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Evaluation of Field Shear Wave Velocity in Deep Soil Mixing Based on Laboratory Studies

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ABSTRACT

One of the reasons behind the stabilization of soft and problematic soil by deep soil mixing is to reduce the amplification of seismic waves which arrive to the ground surface and foundation of buildings. Therefore, it is important to correctly predict the dynamic properties of the improved ground. To address the dynamic properties of deep soil mixing, this paper evaluates the in-situ shear wave velocity of deep soil mixing ($V_{s\text{-}field}$) based on laboratory investigation ($V_{s\text{-}lab}$). The conversion factors, relating to the shear wave velocities of laboratory and field, have been obtained based on a variety of tests including bender element, pulse velocity, and low-amplitude dynamic tests in the resonance column. In this study, the effect of confinement and vertical stress on the dynamic properties of the base stabilized using deep soil mixing technology was evaluated. These effects were combined with the known disturbance and aging influence which is available in the literature.

The research has shown that the most significant factors affecting the shear wave velocity are confinement stress and additional vertical load, which lead to 43% and 17.5% increase respectively.

Keywords: Deep soil mixing, shear wave velocity, conversion factor, bender element, resonant column

1. INTRODUCTION

Deep soil mixing (DSM) is used broadly to improve the geotechnical properties of soft and problematic soils. It can be applied to most types of soils, including sandy and clayey soil. The main objectives of DSM are increasing the bearing capacity, reducing total and differential settlements, and enhancing the dynamic and cyclic properties of the ground in seismic regions.

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With DSM, cementitious material is mechanically mixed with the in-situ soil by using a hollow stem auger and paddle arrangement. Mixing can be done using a single auger or a gang of two to eight augers. As the mixing auger advances, the cementitious materials are pumped and mixed with the soil. After the target depth is achieved, augers are withdrawn while the mixing continues. The improvement can be done employing individual improved soil columns in some cases or the entire soil volume underlying the foundation (mass stabilization or block type treatment).

An essential step in the design of earthquake-resisting structures is to correctly predict the characteristics of the ground motion that comes to the structure base. The subsurface soil properties may amplify or de-amplify the level of ground surface shaking while propagating from the bedrock surface through the overlying soil layers. One of the key subsurface information for dynamic soil and rock properties that determines the soil amplification or de-amplification is the shear wave velocity (V_s). For earthquake engineering design purposes, many codes and standards- such as AASHTO [1], ASCE 7-16 [2], Eurocode 8 [3], and the International Building Code [4] - classify subsurface soil profiles using the average shear wave velocity ($V_{s,30}$) measured for the top 30 m of soil.

Despite of many laboratory studies on the deep soil mixing, including bender element tests [5,6], the ultrasonic pulse [7] and seismic downhole test [8], there are little information regarding the dynamic and cyclic deformation properties of deep mixing treated soil in the geotechnical literature. For the in-situ strength and deformation properties of clayey soil, there are some tests like falling cone or vane shear test [9]. These tests can be indirectly correlated to shear wave velocity. However, the seismic downhole test could directly determine the dynamic in-situ properties according to ASTM D7400 [10]. Furthermore, it is necessary to perform a laboratory investigation to predict the dynamic properties of the stabilized soil at the initiation phase and before the commencement in the building process. The goal of this paper, therefore, is to conduct lab studies that forecast the change of the improved soil dynamic properties due to the loading of soil improvement with the weight of the building and the increase of wave velocities because of aging of soil-concrete. It is worth mentioning that after the building construction, the stabilized bedding is not easily accessible for in-situ field tests, so finding the relation between field and laboratory shear wave velocity is highly important. The considered type of DSM is mass stabilization in clayey soil which will be accomplished by employing overlapping columns. In the single fluid technique, neat binder slurry is injected into the ground and it creates a stabilized soil column with a diameter of about 1 to 2 m. [11].

2. TEST PROCEDURES

The soil layer classification mostly consists of CL, SM, and CL-ML. Tests conducted on soil-cement mixes include bender element testing [12] and sonic testing for measurement of shear wave velocity. For the sample preparation, first, water and cement were mixed, and cement slurry was formed. Then, the cement slurry is mixed with the soil, and cured until testing. The total water-to-cement ratio ($w_t : c$) is 1.7. Then the bender element, pulse velocity, and low-amplitude dynamic tests in the resonance column test were performed on the samples.

Bender elements were piezoelectric bimorphs utilized in pairs to measure the shear wave velocity in a soil specimen. This includes inserting each element near the top and base of a specimen, then applying an excitation voltage to one element to generate a shear wave in the soil. The other elements were used to grab the shear wave that had propagated through the specimen, with its displacement owing to the wave inducing voltage, which was then read by a data acquisition unit. Having the distance between the two elements and monitoring the time needed for the shear wave velocity to propagate, a value of shear wave velocity was acquired.

Bender element tests were conducted for the soil-cement mixtures under no-confinement conditions and confinement condition. Tests were conducted on cured samples which were often cured for 7, 14, 28, and 56 days [13], as shown in Figure 1. Laboratory shear wave velocities were obtained for approximately 28 days of aging. However, the design seismic performance of the improved soil-cement columns was required after the completion of the construction. Piezoelectric elements were carefully inserted into the specimen to ensure full contact. The input signal frequency was varied to determine the strongest output signal, which corresponded to the natural frequency of the sample. Several tests were conducted upon the determination of the input signal frequency. It is preferred to use sin waves as the input frequency, as long as the output can be obtained without significant noise for this wave type. Alternatively, a step function can be considered.

An alternative test can be used to measure wave velocities in the pulse velocity test in accordance with ASTM D2845 [14]. This test can also be carried on selected samples as a check on the Bender test data.

The low-amplitude dynamic tests in the resonance column were conducted on cored elements after the installation of DSM under the building, and they were described in the following sections.

(a) (b)

Figure 1 - a. Demolded cylindrical specimens, b. Samples storage in pools for curing

3. BENDER ELEMENT AND PULSE VELOCITY TESTS

Shear wave velocity measurements were performed on the samples prepared in the same 5-cm diameter and 10 cm length used for the unconfined compression tests based on ASTM D2166 [15]. These tests involve very low strain excitations. Both bender element tests [16] and sonic pulse velocity tests [14] were conducted. Five sonic pulse velocity tests were used to verify the results from the bender element tests.

Table 1 - Bender Element results for unconfined testing of cement Type II samples

Sample ID.	Age (days)	ρ (kg/m^3)	V_{s0} (m/s)
1.1	31	1,786	671
1.2	33	1,824	639
1.3	28	1,743	822
1.4	28	1,778	671
1.5	28	1,761	637
1.6	28	1,849	696
1.7	28	1,863	565
1.8	30	1,878	530
1.9	31	1,684	600
1.10	31	1,692	641
1.11	31	1,691	548
1.12	29	1,720	523
1.13	29	1,692	515
1.14	29	1,732	499
1.15	29	1,599	637
1.16	29	1,605	658
1.17	29	1,602	713
1.18	32	1,639	561
1.19	32	1,633	618
1.20	32	1,601	630
1.21	29	1,613	712
1.22	29	1,601	667
1.23	32	1,675	467
1.24	32	1,677	493
1.25	29	1,802	697
1.26	29	1,816	673
1.27	29	2,005	659
1.28	29	1,851	581
1.29	29	1,850	772
1.30	29	1,844	698

Table 1 - Bender Element results for unconfined testing of cement Type II samples (continue)

Sample ID.	Age (days)	ρ (kg/m^3)	V_{s0} (m/s)
1.31	28	1,826	566
1.32	28	1,906	665
1.33	28	1,826	832
1.34	28	1,762	714
1.35	28	1,858	577

3.1. Factors Considered in Bender Element Tests

The list of samples tested and the corresponding shear wave velocity test, for unconfined conditions (V_{S0}), were given in Table 1 and Table 2 for cement Type II and Type V samples respectively. The last five results (sample ID: 1.31-1.35) presented in Table 1 correspond to the pulse velocity test method. For the primary testing method of bender element tests, the study addressed the effect of loading frequency, effect of bonding between the piezoelectric element and the sample, effect of confinement, effect of sample quality, and effect of natural frequency.

Table 2 - Bender Element results for unconfined testing of cement Type V samples

Sample ID.	Age (days)	ρ (kg/m^3)	V_{s0} (m/s)
2.1	33	1,751	589
2.2	33	1,735	580
2.3	33	1,733	630
2.4	32	1,782	605
2.5	32	1,732	572
2.6	33	1,738	658
2.7	32	1,702	529
2.8	32	1,700	591
2.9	34	1,776	614
2.10	32	1,677	529
2.11	32	1,680	495
2.12	32	1,971	814
2.13	31	1,950	695
2.14	31	1,912	761
2.15	33	1,897	787
2.16	32	1,909	759
2.17	33	1,729	631
2.18	32	1,745	513

Table 2 - Bender Element results for unconfined testing of cement Type V samples (continue)

Sample ID.	Age (days)	ρ (kg/m^3)	V_{s0} (m/s)
2.19	32	1,772	483
2.20	32	1,755	523
2.21	30	1,743	804
2.22	30	1,795	630
2.23	30	1,791	518
2.24	28	1,986	794
2.25	33	1,862	847
2.26	32	1,893	741
2.27	32	1,881	690
2.28	26	1,742	708
2.29	31	1,900	751
2.30	31	1,881	723
2.31	31	1,893	688
2.32	27	1,770	616
2.33	27	1,781	539
2.34	27	1,737	572
2.35	27	1,697	614
2.36	27	1,689	660

The measured output signal depends on the input loading frequency. At certain frequencies, secondary and tertiary wave arrivals may be recorded with greater amplitudes that do not correspond to true shear wave velocity measurements. Therefore, multiple methods of data interpretation techniques were used to identify false peaks such that the correct shear wave velocity pick is made. Three data interpolation techniques were used during data analysis: Time Domain-Peak Arrival Time, Cross-Correlation method, and Power Spectrum Comparison method. The final velocity was obtained as the average of two methods when any of them above provide velocities that were within 10% of each other. If all three methods produce velocities within 10%, then, the velocity is calculated as the average of velocities obtained from all three methods.

Since the samples were too stiff to insert the piezoelectric elements, a small groove was opened in the sample to fit the bender elements. The gap between the piezoelectric element and the sample was filled with natural clay, and different types of materials and conditions were tested to get the best output signal response. The natural moist clay fill provided the best output signal quality.

The shear wave velocity varies as a function of confinement in the field. Therefore, 20 tests were conducted with confinement to assess the functional relationship between the shear wave velocity and the confinement pressure. Table 3 shows typical normalized V_s values for different confinement pressures for cement type V samples.

Table 3 - Normalized shear wave velocity (V_s/V_{so}) results for different confinement pressure for cement type V (sample ID: 2-1)

psi	kPa	Sample ID: 2.1
0	0	1.00
5	34	1.38
10	69	1.41
15	103	1.41
20	138	1.47
25	172	1.47

Since the samples were prepared in relatively small molds compared to the size of field-installed columns (1.6 m vs. 5 cm), small imperfections in the mixing process produced a greater impact on the measured velocity. Therefore, samples were ranked in terms of their quality during testing to obtain a reliable velocity. The sample rating system is qualitatively based on sample external appearance. The rating scale runs from 1 to 5, where 1 is the lowest quality and 5 is the best quality. Only samples with a quality rating of at least 3 were tested. The results showed that sample quality had a noticeable influence on the V_s value. This is observed when comparing quality 3 to quality 5.

For the fundamental mode of vibration, the natural frequency of the sample was approximated using Eq. (1) [17]:

$$f_n = V_s / 4H \tag{1}$$

Where f_n = natural frequency in kHz, V_s= expected shear wave velocity (m/s), H= sample height (mm)

One test at the natural frequency is conducted to check if the maximum response is obtained.

If the resonance is not obtained with the maximum response, the frequency is increased by 10 Hz interval until the maximum response is obtained (first peak response).

3.2. Conversion Factors

The measured shear wave velocities in the laboratory under unconfined conditions were lower than the in-situ shear wave velocity in the field for the following two reasons:

a) **Confinement level:** Confinement level in the field increases the shear wave velocity. Since structural loads increase the confinement, they are also expected to increase the shear wave velocity.

b) **Disturbance or imperfections:** The small laboratory samples with a diameter of 5 cm are more likely to cause a decrease in velocity than for large improved soil columns in the field with a diameter of 1.6 m. Besides, extrusion, cutting, and preparation of the laboratory samples in the laboratory introduce more disturbances to the laboratory samples. These disturbance sources do not exist during the field mixing conditions. The effect of disturbance is more pronounced for shear wave velocity as the small-strain measurements are more prone to disturbance compared to, for example, large-strain unconfined compressive strength.

3.2.1. Confinement Level

Bender element tests were conducted for soil sampled from the site and mixed with cement in the laboratory to determine the conversion factor depending on the confinement level. The tests were conducted at 0 psi, 5 psi, 10 psi, 15 psi, 20 psi, and 25 psi (0, 34, 69, 103, 138, and 172 kPa) confinement levels to measure the effect of confinement. These confinement levels correspond to the confinement levels anticipated for the 20 m deep soil-cement columns. The effect of structural loads was not considered here as a conservative approach. For each test, the shear wave velocity at each confinement level was normalized with respect to the no-confinement shear wave velocity, and they were all plotted on the same normalized shear wave velocity vs. confinement plot (Figure 2). A conversion factor was then obtained to account for confinement, C_{conf}, which means an increase in shear wave velocity compared to the shear wave velocity measured for unconfined conditions. An average conversion factor interpreted of 43%, i.e., C_{conf}=1.43 was interpreted for all depths in the field. This means an increase in shear wave velocity by 43% compared to the shear wave velocity measured in the laboratory for unconfined conditions.

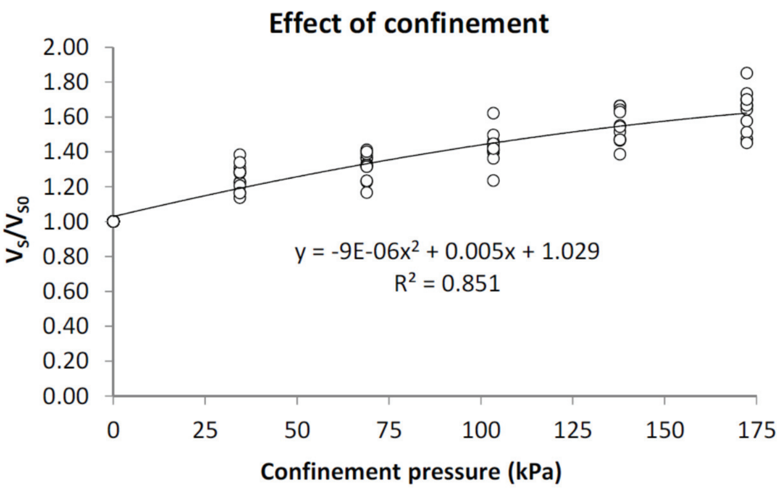

Figure 2 - Normalized Shear Wave Velocity vs. Confinement Pressure Plot

3.2.2. Disturbance Effects

Disturbance effects were quantified by Chiara and Stokoe [18] as shown in Figure 3 for soil samples. Field measured shear wave velocities shown in Figure 3 are consistently higher than laboratory-measured shear wave velocities. If the mean-one standard deviation curve on Figure 3 is considered, the conversion factor from the laboratory to the field is around 20%, i.e, C_{dist}=1.20. This factor corresponds to the average shear wave velocity measured in the laboratory for unconfined conditions, and it is conservative since it is based on the mean-one standard deviation curve and not the mean curve.

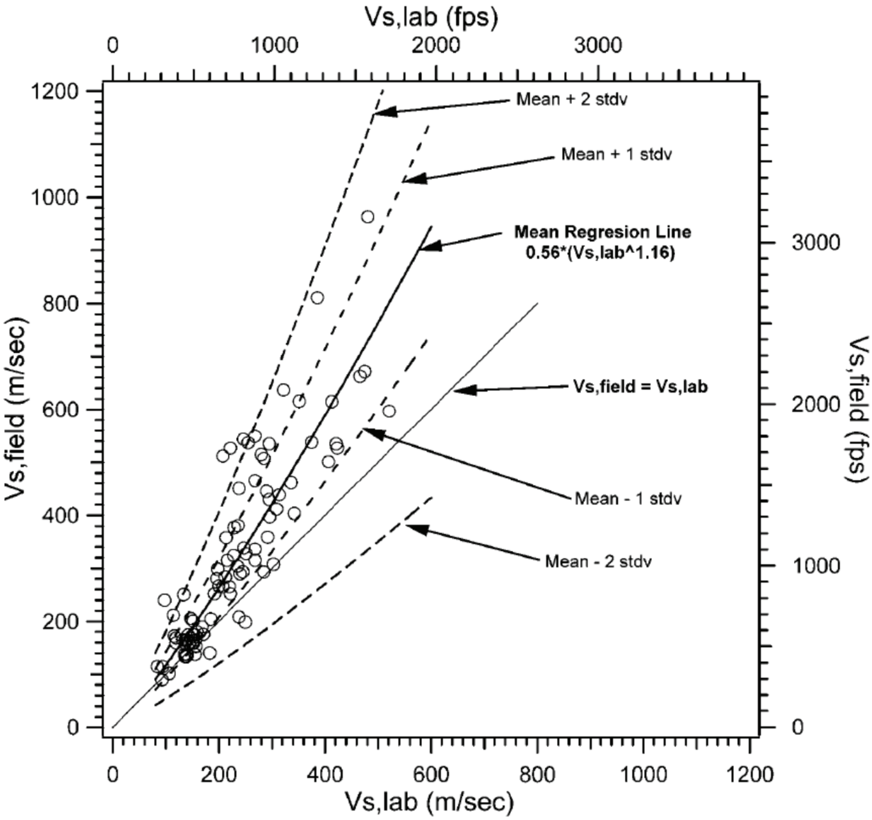

Figure 3 - Disturbance Effects, Field Shear Wave Velocity vs. Laboratory Shear Wave Velocity [18]

4. RESONANCE COLUMN TESTS

The objective of the resonance column test is to determine the elastic shear wave velocity V_s of the soil-cement specimens cored under the building and its dependence upon the values of confining and axial loads.

In this study, the laboratory testing was conducted by the method of low-amplitude dynamic tests in the resonance column within the approximate shear deformation range of 10^{-4}-10^{-2} %.

The soil-cement specimens prepared in the form of a solid cylinder were placed into the triaxial compression chamber. After preliminary consolidation, these specimens were put through the action of dynamic loading by application of variable frequency torsional vibrations to the upper free-end face of the specimen. The soil-cement experiences a simple shear deformation during vibrations. The shear deformation amplitude is determined at each frequency of the specified range, which allows obtaining the amplitude response spectrum of the soil-cement and determines the resonance frequency of the specimen. Using the first natural frequency of the specimen, it is possible to calculate the shear wave velocity (V_s) and then the dynamic shear modulus (G).

The dynamic tests of soil-cements, using the resonance column method, were conducted as per the consolidated-undrained scheme. The reason behind that is any change of the shape or weight of the specimen in the course of testing may change its moment of inertia and distort the final result.

The research was performed using the resonance column with an electromagnetic system for torsional vibration generation, which was drowned inside the triaxial compression chamber. The accelerometers were used as devices for measuring small angular movements (rotations). The loading system supplied the excitation of torsional vibrations within the range from 0 to 200 Hz. The general view of the test facility is shown in Figure 4.

Figure 4 - General View of the Resonance Column

Data processing consists of the calculation of dynamic loading concerning the shear deformations γ for all steps, the determination of its value $\tau_{\max}$ at the resonance frequency ω, and the calculation of the shear wave velocity V_s for this frequency with subsequent determination of the dynamic shear modulus G.

The relative shear deformation was calculated at each excitation frequency spacing by the angular displacements measured as the tangent of the azimuth angle Θ at the point moved away from the specimen center by 0.78r for solid specimens with radius r. Following the results of the calculation, the resonance curve was plotted, from which the resonance vibration frequency value ω is taken at $\gamma=\gamma_{max}$.

The shear wave velocity V_s (m/s) can be computed from the following formula:

$$V_s = \omega h \left(\frac{J}{J_0} \right)^{\frac{1}{2}} \tag{2}$$

where J is the moment of inertia of the specimen (to be determined from the weight and geometric dimensions of the specimen according to the known physical relations at the moment of completion of consolidation); for a solid cylindrical specimen with the weight m and radius r (kg.m^2);

J_0 is the moment of inertia of the power drive of the test facility (to be specified by the manufacturer in the device documentation) (kg.m^2); h is the specimen height (m); ω is the resonance frequency (rad/s).

Having determined the shear wave velocity V_s from Eq. 2, the dynamic shear modulus G (Pa) was calculated from the following formula:

$$G = \rho V_s^2 \tag{3}$$

where ρ is the specimen density (kg/m^3).

Laboratory tests were performed on samples of improved soil drilled out from the building's footing with a diameter of at least 80 mm and a height of at least 160 mm.

The tests were performed on 3 samples, where number of V_s determination is 9. The load combinations are provided in Table 4.

Table 4 - Load combinations for wave velocity measurements

Load No	σ_3 , kPa	σ_1 , kPa
1	100	100
2	100	400
3	100	700

The study obtained the results of the tests showing the impact of the vertical load on the S-wave velocity and confirming the growth of S-wave velocity V_s in the soil-cement footing of the building.

The results of the tests identified the dependence of the S-wave velocity upon the vertical load. Figure 5 shows the S-wave values normalized to the velocity during isotropic compression at 100 kPa for the same specimen.

The expected velocity gain from the direct action of the vertical load can be equal to 15-20 % (average 17.5%) with an increase in the vertical stress σ_1 from 100 kPa to 700 kPa. The obtained weight conversion factor C_{wt} based on the logarithmic regression analysis is as follows:

$$C_{wt} = V_s / V_{s0} = 0.0966 \ln(\sigma_1) + 0.5603 \tag{4}$$

The research results were presented in graphical form in Figure 5, on which trend lines were grouped according to each borehole and vertical loads σ_1. The highest approxiamation confidence coefficient $R^2 = 0.7565$ was obtained for the second trend line which was given in Figure 5. An average conversion factor of 17% (i.e. C_{wt}=1.17) is interpreted for the impact of vertical load.

Figure 5 shows that when the vertical load increases, the shear wave velocity increases according to weight conversion factor function given in Eq. (4). Depending on higher shear wave velocity of approaching wave, the frequency of the soil beneath the building will be higher which results in less vibration period on the superstructure. Based on the less vibration period of the structure, the top displacement can be kept within the acceptable range for more safe and economical design.

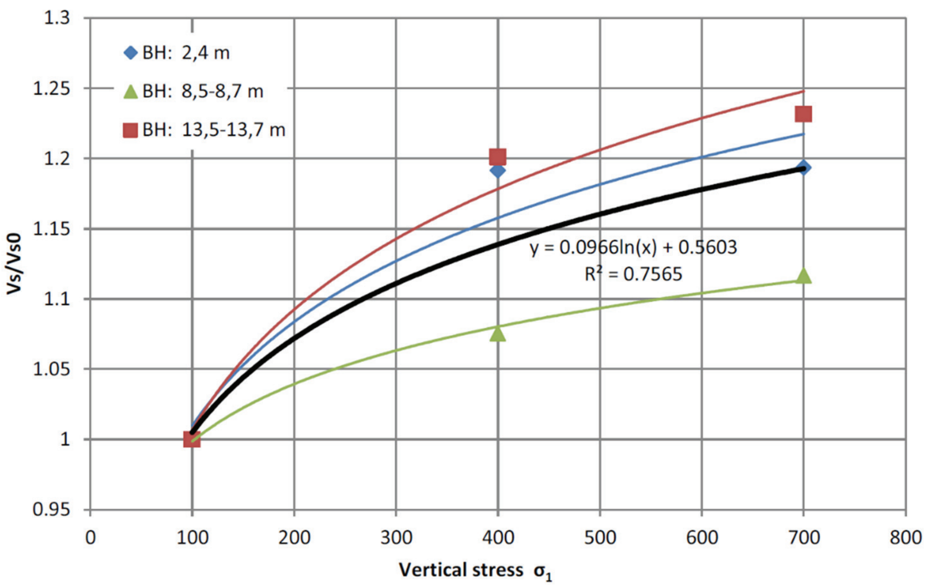

Figure 5 - S-wave velocity increasing ratio vs. vertical load; V_{s0} is the S-wave velocity during isotropic compression at 100 kPa

5. ESTIMATION OF AGING EFFECTS ON THE SHEAR WAVE VELOCITY

According to the FHWA Deep Soil Mixing Manual [13], the effect of aging due to curing can be incorporated using the following equation:

$$f(t) = 0.187\ln(t) + 0.375 \tag{5}$$

Where t is the curing time in days; *f(t)* is the ratio of unconfined compressive strength at time t to the unconfined compressive strength at 28 days.

The shear wave velocity can be correlated to the unconfined compressive strength through the general equation [19].

$$V_s = bf_c^{\varphi} \tag{6}$$

where f_c is the unconfined compressive strength.

The ratio of the S-wave velocity before aging and after aging can be used to cancel out the coefficient *b*

$$\frac{(V_s)_t}{(V_s)_0} = \left(\frac{(f_c)_t}{(f_c)_0}\right)^{\varphi} = (f(t))^{\varphi} \tag{7}$$

For a curing period of 10 years, the ratio *f(t)*, as calculated by Eq. (5), was equal to 1.9. Using the power of 0.2 and 0.25, the range of the shear wave velocity ratio after 10 years varied between 1.13 to 1.17.

Figure 6 shows the values of the aging conversion factor $C_{age} = (f(t))^{\varphi}$ for a period of 10 years using the power of 0.25. For example, the C_{age} coefficient value corresponding to the S-wave velocity, gained in the soil improvement foundation from the age of 1 year to the age of 5.5 years, is equal to 1.05.

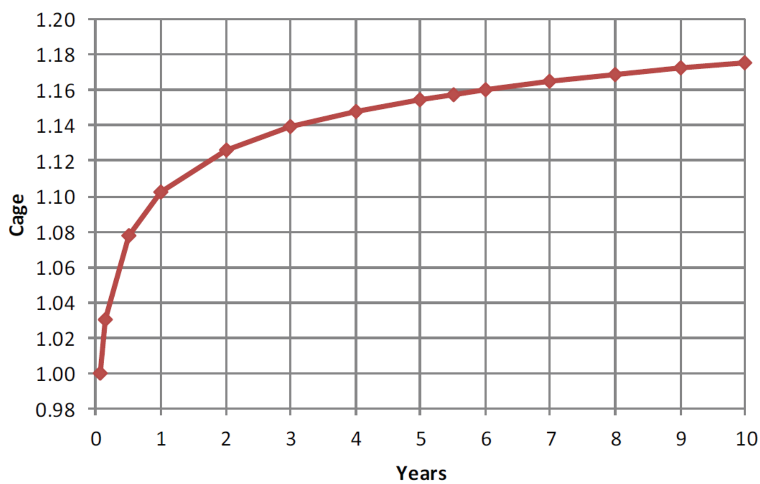

Figure 6 - Conversion Factor C_{age} vs. Age

Considering all the conversation factors, C_{conf}, C_{dist}, C_{age}, and C_{wt} the resulting shear wave velocity was obtained using Eq. (8) for each test.

$$V_{s-field} = V_{s-lab-unconfined}.C_{conf}.C_{dist}.C_{age}.C_{wt} \tag{8}$$

Eq. 8 represents the field shear wave velocity of Deep Soil Mixed Block after accounting for in-situ confinement effect (C_{conf}=1.43), laboratory sample disturbance (C_{dist}=1.20), aging and long-term curing ($C_{age}=1.05$), and the weight of building ($C_{wt}=1.17$). The cumulative conversion factor considering all four mechanisms was 2.1. The latter states the importance of conversion factors as the actual shear wave velocity could be more than twice of the laboratory measured ones.

6. CONCLUSION

This study performed the laboratory studies (bender element, sonic pulse velocity, and resonance column test) to forecast the change of shear wave velocity of deep soil mixing due to confinement, disturbance, aging, and weight of the structure over the improved base. The following main conclusion in accordance with the obtained test results can be drawn as follows:

1. The influence of confinement and vertical stress (load) from the designed structure on the soil-cement base on the propagation wave velocity was revealed and expressed by a logarithmic formulation. During the test, with the increase of confinement and vertical stress on soil-cement samples, an increase in the shear wave propagation velocity was obtained, which is expected to provide more safe and economical design of structures.
2. The current study also specified the conversion factors related to the shear wave velocity of the laboratory and field. These factors can predict the dynamic properties of deep soil mixing from the preliminary design stage of the project especially accounting for disturbance, confinement, aging, and vertical load on the deep soil mixing elements.

Symbols

V_s	: Shear Wave Velocity
$V_{s-field}$	: Field Shear Wave Velocity
V_{s-lab}	: Laboratory Shear Wave Velocity
w_t	: Total Water
c	: Cement
V_{s0}	: Unconfined Shear Wave Velocity
ρ	: Density
f_n	: Natural Frequency

H	: Sample Height
C_{conf}	: Confinement Conversion Factor
C_{dist}	: Disturbance Confinement Factor
t	: curing time
$f(t)$	: Ratio of Unconfined Compressive Strength at a time to the Unconfined Compressive Strength at 28 days
$f(c)$	: Unconfined Compressive Strength
C_{age}	: Aging Conversion Factor
C_{wt}	: Weight Conversion Factor
σ_1	: Vertical Stress
σ_3	: Confinement Stress
G	: Shear modulus
ω	: Resonance Frequency
τ	: Shear Stress
γ	: Shear Deformation
Θ	: Azimuth Angle
J	: Moment of Inertia
J_0	: Moment of Inertia of the Power Drive of the Test Facility
h	: Specimen height
m	: Specimen weight
r	: Specimen radius

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