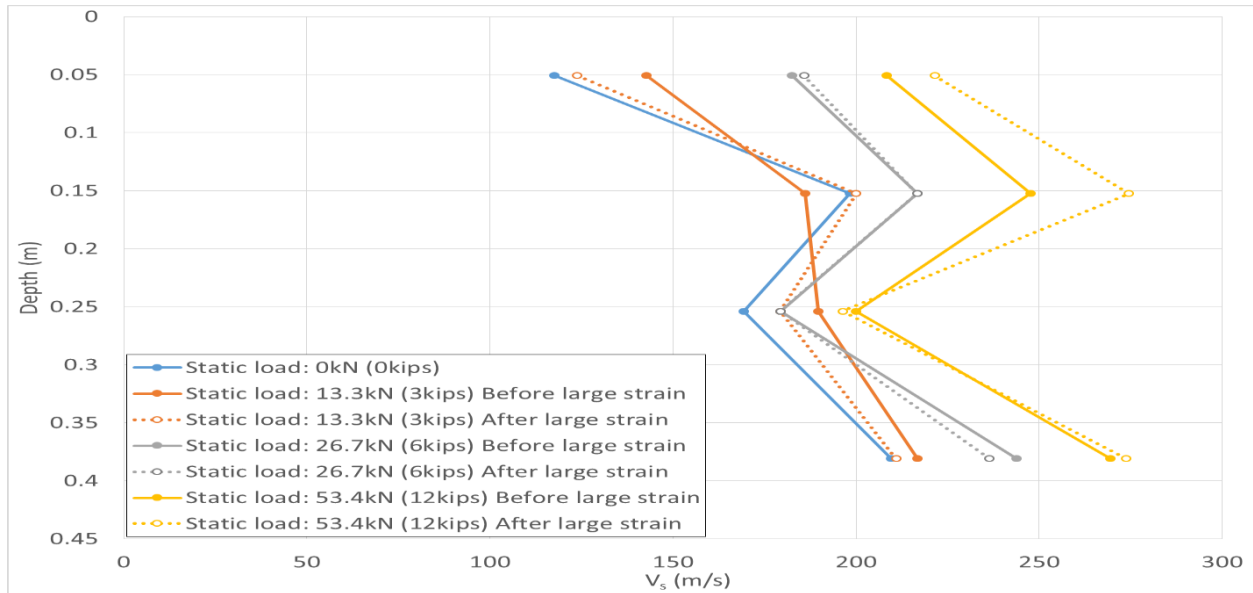


Figure 4.2.3 – Shear wave velocity, V_s , profiles for east borehole from downhole tests at the Fitzpatrick Ranch Site

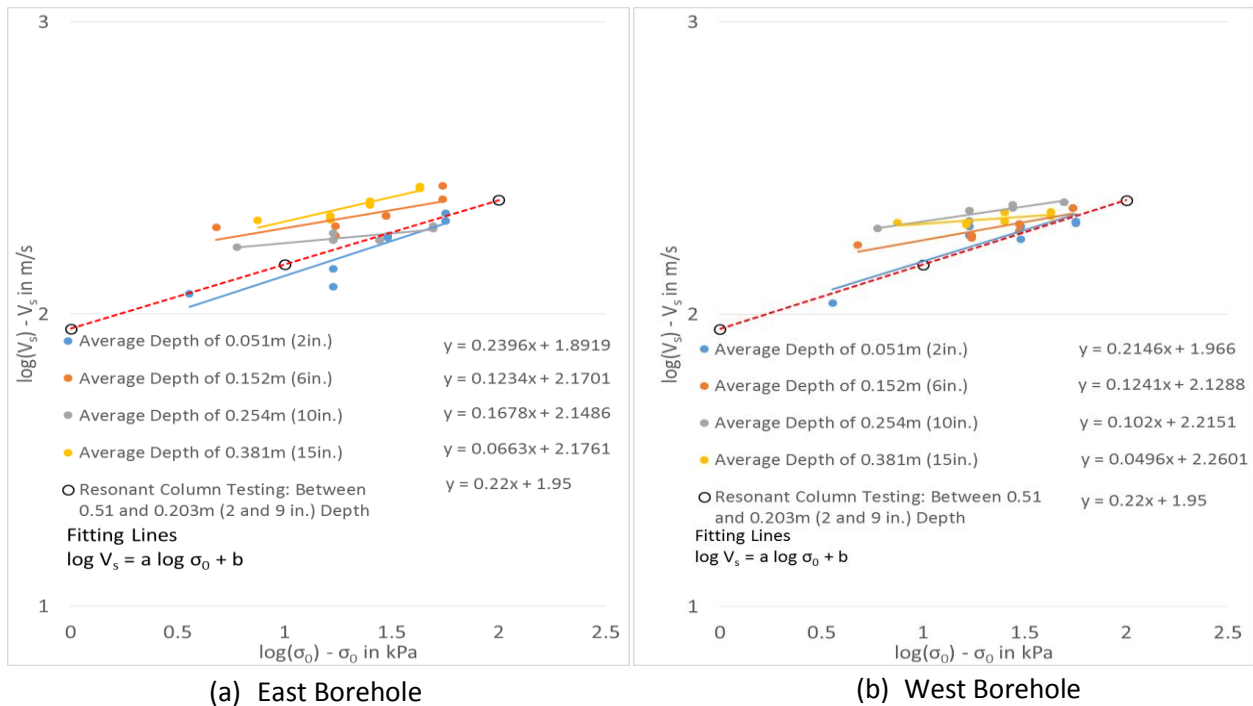


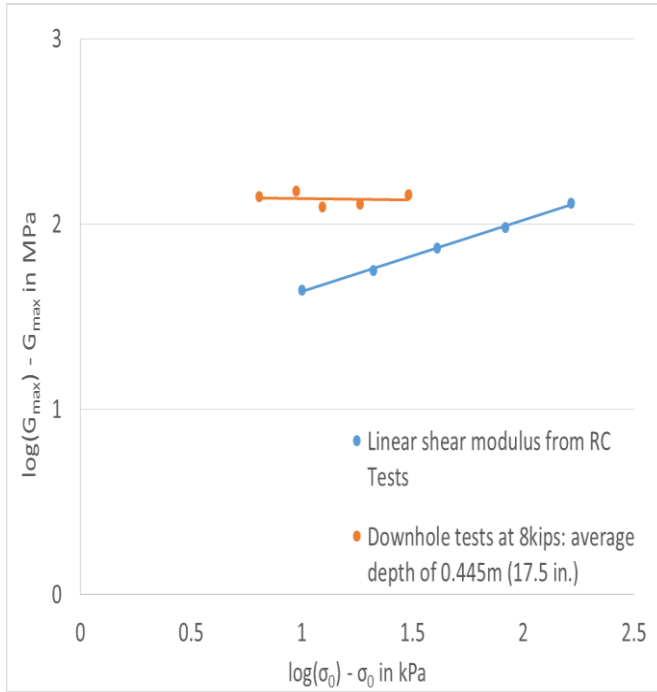
Figure 4.2.4 – $\log V_s$ against $\log \sigma_0$ plots from downhole tests at the Fitzpatrick Ranch Site

4.3. Comparison with Laboratory Results (NGES only)

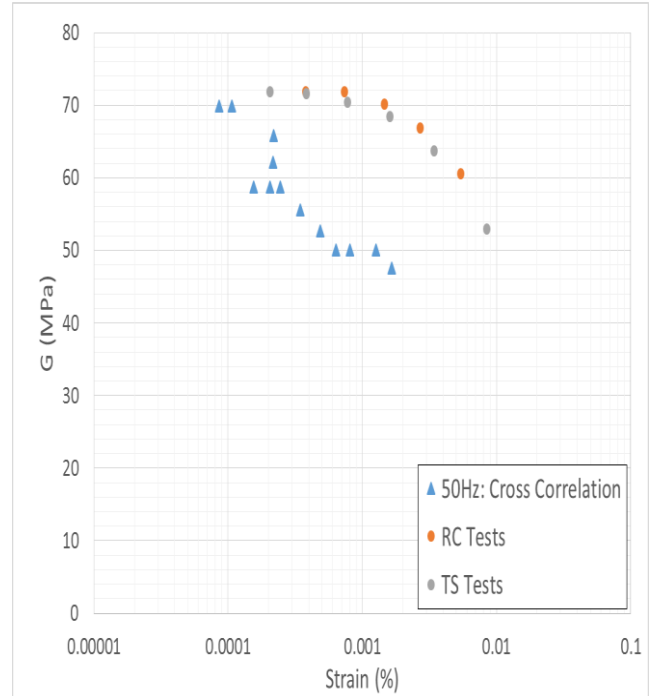
The comparison between field and laboratory test results is presented in Figure 4.3.1. The specimen was collected at a depth of 0.457m (18 in.). For the field test results, outputs from geophone 17 and geophone 20 are analysed as the average depth between the geophones (0.445m or 17.5in.) is the closest to the sampled depth. Resonant column (RC) tests and torsional shear (TS) tests were carried out for laboratory testing.

In Figure 4.3.1 (a), it is shown that the shear wave velocity measured in in-situ tests is higher than that measured in laboratory tests, indicating that the in-situ soil is stiffer. The line for in-situ test results is also less steep than that for laboratory test results, indicating the presence of water menisci. The process of sampling led to evaporation of soil moisture, causing a lower stiffness and a higher sensitivity to confining pressure.

Figure 4.3.1 (b) shows that the results produced by cross correlation method are too coarse to show much indicative information about the soil. The geophones might be too close to measure the shear modulus and shear strain accurately. As the geophones are the two lowest in the middle borehole, their outputs suffer from a larger distortion. The poor quality of the results is caused by these influencing factors together with other problems such as rocking of footing and soil inhomogeneity. The isotropic confining pressure created in the laboratory tests is 41kPa and the estimated in-situ confining pressure at 0.445m depth is 33.3kPa. The linear shear modulus measured from field tests is between 70MPa to 80MPa which appears to agree with the line for laboratory tests in Figure 4.3.1 (a) and is lower than the stiffness predicted in the downhole tests, indicating that the soil structure might have experienced some damage. Besides, the shear moduli measured from the field tests show non-linear response at a smaller strain than the laboratory tests. The discrepancy is probably due to different ways to estimate the characteristic shear strain. Further investigation is needed to verify that taking the average shear strain as the characteristic shear strain is valid for most of the cases.



(a) Comparison with downhole test results



(b) Comparison with steady-state dynamic test results

Figure 4.3.1 – Comparison of downhole and steady state dynamic test results with laboratory results (35.6kN (8kips) static load at the NGES)

5. Discussion

5.1. Steady State Dynamic Tests

A small portion of the results obtained have shown recognisable characteristics that agree with the prediction. However, the overall results fail to show an adequate consistency in quality. While it is proven that in-situ testing is in fact feasible to evaluate the shear modulus-shear strain relationship, improvements have to be made in terms of analysis method and testing techniques to achieve a better consistency. Both the sinusoidal curve fitting method and the by-eye method fail to produce results with sufficient resolution. The sinusoidal curve fitting method is fundamentally not suitable when the data are too distorted from a sinusoidal curve. On the other hand, the by-eye method can be improved to improve its robustness to data quality. So far, the cross correlation method is able to produce results with the most consistent quality, but verification is necessary to show that the method is able to produce accurate results and is robust to poor quality data. Also, the cross correlation method is subject to a larger error when the two signals being compared are too different from each other.

The magnitude of static load applied has a direct relationship with the characteristics of the results. As shown in Section 4.1, for Fitzpatrick Ranch Site, the results under 26.7kN (6kips) static load have the best match with the predicted characteristics. The results under 13.3kN (3kips) are generally scattered and the results under 53.4kN (12kips) show an unusual rise in shear modulus at very small strains. Under a small static load, the concrete footing may have less effective bonding with the soil. In addition, the soil may not be compact enough to ensure a good transmission of shear wave velocity. As a result, the signals picked up by the geophones may become more distorted, reducing the resolution of results. On the other hand, a few possible physical interpretations have been suggested to explain the unusual feature of the results under a high static load.

Besides, the results obtained from both sites have shown a dependency on the dynamic load frequency. In Section 4.1, it has been shown that the results analysed from the 100Hz tests for the NGES do not show clearly the linear and non-linear regions of shear modulus. This could be clarified if a larger range of shear strain was induced during the testing to obtain more of the non-linear characteristics. Therefore, it is recommended that a larger range of dynamic load could be applied in future testing to obtain more complete results of shear modulus-shear strain relationship. On the other hand, the 100Hz test results for the Fitzpatrick Ranch site appear to have distinct characteristics compared to the results for lower frequency tests. Other mechanisms such as rocking, or vertical vibrations might have played a role in this phenomenon. For these mechanisms, the frequency of excitation tends to be a more influential factor governing the response of the footing and the soil compared to just simply shearing. These mechanisms can be identified from the outputs of vertical geophones which detect any pressure waves in the soil. However, vertical geophones were not installed at the Fitzpatrick Ranch Site.

5.1.1. Problems with Analysis Methods

5.1.1.1. Sinusoidal Curve Fitting Method

As shown in Figure 3.1.10, the sinusoidal curve fitting method does not work well when the geophone output is heavily distorted from a sinusoidal curve. The shear modulus and shear strain measured is not representative of the actual values. Even if the geophone output is somewhat

approximately a sinusoidal curve, the shear modulus measured by this method might still have a significant percentage error, since the time lag between a geophone outputs is a very small value.

An illustration of how a fitting curve that seemingly fits well with the geophone output can still lead to a large error in the measurement of time lag is presented in Figure 5.1.1. The time lag measured by cross correlation in this example is 0.0016s. The difference in the locations of peaks between the geophone output and the fitting curve shown in the figure has an order of 0.001s. Hence, even though the geophone output might not be distorted by a large extent, there is still a large percentage error in the measurement of time lag using the sinusoidal curve fitting method.

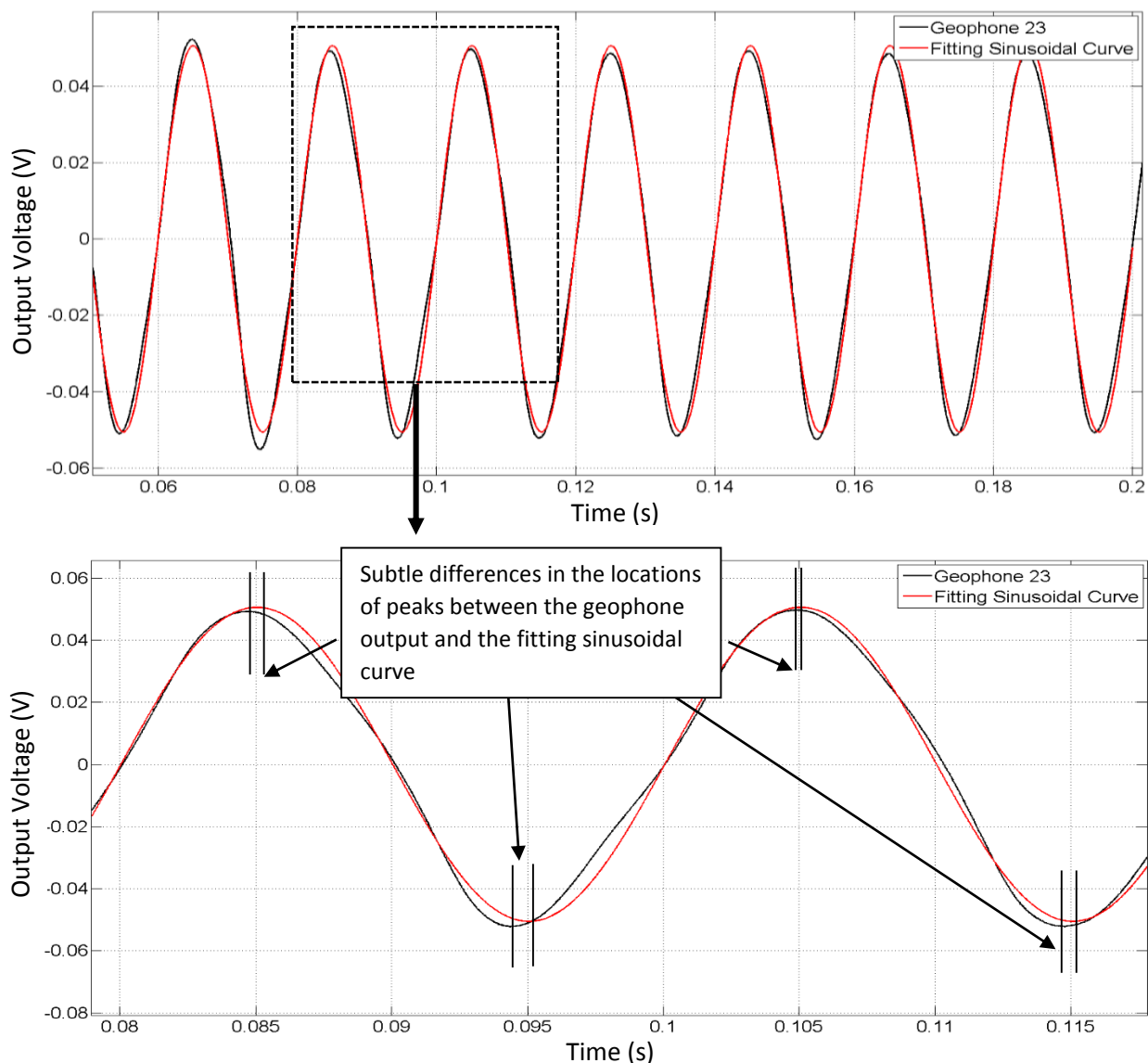


Figure 5.1.1 – Illustration of measurement error of time lag using the sinusoidal curve fitting method (35.6kN (8kips) static load and 50Hz dynamic load at the NGES)

5.1.1.2. By-Eye Method

This method has the advantage of analysing the geophone output directly without changing the nature of the output. However, each cycle in the output is likely to look different from other cycles. The geophone outputs might have some characteristics different from each other. Therefore, the time lag and relative displacement measured at each cycle will be different, as demonstrated in Figure 4.1.11. The spread will be governed by the quality of data and the degree of distortion. Subsequently, this influences the consistency of the analysis. A possible way to improve the consistency of the method is to pick a specific feature to measure the time lag and relative displacement, similar to the way peak D is chosen to measure the time lag in the downhole test data. However, the challenge would be choosing a specific characteristic, which is representative of the actual behaviour of the soil, to compare.

More time will be required to study a large amount of geophone outputs to make an appropriate choice. Multiple characteristics can be chosen to verify that the results are not heavily affected by the choice of characteristic being compared.

Figure 5.1.2 illustrates the possible error in measuring the time lag using the by-eye method. In addition to error related to distortion of geophone output and the averaging of values, possible error might arise from false identification of peak location. The figures show that the zoomed-in view of a peak is very irregular, making it difficult to identify the exact peak location. Smoothing the curve removes the small fluctuations but also changes the shape of the output. In this example, the time lag between the outputs of Geophone 1 and Geophone 5 measured by cross correlation is 0.0027s. The error in locating the peak has an order of 0.0001s, which is 10 times smaller than the sinusoidal curve fitting method.

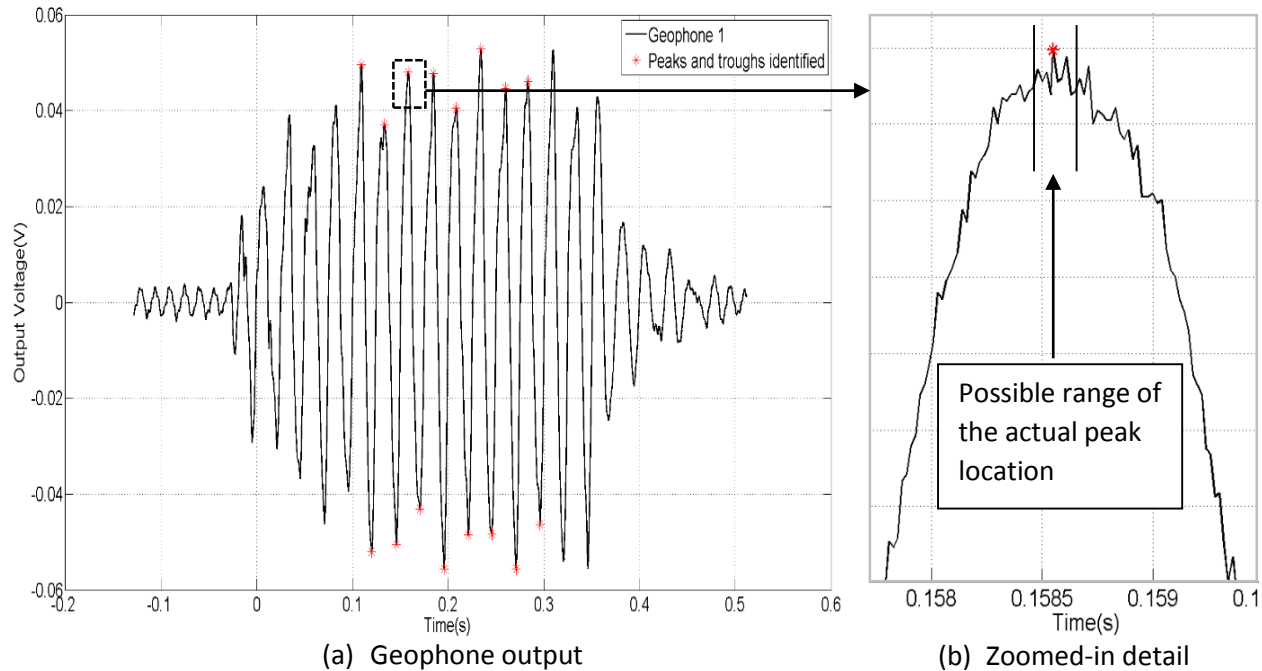


Figure 5.1.2 – Illustration of measurement error of time lag using the sinusoidal curve fitting method (53.4kN (12kips) static load and 40Hz dynamic load at the Fitzpatrick Ranch Site)

5.1.1.3. Cross correlation

As discussed before, the accuracy of cross correlation depends on the similarity of the geophone outputs being compared. The variation in the shape of geophone outputs arises from factors such as soil inhomogeneity, pressure waves, incorrect location or orientation of geophone, etc.

Figure 5.1.3 shows two geophone outputs that lack an adequate resemblance for cross correlation produce an accurate result. The output from Geophone 5 have a much softer texture than Geophone 1. Geophone 5 is installed within the footing while Geophone 1 is the lowest geophone installed in the soil. Therefore, it is expected that the shape of Geophone 5 output is more similar to the shape of the dynamic load input. In this example, the time lag measured by cross correlation is 0.0018s. But the time lag measured by the by-eye method ranges between 0.0018s and 0.0028s.

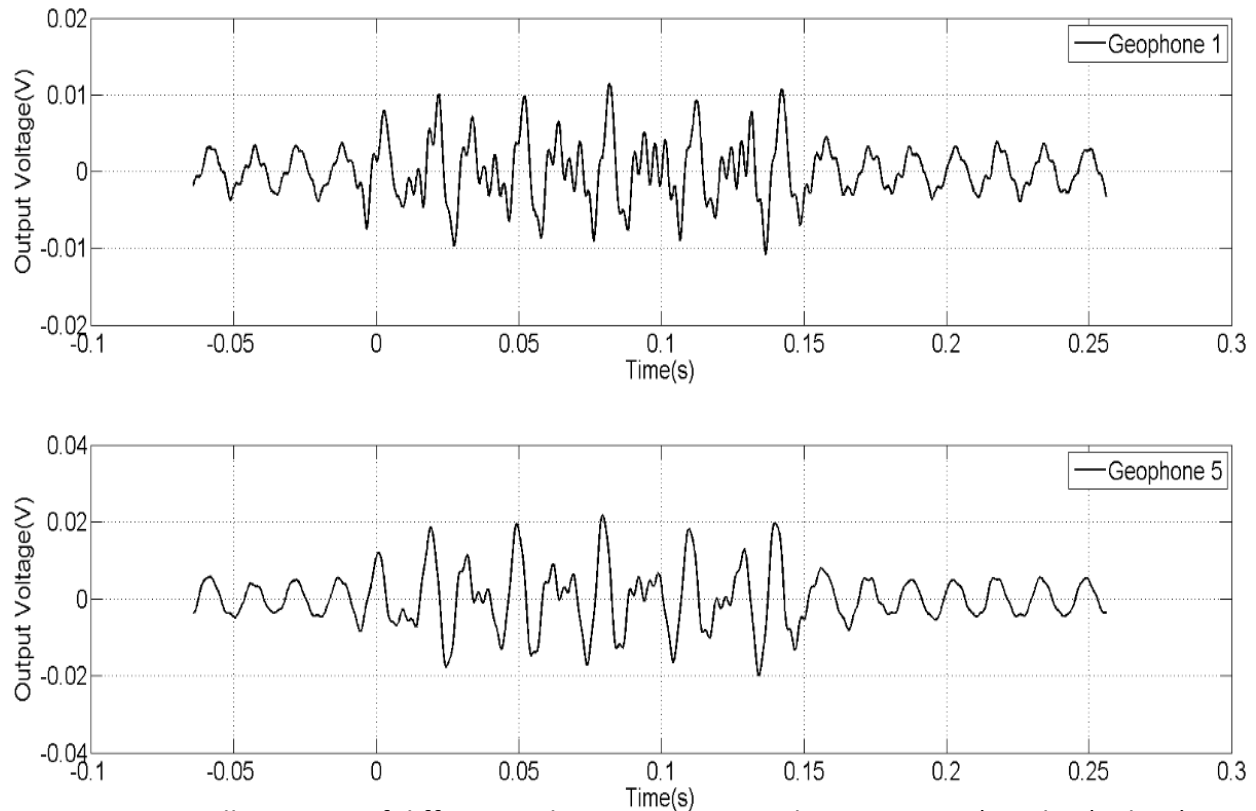


Figure 5.1.3 – Illustration of differences between two geophone outputs (53.4kN (12kips) static load and 100Hz dynamic load at the Fitzpatrick Ranch Site)

5.1.2. Influence of Static Load on Steady-State Dynamic Test Results

It has been shown in the results for the Fitzpatrick Ranch Site that different levels of static loads lead to different characteristics in the results. It has been suggested that for a low static load, the results become coarser due to the poor contact between footing and soil and less compact soil that results in poor wave transmission. The footing is also more prone to rocking.

It has been observed that at small magnitudes of dynamic loads, the outputs from the geophones in the soil appear to be more disturbed. Besides, the time lag measured to calculate the linear shear modulus is very small and thus likely to suffer from a larger percentage error. Nevertheless, it still appears that the shear moduli, which are supposed to be constant in the linear region, are larger at very small shear strain level (see figures in Section 4.1). If analysis error is the only cause, the results should be scattered randomly without bias. Therefore, it is deduced that the soil experienced certain phenomenon to exhibit such characteristics. Two physical explanations have been proposed:

- a) Laboratory testing has shown that cementation accounts for a higher linear shear modulus and a more rapid decrease of shear modulus in the non-linear region. The soil at the Fitzpatrick Ranch site is unevenly cemented. At very small shear strains, the cemented soil contributes to the stiffness, resulting in a higher stiffness measured from the tests. As the shear strain increases in the linear region, possibly some cementation has degraded and the uncemented soil becomes fully mobilised in cyclic shearing. More studies are needed to understand the behaviour of soil with an uneven cementation.
- b) At a high static load, the soil under the footing experiences a large confining pressure. The confining pressure dissipates with distance rather rapidly based on Boussinesq stress distribution. Due to the high differential of confining pressure in the soil, the stiffness of soil under footing is much higher than the soil further away. Then the situation will be such that, at a very small strain, the soil under the footing acts like a rigid body vibrating. The geophones installed within will move with the rigid body and hence the time lag between the outputs is very small. As the shear strain increases, the soil returns to shear in the expected way, giving the linear and non-linear shear modulus characteristics as predicted. More studies should be carried out to understand the behaviour of soil under a high static load.

These physical interpretations have no evidence or study to support them. They are merely hypotheses to interpret the results that have been produced. Further testing with improved techniques needs to be conducted to determine the exact reason behind the observed response. A wider variety of ground conditions should be investigated to understand the effect of cementation and soil inhomogeneity. Vertical geophones can be installed to detect any possible pressure waves and more geophones can be installed further away from the array of geophones to investigate how the shear wave spreads in the soil.

5.1.3. Influence of Dynamic Load Frequency on Steady-State Dynamic Test Results

The results for the Fitzpatrick Ranch Site are generally not as sensitive to the frequency of dynamic loading as the results for the NGES, particularly for the non-linear region. Some of the results for the NGES have a large discrepancy between 50Hz and 100Hz dynamic loading, as shown in Figure 4.1.1 as an example. Rocking of footing might be the cause for such discrepancy. Rocking generates pressure waves that travel diagonally across the geophone array (illustrated in Figure 5.1.4), inducing an extra component in the outputs of the horizontally orientated geophones. As pressure waves travel faster than shear waves in soil, the time lag between outputs measured by cross correlation may become an underestimate of the actual time that the shear wave takes to travel between the geophones, and therefore overestimating the shear modulus. Furthermore, the pressure waves cause strains in other directions. The assumption that the soil was only experiencing horizontal shear strain is no longer valid. A more detailed study is required to investigate the effect of strains induced by rocking on the shear modulus.

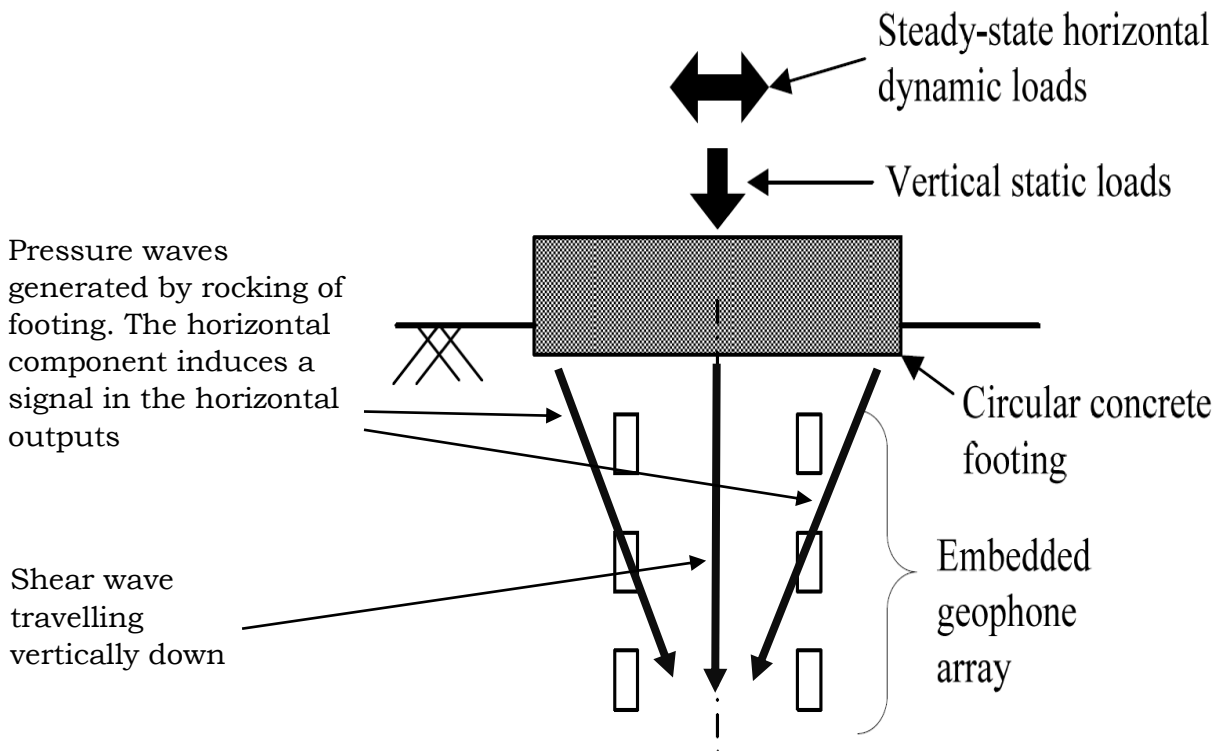


Figure 5.1.4 – Illustration of pressure waves and shear waves generated by a combination of horizontal shearing and rocking

Figure 5.1.5 shows the outputs of the vertical geophones in the east (Geophone 7), middle (Geophone 22) and west (Geophone 37) boreholes during one of the steady-state dynamic tests. The outputs at the east and west boreholes (red and black curves) are out of phase to each other. The outputs at the middle borehole (green curve) has a smaller amplitude than other boreholes. This indicates that some rocking occurred during the tests.

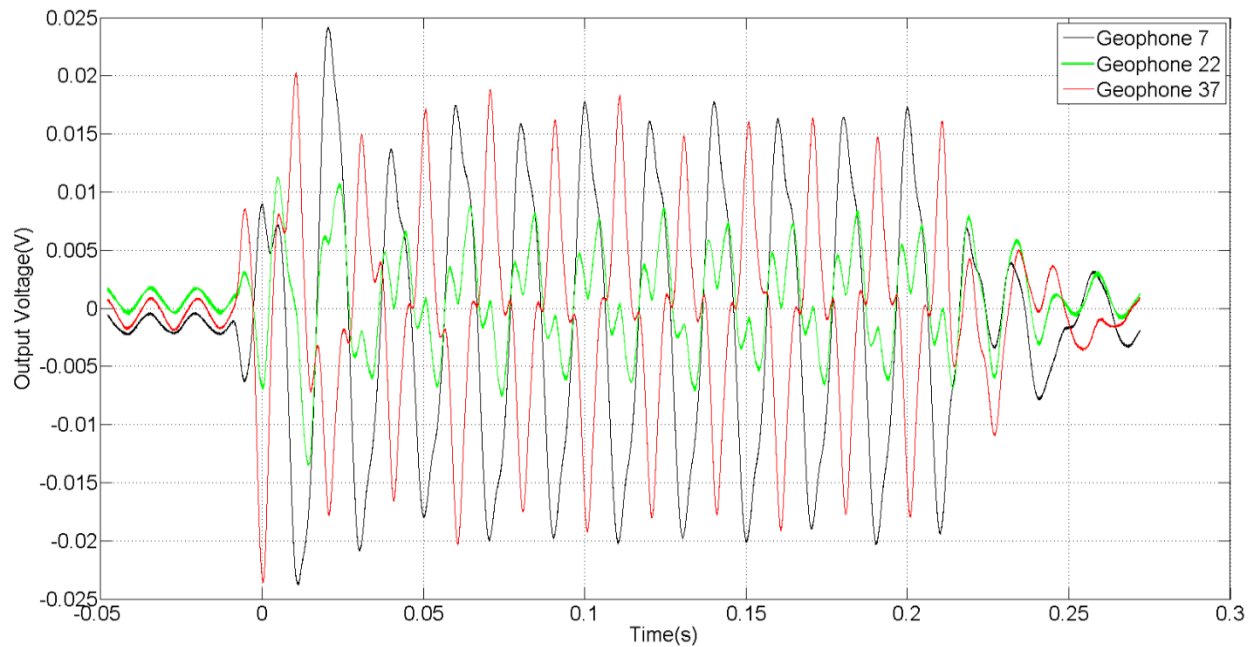


Figure 5.1.5 – Outputs of vertical geophones (35.6kN (8kips) static load and 50Hz dynamic load at the NGES)

The footing and soil have different responses to different dynamic mechanisms. Dynamic impedance functions for a rigid circular foundation on the surface of a homogeneous half-space were derived in Gazetas, 1983. It shows that the horizontal dynamic stiffness coefficients, k_h , can be considered as independent on frequency, while the vertical and rocking dynamic stiffness coefficients, k_v and k_r , depend on both the frequency of excitation and the poisson's ratio, ν . Figure 5.1.6 shows the impedance functions for horizontal shearing and rocking of a rigid circular footing on a homogeneous half-space. The soil at both sites comprises of silty sand which can be assumed to have a poisson's ratio of 1/3. The static stiffness is expressed as K . The dynamic stiffness coefficient, k , is expressed as a fraction of the static stiffness and a_0 is a frequency factor which is a function of frequency, footing size and characteristic shear wave velocity of the soil,

$\omega R/V_s$. The impedance function for rocking reflects that the magnitude of rocking depends on the frequency of excitation, which means that the magnitude of the pressure waves generated is also governed by the frequency. The magnitude of pressure waves subsequently determines its influence on the outputs of horizontal geophones and therefore affects the analysis at different levels. More studies are needed to investigate the influence of pressure waves from rocking on the cross correlation method and its dependency on frequency.

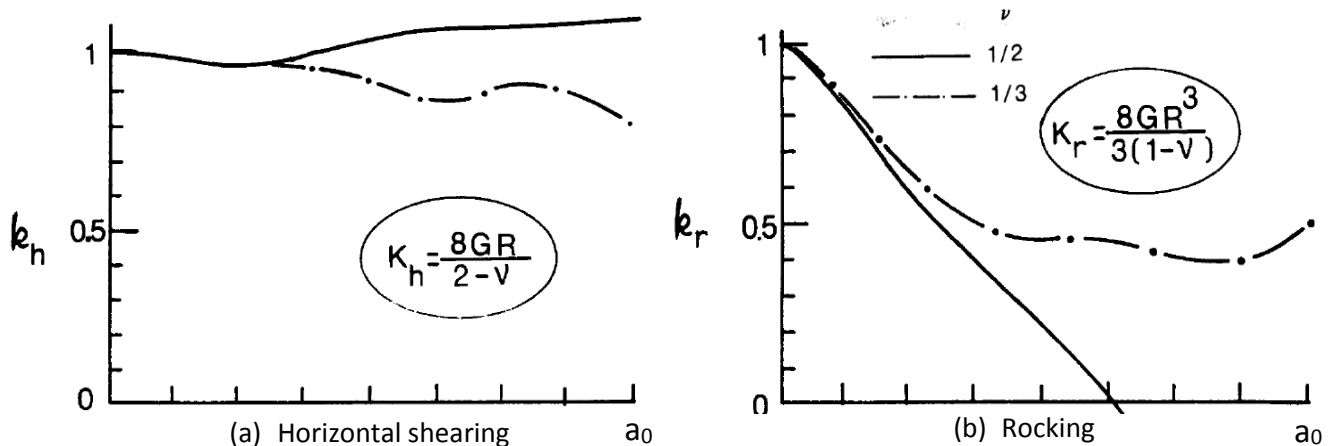


Figure 5.1.6 – Impedance functions for horizontal shearing and rocking of a rigid circular footing on a homogeneous half-space (Gazetas, 1983)

5.2. Small Strain Tests

Results from the small strain tests are used to provide information of the soil conditions below the footing and also clues of the reasons behind certain anomalies in the steady-state dynamic test results. For both locations, the downhole test results show that the linear stiffness within the geophone array is non-uniform. Many factors, such as soil inhomogeneity, disturbance from borehole drilling, backfilling of boreholes, contributed to this non-uniformity. Non-uniform soil stiffness can have an adverse effect on the quality of both the downhole and steady-state dynamic test data. The shear waves travel at different velocities depending on the local soil stiffness, and subsequently arrive at the geophones at different times. As a result, the geophone output may have multiple peaks in some cycles, as illustrated in Figure 5.2.1.

The analysis method for downhole test data is a sensible way to measure linear stiffness as the shear wave peak can be easily identified. To improve the reliability of the results, the small strain tests should be repeated for more times. In order to acquire more information about the soil conditions, the crosshole test data should be analysed too.

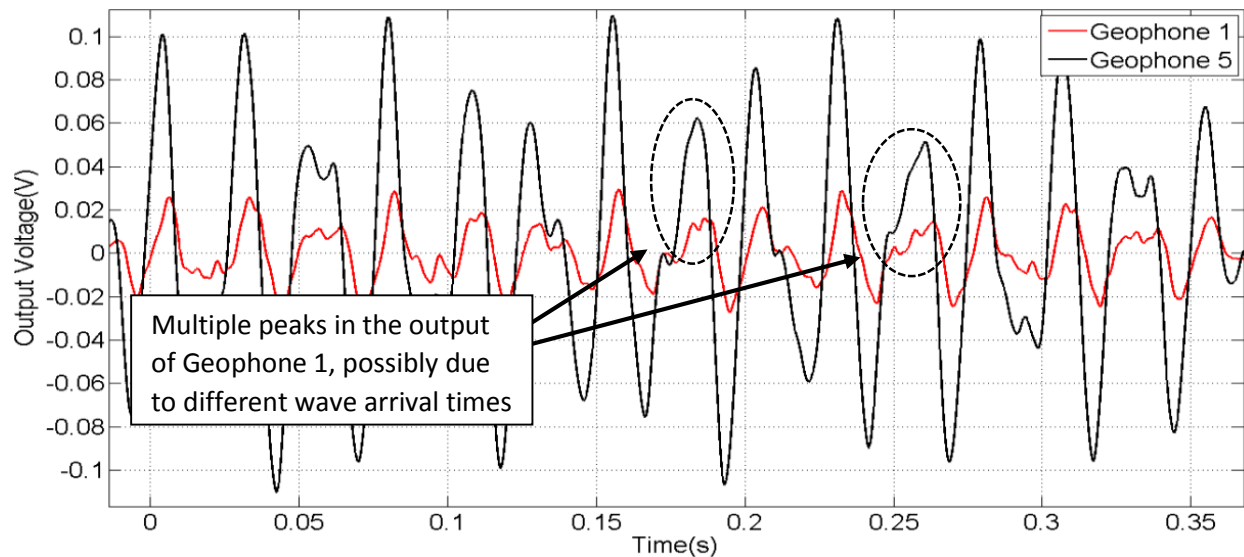


Figure 5.2.1 – Distortion in geophone output due to non-uniform soil stiffness

5.2.1. Discrepancy between Downhole Test and Steady-State Dynamic Test Results

There is a noticeable bias that the linear shear modulus measured from the downhole tests is higher than the value obtained from steady-state dynamic tests. As the time lag being measured is very small, the measurement of linear shear modulus is likely to be more scattered. While measurement resolution is one of the causes for such discrepancy, other factors should be involved to cause the systematic discrepancy.

In the steady-state dynamic test results, it is observed that, at very small strain levels, the shear modulus tends to be larger instead of remaining constant as predicted. As only very low levels of shear strain were induced in downhole tests, it is possible that the linear shear modulus measured agrees with this unusual characteristics of the linear region in the steady-state dynamic test results.

Presence of cementation or water menisci could have possibly played an important role for the discrepancy. In downhole tests, as very low energy impacts were transferred into the soil via the footing, the soil bonded by cementation or water menisci remained intact, permitting a high

shear wave velocity. During the steady-state dynamic tests, as the cyclic shear strain induced continuously increases in magnitude, some breakage of the tension bond between the soil particles occurred and led to a reduction in the linear shear modulus.

6. Conclusion

The findings have shown that the proposed field testing technique is a feasible method to measure in-situ dynamic properties of soil. While the quality and the consistency have yet to be improved, the study has provided us a more complete understanding of soil dynamic properties and ideas of future studies.

The field testing technique for this study has a number of advantages that make it potentially a widely acceptable way to determine the non-linear soil characteristics. The combination of small strain tests and steady state dynamic tests helps us to relate better the linear and non-linear behaviour of soil to external factors such as confining pressure, soil inhomogeneity, etc.

The field testing technique was designed with several improvements on previous testing techniques. It minimizes the disturbances to the soil by requiring only drilling of boreholes for the installation of sensors. The use of vibroseis trucks allows careful control of the dynamic loads and static loads, therefore more control on the induced shear strain. Finally, the field test is likely to be repeatable on most soils except for soils such as cemented soil or sensitive clay.

Despite the improvements, the results reveal a number of issues with the field testing technique and the analysis methods. The dependency of shear modulus on frequency indicates the possibility of rocking involved, which produces pressure waves that interfere with the horizontal geophone signals. The geophone output quality is also affected by poor contact between footing and soil, soil inhomogeneity, uneven cementation and irregularities around geophones, etc. Besides, the range of dynamic loading should be expanded to obtain more results of the non-linear behaviour of the soil as the current results only show a fraction of the non-linear curve.

There are a number of potential ways to improve the test method and to ensure a good quality in raw data. To understand better how the shear wave is distributed in the soil, geophones can be installed further away from the footing both laterally and vertically. Vertical geophones should

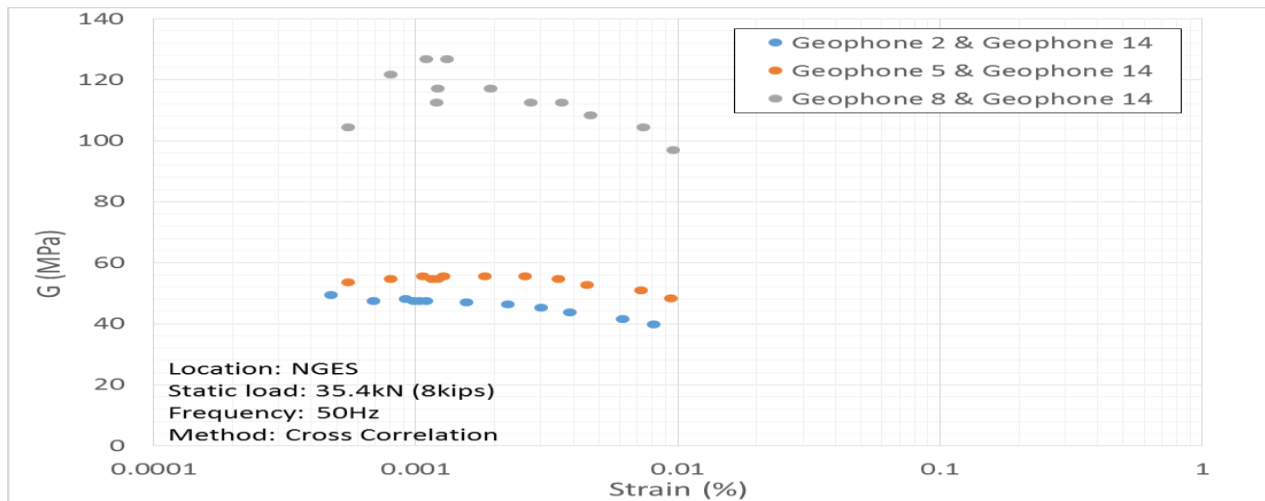
be installed to monitor the magnitude of rocking. Finally, it would be beneficial to check the dynamic load input and geophone outputs during the test to ensure everything is under control. If any of these appear unusual, the test should be repeated. However, this is not suitable for cemented soil as the cyclic shear would have caused irreversible damage to the cementation.

Three analysis methods, the sinusoidal curve fitting method, the by-eye method and cross correlation, have been carried out. It has been shown that sinusoidal curve fitting method is not feasible given that the geophone output is never close to a perfect sinusoidal curve. The results produced by the by-eye method are too scattered to be acceptable. Cross correlation has so far produced the most consistent results with adequate resolution but it lacks verification. The by-eye method could potentially be the best method with some modifications to improve the measurement resolution. Instead of comparing the chosen cycles within the steady-state range, a specific feature of the geophone outputs could be selected for the measurement. However, the biggest challenge will be on determining the feature to be compared. An extensive study of the geophone outputs will be required.

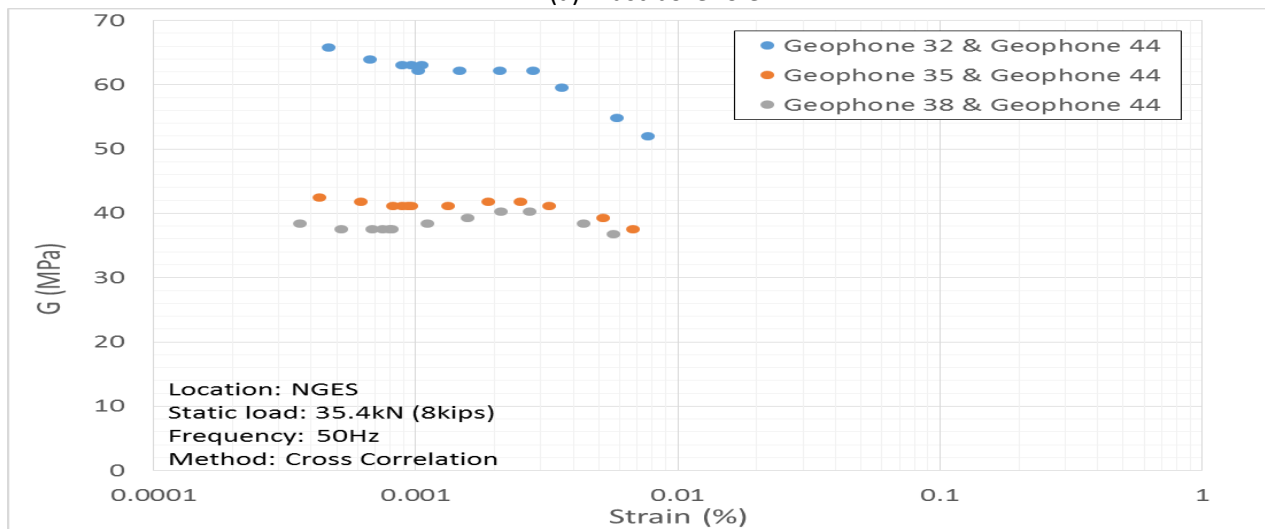
The study has brought about ideas of future studies to help us interpret the results better. Previous studies on the characteristics of shear modulus of cemented soils have been based on laboratory tests. Studies on in-situ determination of unevenly cemented soil would possibly help us clarify why the current results have a rising trend of shear modulus at very small strain. To justify the dependency on frequency, the influence of rocking of the footing should be investigated more thoroughly. Finally, the comparison with laboratory test results show that there might be a discrepancy in the determination of shear strain. The assumption that the characteristic shear strain is the average shear strain in the steady-state range would require more verification to be accepted.

Appendix

A: Steady-State Dynamic Test Results

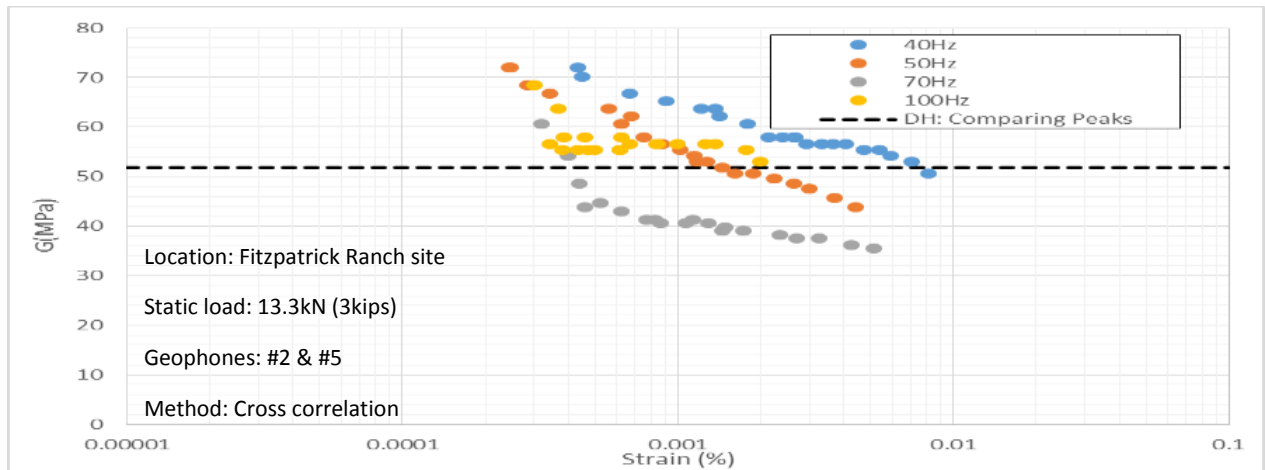


(a) East borehole

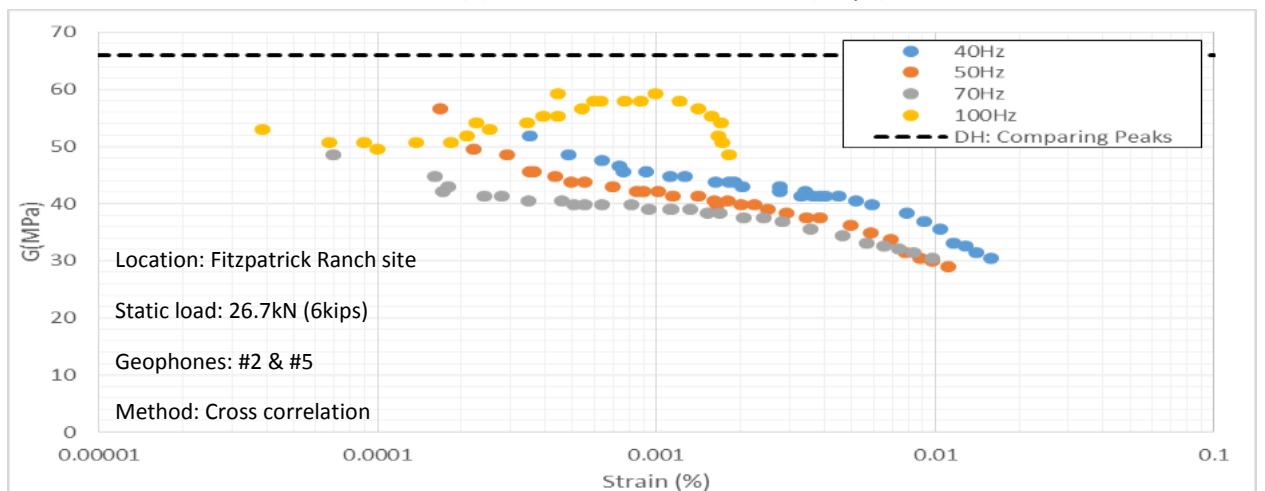


(b) West borehole

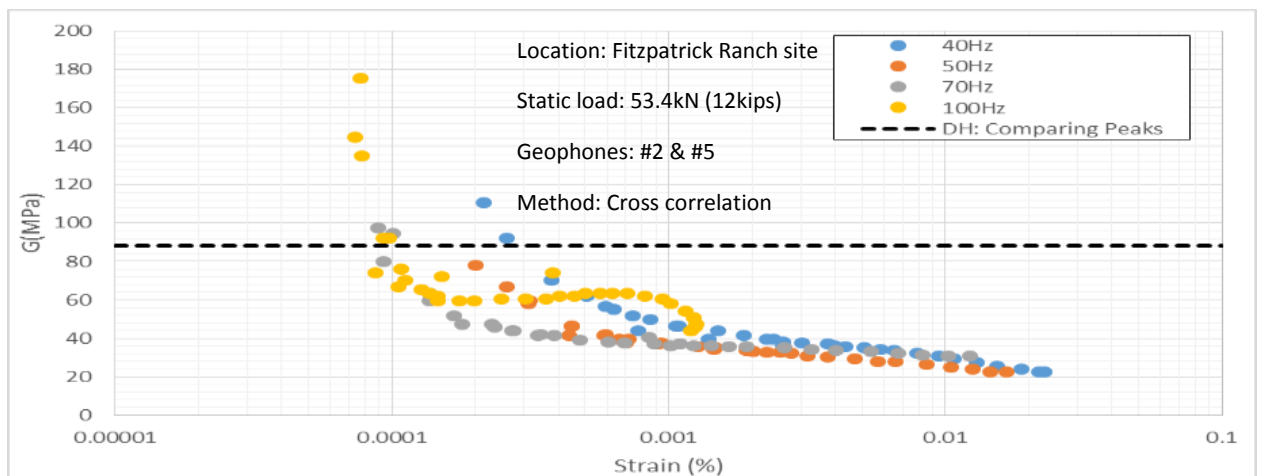
Figure A.1 – Steady-state dynamic test results for east and west boreholes produced by cross correlation (35.6kN (8kips) static load at the NGES)



(a) Under static load 13.3kN (3kips)



(b) Under static load 26.7kN (6kips)



(c) Under static load 53.4kN (12kips)

Figure A.2 – Steady-state dynamic test results for east borehole produced by cross correlation (Static Load: (a) -13.3kN (3kips); (b) - 26.7kN (6kips); (c) - 53.4kN (12kips); at the Fitzpatrick Ranch Site)

B: Downhole Test Results

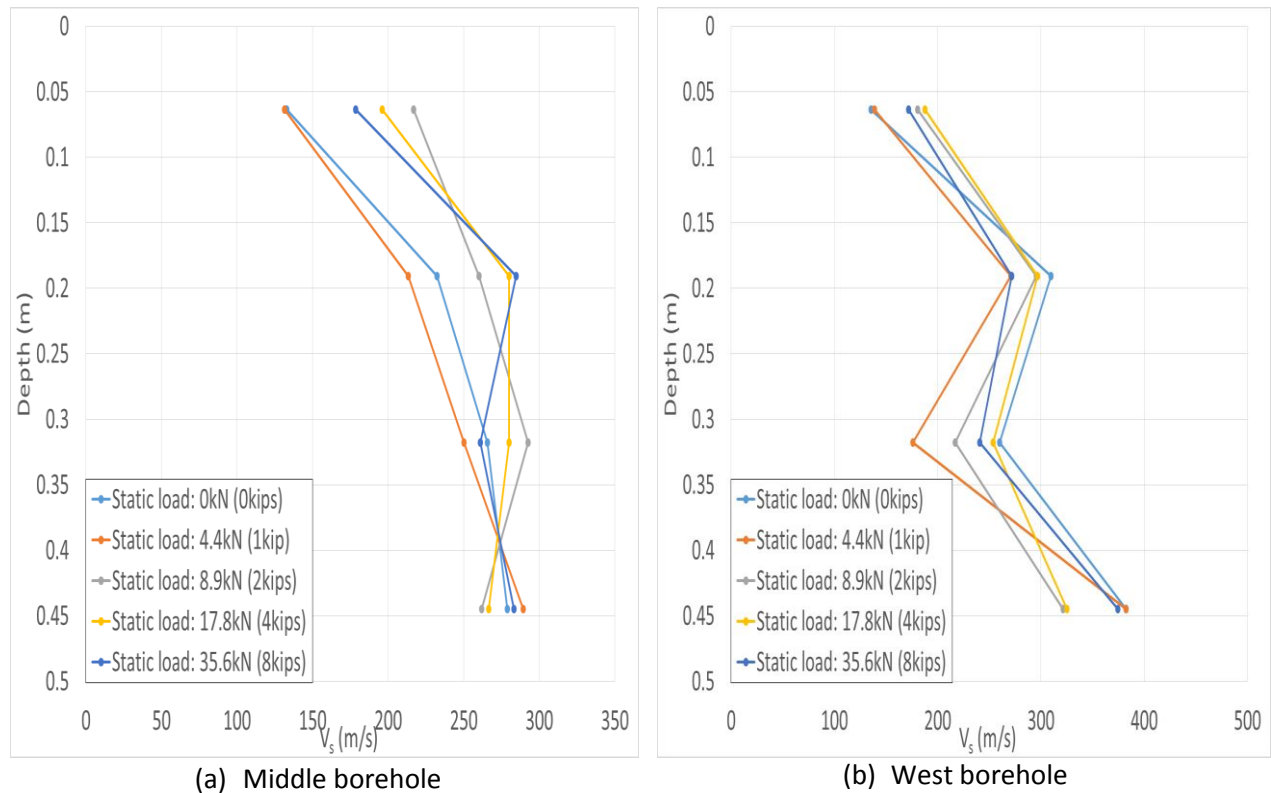


Figure B.1 – Shear wave velocity profiles for middle and west boreholes from downhole tests at the NGES

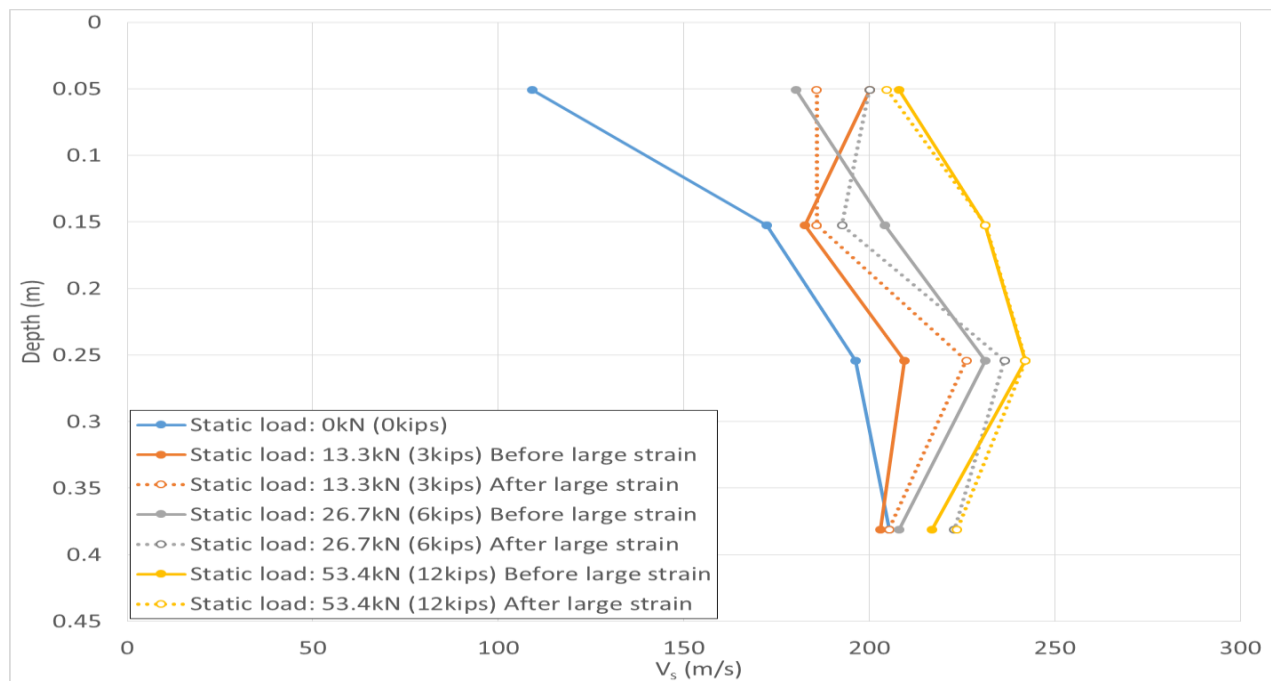


Figure B.2 – Shear wave velocity, V_s , profiles for west borehole from downhole tests at the Fitzpatrick Ranch Site

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