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IN-SITU AND LABORATORY SHEAR WAVE VELOCITIES

VELOCITE DE L'ONDE DE CISAILLEMENT IN SITU ET EN LABORATOIRE

ПОЛЕВЫЕ И ЛАБОРАТОРНЫЕ ИССЛЕДОВАНИЯ СКОРОСТИ СДВИГОВЫХ ВОЛН

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SYNOPSIS. Shear wave velocities were measured in-situ at four sites by using the cross-hole seismic survey method. Laboratory resonant column tests on samples taken from each borehole also established the shear wave velocity, v_s, for these soils under simulated field conditions. Comparable values of v_s were obtained by the two methods for silty sand samples, with laboratory values less than 5 percent lower than field values. For shale samples, laboratory values were 7 percent lower than field values for two samples and were 29 percent lower for a third sample. Laboratory values of v_s were 20 to 30 percent below field values for sandy silt and clayey silt samples, and each laboratory sample exhibited a significant effect of time of applied loading.

INTRODUCTION

Design or analysis of dynamically loaded foundations requires information about the shear modulus of soils under the design conditions. The shear modulus, G, can be evaluated from the shear wave velocity, v_s, in the soil by

$$G = \frac{\gamma}{g} v_s^2 \qquad (1)$$

in which γ is the unit weight of the soil, and g is the acceleration of gravity. Thus determinations of the shear wave velocity in soils under various conditions permit calculation of G for these same conditions.

Seismic methods are available for evaluating wave velocities in zones of soils in-situ. These methods measure the time for transmission of low-energy vibrations along selected paths in the soil mass, thereby determining the dilatational or shear wave velocities. Laboratory methods permit studies of the influence of loading conditions and material properties on wave velocities in samples of soils. Thus the laboratory tests provide a means for extrapolating the low amplitude field data to conditions more representative of the design situation.

Very few dynamic studies have been conducted on "undisturbed" samples recovered from the field, and only a portion of these have included comparison with field values (see, for example, Swain, 1962, Ballard and Leach, 1969, and Dobry and Poblete, 1969). These comparisons indicated a considerable scatter of the results This study presents data obtained from four field sites at which in-situ shear wave velocities were measured and "undisturbed" samples were taken for laboratory tests. Comparisons of the shear wave velocities obtained in the field and from resonant column tests in the laboratory are presented, and the significance of the testing parameters are discussed.

FIELD MEASUREMENTS OF SHEAR WAVE VELOCITY

The cross-hole seismic survey method (Stokoe and Woods, 1972) was employed in the field to determine in-situ shear wave velocities. By this method, the in-situ velocity of a horizontally-travelling shear wave was measured by monitoring the travel time of the shear wave between the bottoms of two boreholes a known distance apart (see Fig. 1). The shear wave was generated by a vertical impulse applied at the bottom of one borehole, and its arrival at the bottom of the second borehole was monitored with a vertical velocity transducer. Waves travelling in both directions were measured and the average of the two values was used. By advancing both boreholes to equal depths for each survey, shear wave velocity versus depth profiles were determined at selected intervals from the surface. Data concerning the field test procedures are given in columns 2 to 4 of Table I.

Samples were recovered from the same boreholes used for the cross-hole tests. Soil samples were obtained with a 5-cm OD thin-walled sampler (area ratio equal to 16 percent) hydraulically pushed into the soil. Shale samples were obtained by a coring operation with an NX-diam. diamond coring bit.

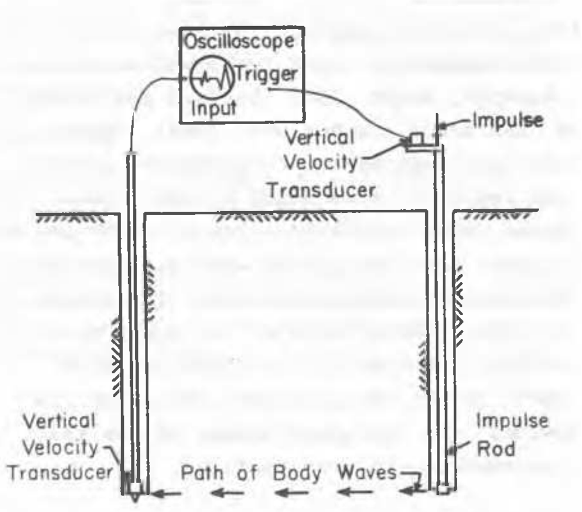


Fig. 1 Cross-Hole Seismic Surveying Method

LABORATORY MEASUREMENT OF SHEAR WAVE VELOCITY

Samples recovered during the cross-hole investigations were tested in the laboratory using the resonant column method. The soil specimens were excited by an oscillator developed by B. O. Hardin while the shale specimens were tested in an apparatus developed by V. P. Drnevich. Both types of equipment and the associated testing procedures are described in Richart, Hall, and Woods (1970)

In each test, the specimen and driving device were confined in a pressure cell and tests were conducted under hydrostatic conditions. Each specimen was surrounded by a water jacket and the confining chamber was pressurized with air and nitrogen for the soil and shale samples, respectively. The initial confining pressure, $\bar{\sigma}_0$, was held constant over a certain time interval while the change in v_s was measured. Then the confining pressure was increased, and the procedure was repeated. Data on the specimen characteristics and the loading conditions are given in columns 5 through 12 of Table I.

The shear wave velocity of all specimens was determined at a maximum shearing strain amplitude of 1.0×10^{-5} cm/cm. This order of amplitude has been shown to have no effect on the soil structure, and measured values of v_s at this amplitude can be considered equal to the small-amplitude reference value, $v_{s(max)}$, as defined by Hardin and Drnevich (1972a). Therefore, in this study the influence of differences in strain amplitude will be neglected when comparing values of v_s determined by the resonant column method in the laboratory and by the cross-hole method in the field.

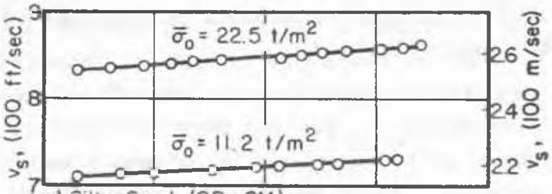
RESULTS OF LABORATORY TESTS

When "undisturbed" samples are tested in the laboratory to determine the low-amplitude shear wave velocity, the most significant test variables which influence v_s are the confining pressure (magnitude) and its duration (time effects) and the shearing strain amplitude applied. Before comparison of the laboratory and field values of v_s at comparable low strain amplitudes, it is necessary first to examine the effects of the magnitude and duration of the confining pressure on the laboratory results.

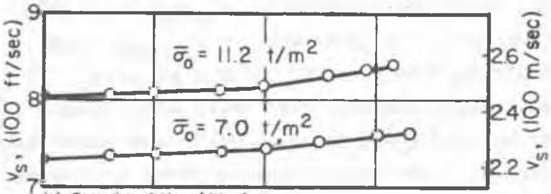
Effect of Duration of Confining Pressure

Typical results of the change in shear wave velocity with time at a constant confining pressure for the three soils tested are shown in Fig. 2. These results show that v_s continued to increase with time over the duration of testing time employed. For the silty sand, v_s increased linearly with the logarithm of time. However, for the more cohesive soils, the sandy silt and the clayey silt, the v_s-log time relationship was represented by two straight lines with a distinct change in slope. The time at which the change in slope occurred for the sandy silt and clayey silt was somewhat less than 1000 min., and the results are given in columns 3 and 4 of Table II.

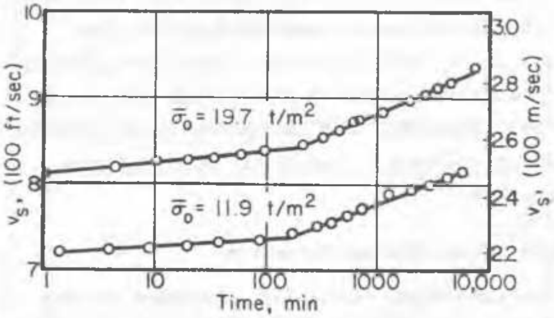
This increase in v_s with time, represented by the v_s-log time relationship for the silty sand and by the second straight line of the v_s-log time relationship for the sandy silt and clayey silt, will be referred to as the secondary time effect. This secondary time effect cannot be accounted for merely by the change in void ratio. No attempt is made in



a) Silty Sand (SP-SM)



b) Sandy Silt (ML)



c) Clayey Silt (CL-ML)

Fig. 2 Typical Results of Change in v_s with Time under Constant Cell Pressure

Table I Field and Laboratory Test Data

Material	Cross-Hole Tests			Resonant Column Specimen Data							Time of Pressure Increment days
	Max. Depth m	Borehole Spacing m	Depth Interval m	Depth m	Diam. cm	Ht. cm	Void Ratio	γ gm/cm³	w %	S_γ %	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Silty Sand (SP-SM)	6.1	2.9-5.4	0.3-1.0	3.3 4.3	3.56 3.56	10.00 10.00	0.82 0.72	1.57 1.65	7 6	23 22	1-5
Sandy Silt (ML)	4.6	3.0	0.3-0.6	2.0	3.56	8.00	0.77	1.67	10	35	1-4
Clayey Silt (Cl-ML)	3.7	2.5	0.3-0.6	2.4	3.83	9.90	0.40	2.26	12	84	3-5
Shale	12.2	5.9	0.6-1.0	11.3 11.6 11.9	5.36 5.38 5.38	10.50 13.51 8.02	- - -	2.61 2.60 2.60	- - -	- - -	0.01-1

this study to explain the cause of the secondary time effect, but the intent is to show its effect in testing "undisturbed" samples and to compare this effect with results found from artificial soils prepared in the laboratory.

The results of the secondary time effect on the increase in v_s per log cycle of time for the three soils are given in columns 5 and 6 of Table II. These results were determined by dividing the changes in v_s per log cycle of time by v_s at 1000 min. ($v_{s(1000)}$) and multiplying the results by 100 percent. These values compare very well with those found by Afifi and Woods (1971) and show that in general, secondary time effects increase as cohesiveness increases.

For the three shale samples tested, the change in v_s with time at a constant confining pressure varied in magnitude with each sample However, the secondary time effects noted in columns 5 and 6 of Table II are negligible.

Effect of Confining Pressure

The influence of confining pressure on the shear wave velocity of the soil samples tested is shown in Fig. 3. The values of v_s plotted in the figure were measured after 1000 min. at each $\bar{\sigma}_0$. This time interval was selected because it included some secondary time effect and because it was a convenient duration of testing. For the clayey silt, Fig. 3c, a second v_s-log $\bar{\sigma}_0$ relationship is also shown which was determined from a linear extrapolation to a time of 20 years for the v_s-log time relationship measured at each confining pressure. This second v_s-log $\bar{\sigma}_0$ relationship is shown to demonstrate the importance of the secondary time effect in determining the laboratory values of v_s for this particular soil.

Figure 3 shows that for all the "undisturbed" soils tested the v_s-log $\bar{\sigma}_0$ relationship could be approximated by two straight lines which intersected at a $\bar{\sigma}_0$ value very nearly equal to the preconsolidation confining pressure. In addition, the slope of the v_s-log $\bar{\sigma}_0$ relationship above the preconsolidation pressure was always larger than the slope of the same relationship below the preconsolidation pressure. The results in each case are given in columns 7 and 8 of Table II. Similar results have also been shown by Hardin and Drnevich (1972b)

Values of shear wave velocity determined from empirical expressions presented by Hardin and Richart (1963) are also shown in Fig. 3. For round-grained sands ($0.3 < e < 0.8$)

$$v_s = \left[170 - (78.2)\ e\right]\ (\bar{\sigma}_0)^{0.25} \qquad (2)$$

Table II Field and Laboratory Test Results

Material	Sample Depth m	Secondary Time Effect				Slope of log v_s - log $\bar{\sigma}_0$ Relationship		Shear Wave Velocity m/sec		
		Time of Slope Change, min		Increase in v_s per log cycle, %		Normally Consol.	Over-Consol.	Field	Lab	Empirical
		Range	Average	Range	Average					
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Silty Sand (SP-SM)	3.3 4.3	- -	- -	0.6-1.6 1.7-2.1	1.0 1.9	0.23 0.25	0.13 0.15	186 207	186 217	180 207
Sandy Silt (ML)	2.0	50-200	125	1.7-2.8	2.0	0.28	0.22	293	207 (K_o=2)	214 (K_o=2)
Clayey Silt (CL-ML)	2.4	180-900	395	4.0-6.9	5.3	0.30	0.11	262	186 (1000 min) 229 (20-yr.)	244
Shale	11.3 11.6 11.9	6-12	10	0.0-0.5	0.2	-	-	1010 1050 1160	940 980 830	- - -

For angular-grained sands ($0.6 < e < 1.3$)

$$v_s = \left[159 - (53.5)\ e\right]\ (\bar{\sigma}_0)^{0.25} \qquad (3)$$

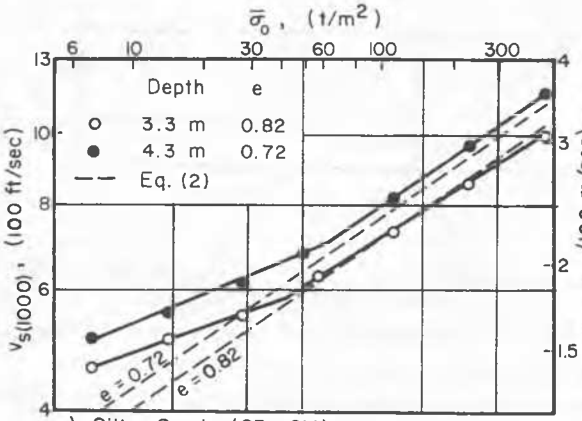
In Eqs. (2) and (3), v_s is in ft/sec; $\bar{\sigma}_0$ is in lb/ft^2; and e is the void ratio.

As can be seen from Fig. 3, good agreement between empirical and laboratory values of v_s was found for the silty sand and sandy silt samples tested in the normally-consolidated range. Similarly, good agreement was found between empirical and 20-year extrapolated values of v_s for the clayey silt sample tested in the normally-consolidated range. On the other hand, the behavior of all the samples in the over-consolidated range was not predicted as well by the empirical relationships even if a correction factor (see Hardin and Drnevich, 1972b) based on the over-consolidation ratio, OCR, and plasticity index of the soil was applied. For the samples tested, the maximum effect of this correction would be an increase in v_s of about five, two, and zero percent for the clayey silt, sandy silt, and silty sand, respectively, with this effect occurring at the lowest confining pressure (maximum OCR) and decreasing as the pressure increased.

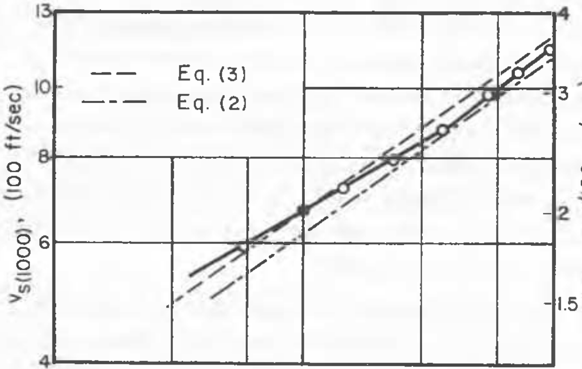
The influence of confining pressure on v_s for one of the shale samples tested is shown in Fig. 4. This sample was subsequently broken, purposely, along two approximately horizontal surfaces about at third points in the sample height in an attempt to reproduce conditions of horizontal jointing which existed in the field. The results of tests on the broken sample are also shown in Fig. 4. For the intact sample, increasing the confining pressure produced only a small increase in v_s for pressures greater than the in-situ confining pressure. For confining pressures below the in-situ pressure, the influence of $\bar{\sigma}_0$- changes on v_s was greater, because cracks which opened as a consequence of stress relief during sampling were again closed as pressure was reapplied. The presence of a totally fractured surface was to reduce the value of v_s significantly, as shown by the lower curve in Fig. 4.

COMPARISON OF FIELD, LABORATORY, AND EMPIRICAL VALUES OF SHEAR WAVE VELOCITY

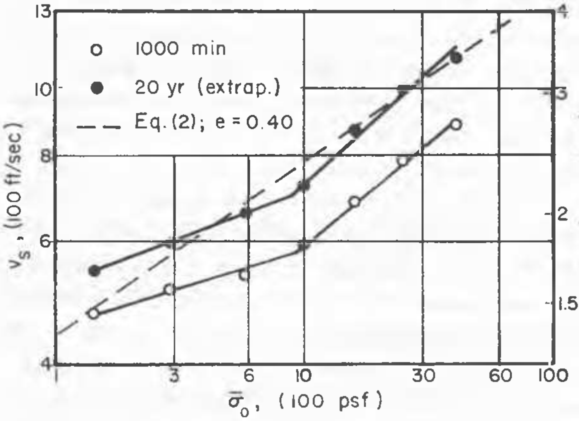
A comparison of values of v_s determined in the field, in the laboratory, and from empirical expressions is given in columns 9 to 11 of Table II. The field values are the values that were determined at depths within 0.3 m



a) Silty Sand (SP-SM)



b) Sandy Silt (ML)



c) Clayey Silt (CL-ML)

Fig. 3 Change in v_s with $\bar{\sigma}_0$ for Soil Samples

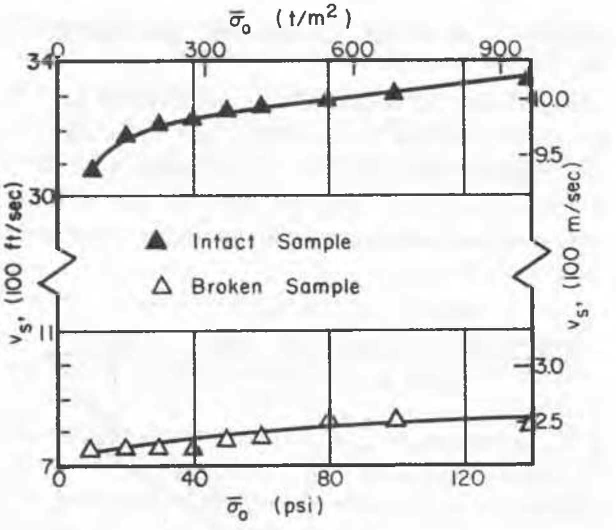


Fig. 4 Change in v_s with $\bar{\sigma}_0$ for Shale Sample from a Depth of 11.6 m

of the sample depth. The laboratory and empirical values were determined from the results shown in Figs. 3 and 4 and calculations of the in-situ confining pressure.

For the silty sand, very good agreement was found between laboratory and field values. Empirical values of v_s (Eq. (2)) also agreed very well with the laboratory and field results provided the proper values of void ratio and $\bar{\sigma}_0$ were used. In all cases, the maximum variation between values of v_s was less than five percent.

For the sandy silt, the laboratory value of v_s was about 30 percent less than those measured in the field. The laboratory value was taken from Fig. 3c at a $\bar{\sigma}_0$ which was determined from the site using a lateral stress ratio, K_0, equal to two. This was done because in the past ten years a surcharge on the order of 10 m to 15 m had been removed from the site, and the horizontal stresses were assumed not to have been completely relieved. However, even with this correction for $\bar{\sigma}_0$, the laboratory value significantly underestimated the field values. This result cannot be explained by time effects which the laboratory test showed to be small, but the comparison indicates that there were significant factors affecting the in-situ value of v_s which were not accounted for in the laboratory testing of this material. The empirical values of v_s predicted by Eqs. (2) and (3) agreed quite well with the laboratory results and therefore did not agree well with the in-situ results.

For the clayey silt, the value of v_s in best agreement with the in-situ v_s was that value determined by the empirical relationship, Eq. (2). In this case, the empirical value was about seven percent less than the field result. Values of v_s determined in the laboratory were 20 percent and 13 percent less than the in-situ v_s for the 1000-min. and the 20-year value, respectively. However, the laboratory values do show the importance of time in testing this clayey silt, with the closest agreement between laboratory and field results occurring for the extrapolated value of v_s at 20 years.

For the shale, laboratory values of v_s generally agreed quite well with in-situ values. The laboratory results for the samples from 11.3 m and 11.6 m were only about seven percent less than the field results at these depths. On the other hand, the laboratory value for the sample from 11.9 m was about 29 percent less than the in-situ v_s. However, it should be noted that these laboratory tests determined the shear wave velocity travelling in the vertical direction while the cross-hole method samples in the horizontal direction. Some anisotropy should be expected in a horizontally-bedded material such as this shale.

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CONCLUSIONS

Measurements of the dynamic shear wave velocity, v_s, in soil and shale were conducted in the field by the cross-hole seismic method and in the laboratory by the resonant-column method. General conclusions based on these measurements are:

1) For all soils tested in the laboratory, v_s increased with increasing confining pressure, $\bar{\sigma}_0$. The v_s - $\bar{\sigma}_0$ relationship plotted on log-log paper could be approximated by two straight lines intersecting at approximately the maximum preconsolidation pressure, with the rela-

tionship above the maximum preconsolidation $\bar{\sigma}_0$ having the steeper slope. For the shale tested, v_s also increased with increasing $\bar{\sigma}_0$ but to a much lesser extent than found for the soils tested.

2) A secondary time effect occurred during all laboratory tests at a constant $\bar{\sigma}_0$. This was indicated by an increase in v_s with time. For the soil samples, this secondary time effect increased in magnitude as the soil became more cohesive and agreed well with values predicted by Afifi and Woods (1971). For the shale samples, the secondary increase in v_s was negligible.

3) In every case except for the silty sand, v_s measured by the field method exceeded v_s measured by the laboratory method by at least seven percent. For the sandy silt, very close agreement between laboratory and field values of v_s was found. However, when v_s is determined by the laboratory method, the appropriate confining pressure must be selected, and secondary time effects must be evaluated.

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