Effect of Fly-Ash Stabilization on Stiffness Modulus Degradation of Expansive Clays

Sireesh Saride, M.ASCE[1]; and Troyee Tanu Dutta[2]

Abstract: Expansive soils undergo a considerable amount of volume change because of moisture fluctuations and cause severe damage to lightweight structures. In this study, an attempt is made to stabilize the expansive soils to improve their swelling, stiffness, and damping properties with class C fly ash. The percentage of fly ash was varied between 5 and 20% by dry weight of the soil. A series of fixed-free type resonant column tests were performed to determine the dynamic properties, including shear modulus (G) and damping ratio (D); and Poisson's ratio (ν) of untreated and fly ash treated expansive soils. Prior to the testing, the specimens were cured for 1, 7, and 28 days in a humidity chamber. It is observed that the shear modulus of expansive clay has increased and corresponding damping ratio has decreased with increase in the fly ash content. The curing time has practically negligible influence on the normalized shear modulus and damping ratio of the treated specimens. A generalized equation is proposed for the upper and lower bound normalized shear modulus degradation curves for stabilized clays. An economical design alternative for a vertically vibrating machine is suggested based on the improved dynamic soil properties of the clay. **DOI: 10.1061/(ASCE)MT.1943-5533.0001678.**

Author keywords: Expansive soils; Fly ash; Shear modulus; Damping ratio; Curing period; Resonant column test.

Introduction

Expansive soils, also referred as black cotton soils, predominantly constituted with montmorillonite clay mineral exhibit volumetric changes due to seasonal moisture fluctuations and will cause severe damage to lightly loaded structures build on them. The resulting swell–shrink behavior of expansive soils leads to a drastic variation in their strength and stiffness properties under various loading conditions and results in premature failures of structures, such as machine foundations and pavements. The problems associated with the expansive soils are prevalent worldwide. Expansive soils are one of the major natural hazards, causing at least US$15 billion of damage, annually, to various civil engineering infrastructure in the United States alone (Azam et al. 2003). Expansive soils are often treated with calcium-based stabilizers to improve their swelling and stiffness characters. Conventional stabilizers, such as lime and cement, are widely adopted to treat the expansive soils for years. Recently, low calcium based stabilizers, such as fly ash, ground granulated blast furnace slag (GGBFS), or a combination of low and high calcium stabilizers, such as cement-fly ash, lime-fly ash, and cement-GGBF, have also been promoted to stabilize the expansive soils, to economize the stabilization process, and to promote sustainability in construction industry (Jones 1958; Ouf 2001; Puppala et al. 2003). Among the low calcium-based stabilizers, fly ash is an end product, which is obtained from a flue gas of a furnace that is fueled with coal. Fly ash can supply a slew of divalent and trivalent cations (Ca^{2+}, AL^{2+}, and Fe^{3+}) under ionized conditions, which causes flocculation of dispersed clay particles to effectively stabilize expansive soils (Cokca 2001). There is a tremendous production of fly ash from thermal power plants across the world. By the financial year 2017, the estimated production of fly ash in India alone is approximately 300–400 million tons (Haque 2013). It leads to severe environmental concerns and safe disposal issues if not successfully utilized in bulk quantities in various civil engineering applications. In the present study, a class C fly ash obtained from the Neyveli Lignite Corporation's thermal power plant is used to treat a moderately expansive soil having a free swell index (FSI) of 50%. The percentage of fly ash is increased from 5 to 20% by dry weight of the soil to improve the swelling characteristics and dynamic properties of the soil.

Background

Soil Stabilization

Several research studies have demonstrated the successful utilization of low calcium based stabilizers, such as ground granulated blast furnace slag (GGBFS), fly ash, bottom ash, and pond ash to treat the expansive soils (Ferguson 1993; Cokca 2001; Puppala et al. 2003). However, fly ash is a high potential stabilizer because it generally contains a sufficient amount of calcium oxide (CaO), silicon dioxide (SiO_2), and aluminum oxide (Al_2O_3), along with other required basic mineral oxides necessary for forming pozzolanic compounds. Fly ash is classified as either class C or F based on the available CaO content present. According to the American Society for Testing Materials (ASTM C 618-15), a fly ash can be classified as class C if the CaO content about 20% and sum of SiO_2, Al_2O_3 and Fe_2O_3 is less than 70%. The addition of calcium-based stabilizer creates a high pH environment to dissolve the silica and alumina of the base soil. It also provides sufficient free calcium for long-term strength gain by pozzolanic reactions. Thompson (1966) reported that the cation exchange, flocculation, carbonation reactions, and pozzolanic reactions lead to stabilization of clayey soils when lime was added. The first two mechanisms result in

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improvement in workability and occur because of change in charges of the clay.

Ferguson (1993) employed fly ash produced from subbituminous soil to reduce the swell potential of an expansive soil and to improve the capacity of the subgrade to carry traffic loads. Nicholson and Kashyap (1993), by using high-quality fly ash to treat poor to marginal type of soil, reported an improvement in the unconfined compressive strength (UCS), increase in California bearing ratio (CBR) of more than 10 times, reduction in plasticity, and reduction in swelling of the treated soils. Cokca (2001), by treating expansive soil with both class C and F type fly ashes, observed that there is a reduction in plasticity index (PI), swell potential, and activity of the soil with increase in the percentage of stabilizer and curing periods. Edil et al. (2006), by treating soft, fine grain soil with class C and class F fly ashes, observed a significant improvement in CBR and resilient modulus (M_r) of the soil. Phanikumar and Sharma (2007) reported a reduction in free swell index, swell pressure, swell potential, and compressibility of expansive soil when treated with class C fly ash. By treating organic soils with class C and class F fly ash, Tastan et al. (2011) observed that with the increase in fly ash percentage, there is an increase in unconfined compressive strength and resilient modulus (M_r) of organic soils, but this increase is highly susceptible to compaction moisture content. Further, the engineering properties of the clay can be improved by activating the fly ash with alkali activators, such as sodium hydroxide (NaOH) and calcium carbide ($CaCO_3$) (Phetchuay et al. 2014).

Dynamic Properties of Stabilized Soils

A lot of research has been undertaken to ascertain the dynamic properties of sands as well as nonexpansive clays (Ishimoto and Iida 1937; Hardin and Black 1968; Seed and Idriss 1970; Hardin and Drnevich 1972; Kokusho et al. 1982; Dobry and Vucetic 1987; Zhang et al. 2005). However, limited studies were performed to determine the dynamic properties of untreated and treated expansive soils (Chae and Chiang 1978; Au and Chae 1980; Fahoum et al. 1996; Hoyos et al. 2004). Chae and Chiang (1978) made the first study to determine the dynamic properties of treated and untreated soils. Resonant column (RC) tests were performed on uniform sand and silty clay with cement, lime, and lime-fly ash mixes. It was observed that the dynamic properties of clayey soils can be improved through stabilization. A similar observation was made by Au and Chae (1980) on expansive soil treated with salts, lime, and lime-salt mixtures. Fahoum et al. (1996), by performing a series of cyclic triaxial tests on lime-treated sodium montmorillonite clay and calcium montmorillonite clay, observed a significant increase in shear modulus (G) and considerable reduction in damping ratio (D) with treatment. Hoyos et al. (2004) demonstrated that small-strain shear modulus ($G_{\max}$) is greatly susceptible to compaction moisture content of chemically treated sulfate rich clays in resonant column tests. By performing bender element tests on soft clay treated with fiber and cement, Fatahi et al. (2013) concluded that small-strain shear modulus ($G_{\max}$) increases with the increase in cement content. It was also mentioned that small-strain shear modulus ($G_{\max}$) decreases with the increase in carpet fibers but increases with increase in polypropylene fibers. It is very well understood that the performance of stabilized soils, especially expansive soils, is dependent on the curing time; however, the dynamic behavior of the treated expansive soils under different curing periods is not very well understood to date.

Existing literature clearly shows a gap in understanding the dynamic properties of stabilized expansive soils at varying curing intervals.

Objectives

The overall objective of the study is to determine the stiffness of fly ash stabilized expansive soils under small-strain dynamic loading conditions. The following specific objectives are designed:

1. To study the dynamic behavior of stabilized expansive soils using resonant column studies and to determine the influence of shear strain, confining pressure (σ_3), fly ash dosage and curing time on the dynamic properties including shear modulus (G) and damping ratio (D); and Poisson's ratio (ν);
2. To obtain a generalized upper and lower bound normalized degradation curves for stabilized expansive soils;
3. To visualize and quantify the mineralogical changes, which are responsible for the improved dynamic properties, in the treated expansive soils, though X-Ray Diffraction (XRD) tests; and
4. To demonstrate a design problem based on the data obtained from the present study.

Materials Used

Expansive Soil

The expansive soil used in this study is dark brownish clay obtained from the Indian Institute of Technology Hyderabad campus. The fines content (<75 μ) of the soil is approximately 70% and the clay fraction is 40%. The Atterberg limits including liquid limit (LL), plastic limit (PL), and plasticity index (PI) of the soil are obtained according to American Society for Testing and Materials standard [ASTM D 4318 (ASTM 2005)] and observed to be 58%, 20%, and 38, respectively. The soil classification based on the American Association of State Highway and Transportation Officials [AASHTO M 145-91 (AASHTO 2000)] and Unified Soil Classification System (USCS) [ASTM D 2487 (ASTM 2011)] is A-7-6 and CH, respectively. The soil is having a free swell index of 50% and can be classified as a moderately expansive soil, according to the ASTM D 2487 (ASTM 2011), Bureau of Indian Standard [IS 1498 (BIS 1970)], and Sridharan and Prakash (2000). In addition, the degree of expansiveness of the soil can be considered as high based on the LL value (Chen 1975) and medium based on the PI value (Chen 1975; Holtz and Gibbs 1956; Sridharan and Prakash 2000). The specific gravity of the soil (G_s) is 2.8. The standard Proctor's compaction test is performed on the soil, according to ASTM D 698-12e2 (ASTM 2012), and observed that the optimum moisture content (OMC) of the soil is 22% and the maximum dry unit weight is 16.51 kN/m^3 (Fig. 1).

Fly Ash

The fly ash is collected from Neyveli Lignite Corporation Limited, Tamil Nadu state of India. The specific gravity of the material is 2.16. The fines content in the fly ash is 52% with a liquid limit of 36%. An X-ray florescence (XRF) test was performed to obtain the fly ash's chemical composition. The chemical compounds, which are responsible for pozzolanic reactions, are only presented in Table 1. Because the amount of calcium oxide (CaO) is 16.5% and the sum of oxides (SiO_2, Al_2O_3, and Fe_2O_3) is less than 70%, the fly ash is classified as Class C based on ASTM C 618-15 (ASTM 2015). However, based on USCS, the fly ash is classified as silt with low compressibility (ML).

Sample Preparation

The standard Proctor's compaction tests were performed on untreated as well as fly ash treated samples prior to the preparation

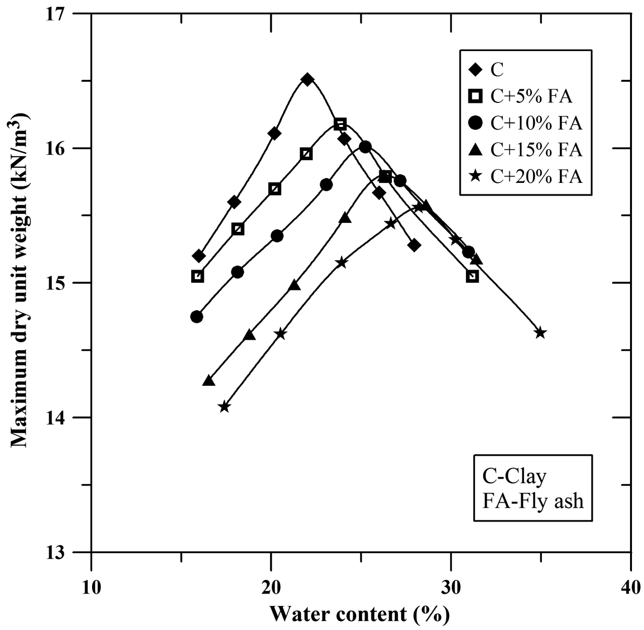

Fig. 1. Compaction characteristics of treated and untreated clay

Table 1. Chemical Composition of Fly Ash

Chemical compound	Quantity (%)
Silicon dioxide (SiO_2)	34.52
Calcium oxide (CaO)	16.45
Iron oxide (Fe_2O_3)	4.94
Aluminum oxide (Al_2O_3)	28.87
Magnesium oxide (MgO)	3.04

of soil samples for RC studies. For stabilization of expansive soil, fly ash content of 5–20% with 5% intervals was added by dry weight of the soil. Fig. 1 illustrates the Proctor compaction test data for both untreated and fly ash treated soils. It can be noticed that with an increase in the percentage of fly ash content, as expected, the maximum dry unit weight decreases and the OMC increases. The increase in OMC is due to increase in the fines content, which requires higher water content because of increased specific surface area of the soil-fly ash mixture. Because the specific gravity of the fly ash is less than the soil, the maximum dry unit weight decreases with an increase in the fly ash dosage.

The soil specimens for resonant column tests were then prepared in a constant volume mold of size 50 × 100 mm at their corresponding optimum moisture and maximum unit weights. For the preparation of untreated specimens, the dry soil was mixed properly with desired quantity of water, which was calculated from the OMC of the soil. For stabilized soils, the predetermined amount of fly ash (5, 10, 15, and 20%) was mixed initially with the dry soil, and then, the water was added according on the amount calculated from the optimum moisture content of the treated soil obtained from Fig. 1. For obtaining a uniform mixture, the mixing was continued for approximately 5 min. The samples were then compacted in a constant volume mold of size 50 mm diameter and 100 mm length to their maximum dry unit weight in three layers under a static compaction in a triaxial loading frame. The prepared specimens were then cured for 1-, 7- and 28-day periods in a humidity chamber to perform RC tests at a given curing time. At least two identical specimens were casted for all soil-fly ash mixtures to ensure repeatability of the test results.

Test Methods

Free Swell Index (FSI) Test

Free swell index (FSI) tests were performed to determine the expansion potential of expansive soil in water and kerosene. It is used to determine the expansion of soil caused by diffuse double layer repulsion and changes in the soil fabric (Holtz and Gibbs 1956). In this test, ten grams of oven-dried soil passing 425 μm was poured into graduated cylinders of 100 mL capacity. One of the cylinders is filled with distilled water and another with kerosene to 100 mL label. It was stirred by means of a glass rod to release the entrapped air from the mixture and left it to settle for 24 h. After 24 h, final volumes of the soils were read out

$$\text{Free swell index, FSI}(\%) = \frac{(V_d - V_k)}{V_k} \times 100 \tag{1}$$

where V_d = volume of the soil sample recorded in a distilled water cylinder; and V_k = volume of the soil sample recorded in kerosene cylinder.

Resonant Column Test

A fixed-free type resonant column (RC) test apparatus was used to apply torsional shear stress on a 50 mm diameter and 100 mm length specimen. Prior to the tests, specimens were cured for 1, 7, and 28 days in a humidity chamber. The RC tests were then performed according to ASTM D 4015-92 (ASTM 1993). A wide range of shear strains between 10^{-4} and 10^{-1}% and isotropic confining pressures between 25 and 200 kPa were adopted. The shear modulus, G is obtained from the backward analysis of resonant frequency and specific characteristics of the device. The damping ratio (D) is obtained using the logarithmic decrement method. The low strain shear modulus ($G_{\max}$) and damping ratio ($D_{\min}$) are directly obtained corresponding to 10^{-4}% shear strain.

Determination of the Poisson's ratio (ν) requires the resonant column test to be performed in both torsional as well as the flexural mode of excitation. Cascante et al. (1998) gave the circular resonant frequency for a soil specimen of length L by using the Rayleigh's method and considering N distributed mass m_i as

$$\omega_{\mathrm{f}}^2 = \frac{3EI_{\mathrm{b}}}{L^3\left[\frac{33}{140}m_{\mathrm{T}} + \sum_{i=1}^{N} m_i h(h0_i, h1_i)\right]} \tag{2}$$

$$h(h0_i, h1_i) = m_i\left[1 + 3\frac{(h1_i + h0_i)}{2L} + \frac{3}{4}\left(\frac{h1_i^2 + h1_i h0_i + h0_i^2}{L}\right)^2\right] \tag{3}$$

where $h0_i$ and $h1_i$ = heights of mass i, measured from the top of the soil specimen, respectively; E = Young's modulus of the soil specimen; I_b = area moment of inertia of the specimen; and m_T = mass of the soil specimen.

Eq. (3) can also be presented in terms of center of gravity y_{ci} and area moment of inertia, with respect to center of gravity I_{yi} of each mass m_i

$$h(y_{ci}, I_{yi}) = 1 + \frac{3y_{ci}}{L} + \frac{9}{4L^2}\left[\frac{I_{yi}}{m_i} + y_{ci}^2\right] \tag{4}$$

Because of complex geometry, the area moment of inertia I_y for the drive system is determined experimentally.

Now, the Poisson's ratio is determined using

$$\nu = \frac{1}{2}\frac{V_{LF}^2}{V_s^2} - 1 \tag{5}$$

where V_{LF} = longitudinal wave velocity which can be calculated using Eq. (6).

V_s = shear wave velocity

$$V_{LF} = \sqrt{\frac{E}{\rho}} \tag{6}$$

where E = Young's modulus of the soil specimen determined using Eq. (2); and ρ = density of the soil specimen.

X-Ray Diffraction (XRD) Studies

X-ray diffraction studies were carried on both untreated and treated expansive soils to identify the hydrated compounds present in the mixes. In the XRD test, powdered specimens are subjected to the intense X-ray beam, and the diffracted beam is detected with the help of a detector. The analog data is then converted to digital data by the detector and can be plotted for getting the peaks of different compounds. In the present study, XRD data were obtained by using PANalytical X'Pert Pro MPD diffractometer in a θ–θ configuration employing C_uK_α radiation ($\lambda = 1.54$ A°) with a fixed divergence slit size 0.5° and a rotating sample stage. The powdered specimens were placed in a sample holder. By using an X'Celerator detector, specimens were then scanned between 5 and 100°. PCPDFWIN software was used to analyze the peaks and identify the minerals present in the specimen.

Results and Discussion

Initially, the influence of fly ash dosage on the index properties and swelling characteristics of untreated and fly ash treated expansive soil specimens is discussed. The influence of fly ash dosage and curing intervals on the dynamic properties of stabilized soils is discussed, and the improvement in dynamic soil properties is supported with a series of mineralogical studies. Finally, an economical and vibration-free design of a machine foundation, resting on a stabilized expansive soil, subjected to vertical vibrations is demonstrated.

Effect of Fly Ash on Atterberg Limits

Table 2 presents the variation of Atterberg limits with an increase in the fly ash content. It is seen that the liquid and plastic limits (LL and PL) of the soil increase, whereas plasticity index (PI) of the soil decreases with an increase in the fly ash content. The increase in liquid limit and plastic limit is because of flocculation and the conglomeration of the clay particles and binding properties of fly ash, which increases the hydrophilic nature and, in turn, increases the liquid limit and plastic limit of the soil. However, the increase in the plastic limit is greater than the liquid limit, causing a corresponding decrease in the PI of the soil. This confirms the fact that the decrease in PI is not due to the reduction of liquid limit, but due to the increase in the plastic limit of the soil. Furthermore, based on the PI, the degree of expansion of the treated soils for fly ash content ≥15% can now be considered as low (Chen 1975; Holtz and Gibbs 1956).

Table 2. Atterberg Limits and Free Swell Index of Untreated and Fly Ash Treated Soils

FA (%)	LL (%)	PL (%)	PI (%)	FSI (%)	Percent reduction in FSI
0	58	20	38	50	—
5	60	28	32	45	10
10	63	36	27	30	40
15	66	48	18	20	60
20	68	58	10	10	80

Effect of Fly Ash on Free Swell Index

The FSI tests were conducted on untreated and treated samples at room temperature. Table 2 presents the variation of FSI of the soil with an increase in the dosage of fly ash. It is observed that there is a considerable reduction in the FSI of the soil, with increase in the percentage of fly ash. A 30% reduction in FSI is noticed when fly ash dosage increased from 5 to 10%; thereafter, a constant 20% decrease in FSI is recorded for every 5% increase in fly ash content. As high as an 80% reduction in FSI is observed at 20% fly ash content. A further increase in the fly ash content would nullify the FSI; however, higher fly ash dosages are not considered in the present study because the required pH concentration of 12.4 was obtained at a fly ash content of 20%. The reduction in FSI is because of the abundant supply of multivalent cations (e.g., Ca^{2+}, Al^{3+}, and Fe^{3+}) from Class C fly ash, which causes flocculation of clay particles by cation exchange. Because of flocculation, the specific surface area and water affinity are greatly reduced, which cause a reduction in the free swell index of the treated soils. Based on the FSI, the degree of expansiveness of the treated soils can be considered as low according to ASTM D 2487 (ASTM 2011) and IS 1498 (BIS 1970).

Effect of Fly Ash and Curing Time on Dynamic Properties of Soil

Fig. 2 depicts the normalized shear modulus degradation ($G/G_{\max}$) curves of fly ash treated specimens cured for 28 days subjected to 25, 100, and 200 kPa confining pressures. It is observed that the ($G/G_{\max}$) value decreases from 1 to 0.1 with an increase in the shear strain from 10^{-4} to 10^{-1}%, respectively. The $G/G_{\max}$ is gradual from 10^{-4} to 10^{-3}% shear strain and significant thereafter. This is because of the high stiffness offered by the treated specimens against shear at low strain range; however, at higher shear strain levels, there will be a particle–particle slippage, and breakage of pozzolanic bonds lead to a high degradation of shear modulus. It is interesting to note that the degradation of shear stiffness is higher for higher dosages of fly ash because the specimens treated with a high dosage of stabilizer exhibit higher initial shear stiffness. The cementation effect of fly ash caused by the pozzolanic reactions between the soil and the fly ash provides a confinement effect at the clay particle–particle interfaces. This causes higher rigidity of the treated specimen resulting in higher shear stiffness. The formation of pozzolanic compounds can be visualized in mineralogical studies, which are discussed in subsequent sections. Similar observations were made by D'Onofrio and Penna (2003) and Delfosse-Ribay et al. (2004) in lime-treated silty sands and grouted sands, respectively. Fig. 3 depicts the influence of the curing interval on the normalized shear modulus degradation of 20% fly ash treated specimens at different confining pressures. It is evident that the influence of the curing interval on the modulus degradation ($G/G_{\max}$) of the treated soil at different confining pressures is practically negligible, although there is an increase in the shear modulus with curing interval. This is because of the uniform variation in

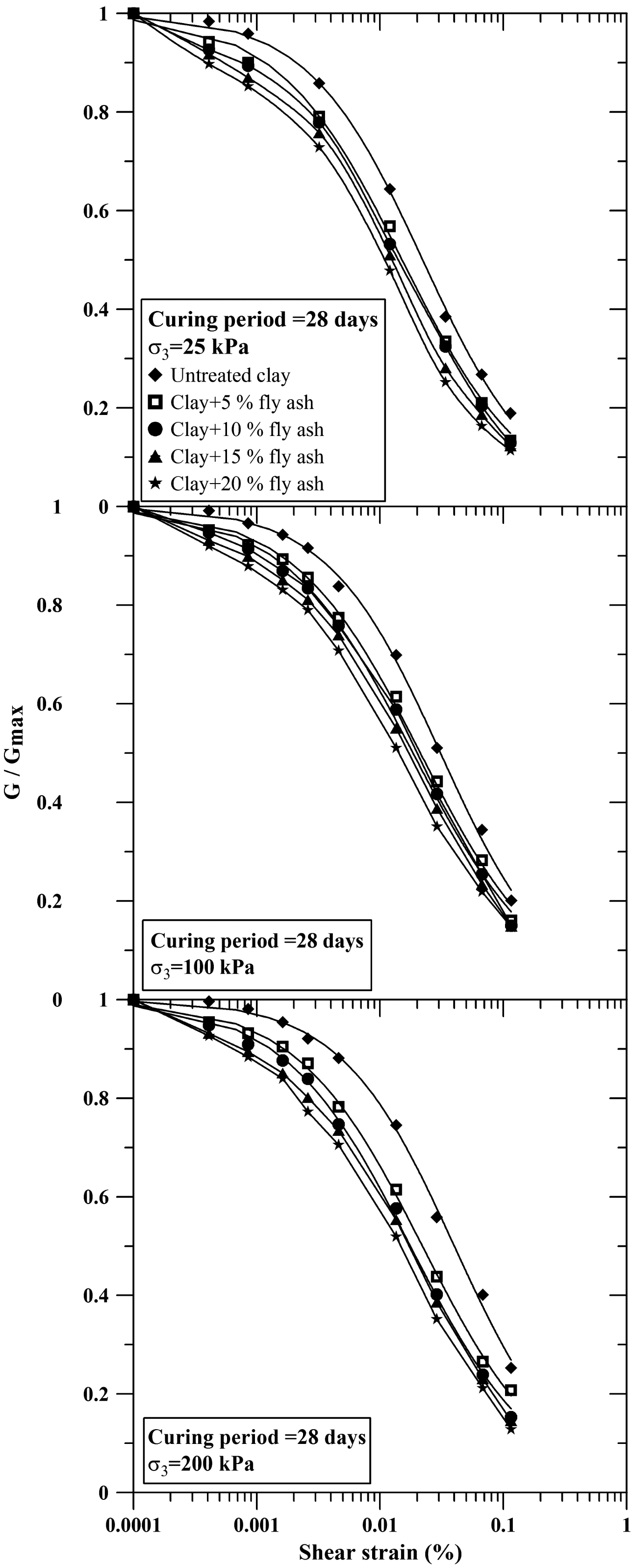

Fig. 2. Variation of modulus degradation (G/G_{max}) with shear strain for a 28-day curing period

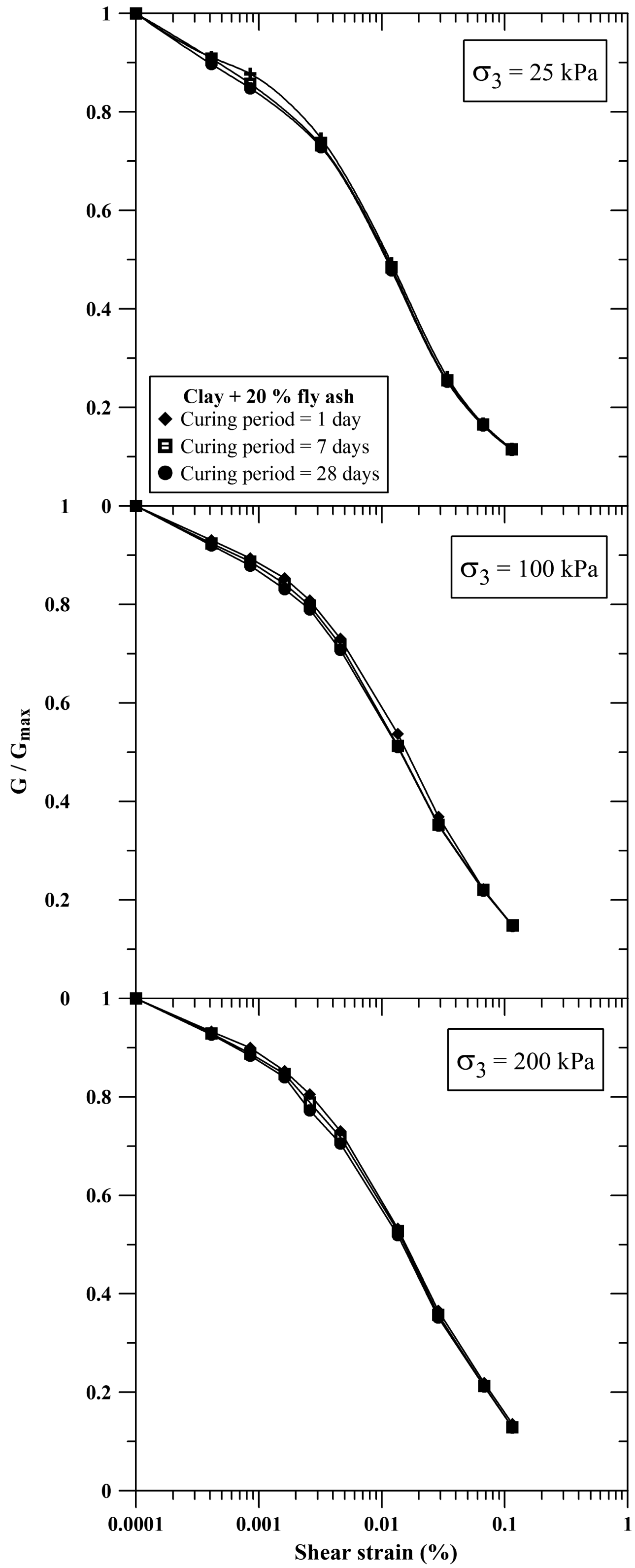

Fig. 3. Variation of modulus degradation (G/G_{max}) with shear strain for different curing periods

shear modulus (G) of the treated specimens at different curing periods. Similar observations were made at other dosages of the fly ash.

The present study has generated extensive normalized modulus degradation data from a series of experiments on the fly ash stabilized expansive clay specimens at different fly ash dosages, curing periods, and isotropic confining pressures. The data sets are collectively used to obtain a range of G/G_{max} values for different shear strain values to develop lower and upper bound modulus degradation curves for the treated clays, as presented in Fig. 4.

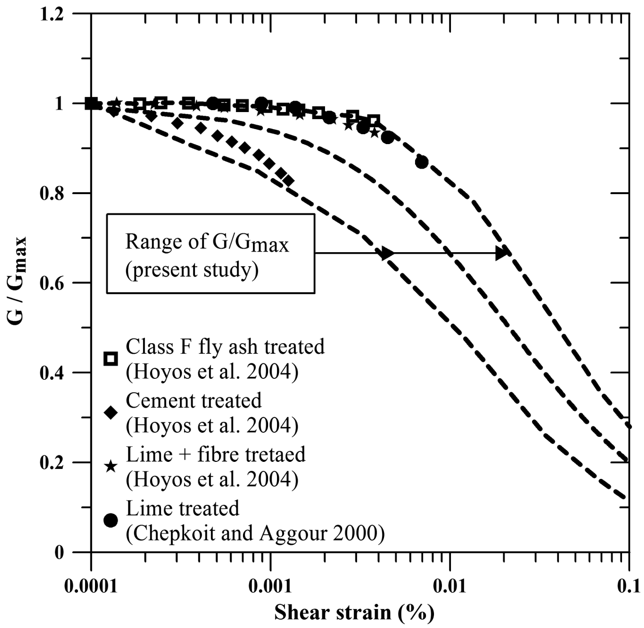

Fig. 4. Validation of normalized modulus degradation (G/G_{max}) upper and lower bound curves

The upper bound represents the normalized modulus degradation values corresponds to untreated clays and clays treated with low-calcium stabilizers subjected to high confining pressures. The lower bound represents the modulus degradation values of clays stabilized with high stabilizer content and subjected to low confining pressures. Fig. 4 presents the range of modulus degradation (G/G_{max}) with shear strain for all the fly ash treated expansive clay specimens. Generalized upper and lower bound modulus degradation curves can be developed from the present data because the behavior of the expansive clay has been nullified with the stabilization process.

A generalized equation for the upper and lower bound (G/G_{max}) curves for the present test data is given below

$$\frac{G}{G_{max}} = A\left(1 + \frac{B\bar{\gamma}}{C}\right)^{(-1/B)} \tag{7}$$

where A, B, C = fitting parameters; and $\bar{\gamma}$ = shear strain.

The fitting parameters for the upper and lower bound curves with a high regression coefficient, R^2 of approximately 0.99, are given in Table 3.

The proposed upper and lower bounds of the normalized modulus degradation curves are validated with the available data from the literature on expansive and high PI clays treated with various stabilizers (Hoyos et al. 2004; Chepkoit and Aggour 2000). Hoyos et al. (2004) have performed low strain RC tests on sulfate-rich expansive soils treated with various stabilizers to study the dynamic properties of the soil. The expansive clay was treated with 20% class F fly ash, 5–10% type V sulfate resistant cement, and 8% lime with 0.3% fiber. The specimens were cured for 7 days, and resonant column tests were performed between low to medium shear strain intervals (10^{-4} to 10^{-2}%) and 17.25 to 138 kPa confining pressures. The G/G_{max} values were calculated from their study and presented in the Fig. 6 to validate the proposed bounds. It can be seen that the G/G_{max} values of class F fly ash and lime with fiber treated specimens plot towards the upper bound curve. This may be because of the low CaO content (1.1%) present in the class F fly ash used in their study, which might have produced marginal pozzolanic reactions in sulfate rich expansive soil specimens during 7 days curing time. The G/G_{max} of 10% cement treated specimens plot towards the lower bound curve. High modulus degradation is expected in these specimens because the cement content is high. In another study, Chepkoit and Aggour (2000) have performed resonant column studies to determine the dynamic properties of lime stabilized (8%) high PI clays. The samples were compacted to their respective OMCs, cured in a humidity chamber, and then kept in an oven maintained at 105°C for 65 h before testing at 1–210 kPa isotropic confining pressures. It can be seen that the modulus degradation data of the lime-treated clay specimens plots along the upper bound curve (Fig. 4). The initial low-strain shear modulus (G_{max}) is expected to be low in these specimens because of the aggressive curing conditions maintained in their study. This might be the reason for lime-treated specimens to plot towards the upper bound curve.

Table 3. Fitting Parameters for the Upper and Lower Bound Curves from the Present Study

Parameter	A	B	C	R^2
Upper bound	1.014	0.045	0.841	0.99
Lower bound	0.965	0.009	1.466	0.99

The normalized modulus degradation data obtained from high PI clays; sulfate-rich, expansive clays; moderate expansive clays stabilized with lime, with and without fibers; and cement and class C and class F fly ash fall within the proposed upper and lower bound curves. It is also evident now that the stabilized clays behave very similarly under dynamic loading conditions, although their behavior under static loading conditions may differ. Hence, the proposed curves may be generalized and can be used to estimate the normalized modulus degradation (G/G_{max}) data of any type of stabilized clays.

Fig. 5 presents the variation of the damping ratio (D) with shear strain for 25, 100, and 200 kPa confining pressures for 28-day cured specimens. The damping ratio is a measure of the dissipation of energy during the cyclic loading. The higher the degree of particle slippage and particle rearrangement, the higher is the damping ratio of the soil (Fahoum et al. 1996). There is an increase in the damping ratio with increase in the shear strain. The increase is gradual from 10^{-4} to 10^{-3}% and observed to be significant thereafter. With increase in the strain level, there are greater chances of particle slippage and rearrangement, which results in a higher value of damping ratio. It is also observed that there is a slight reduction in damping ratio (D) with increase in the fly ash content. Addition of fly ash increases the rigidity of the specimen, which causes the reduced particle slippage and particle rearrangement resulting in a lower damping ratio. Similar observations were made by Acar and El-Tahir (1986), Dobry and Vucetic (1987), and Fahoum et al. (1996). Fig. 6 gives the variation of damping ratio (D) with shear strain for 25, 100, and 200 kPa confining pressures for different curing periods for 20% fly ash treated specimens. Curing interval has practically no effect on the damping ratio and can be neglected for any design purposes. This can be attributed to the stabilizing effect of the fly ash on the expansive clay. Similar observations were made for specimens prepared at other dosages of fly ash as well.

Fig. 7 exhibits the variation of Poisson's ratio (ν) with shear strain and confining pressures for 28-day cured specimens. It is seen that with increase in the shear strain, the Poisson's ratio of the untreated and treated specimens increases. The increase is marginal up to a shear strain of 9×10^{-4}%, thereafter, a gradual increase in Poisson's ratio of the soils is observed. It is also noted that the Poisson's ratio decreases with an increase in the fly ash

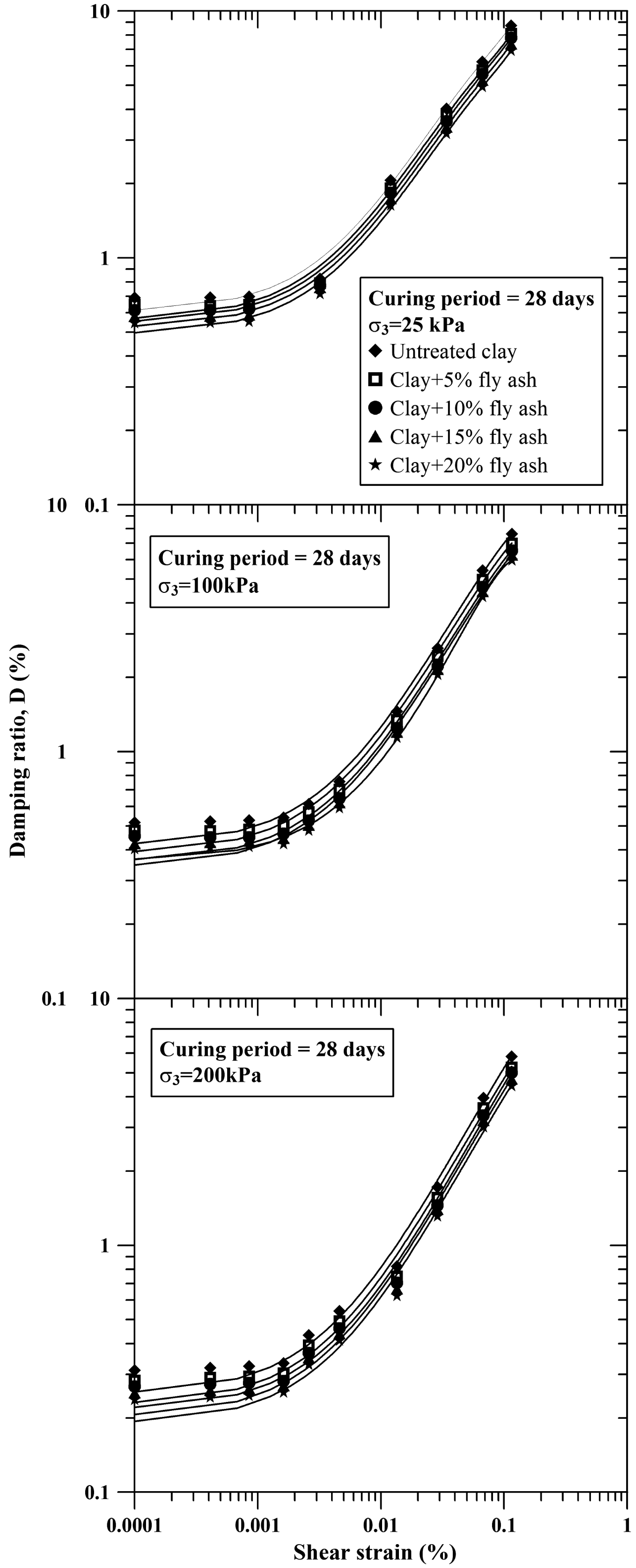

Fig. 5. Variation of damping ratio (D) with shear strain for a 28-day curing period

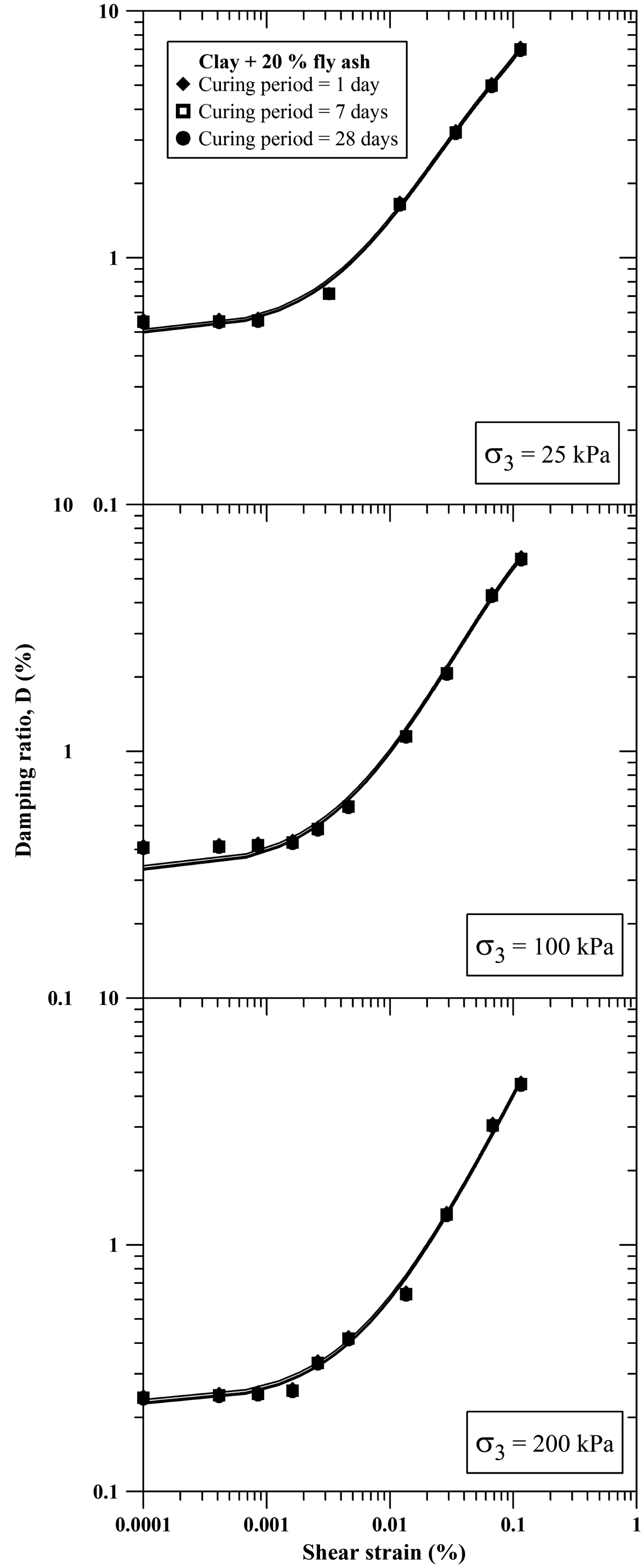

Fig. 6. Variation of damping ratio (D) with shear strain for different curing periods

dosage. Due to increase in the fly ash content, the stiffness of the soil increases, which gives higher resistance to the specimen deformation that leads to the decrease in the Poisson's ratio. Fig. 8 illustrates the influence of curing interval on the variation of Poisson's ratio (ν). There is a slight reduction in Poisson's ratio with increase in the curing period, unlike on the normalized shear modulus degradation (G/G_{max}) and damping ratios, where the influence of curing interval is negligible. Similar observations were made at other dosages of fly ash.

Fig. 9 presents the variation of G/G_{max} with the Poisson's ratio of fly ash treated expansive soil at different fly ash contents, confining pressures, and curing periods. A wide band of Poisson's ratio

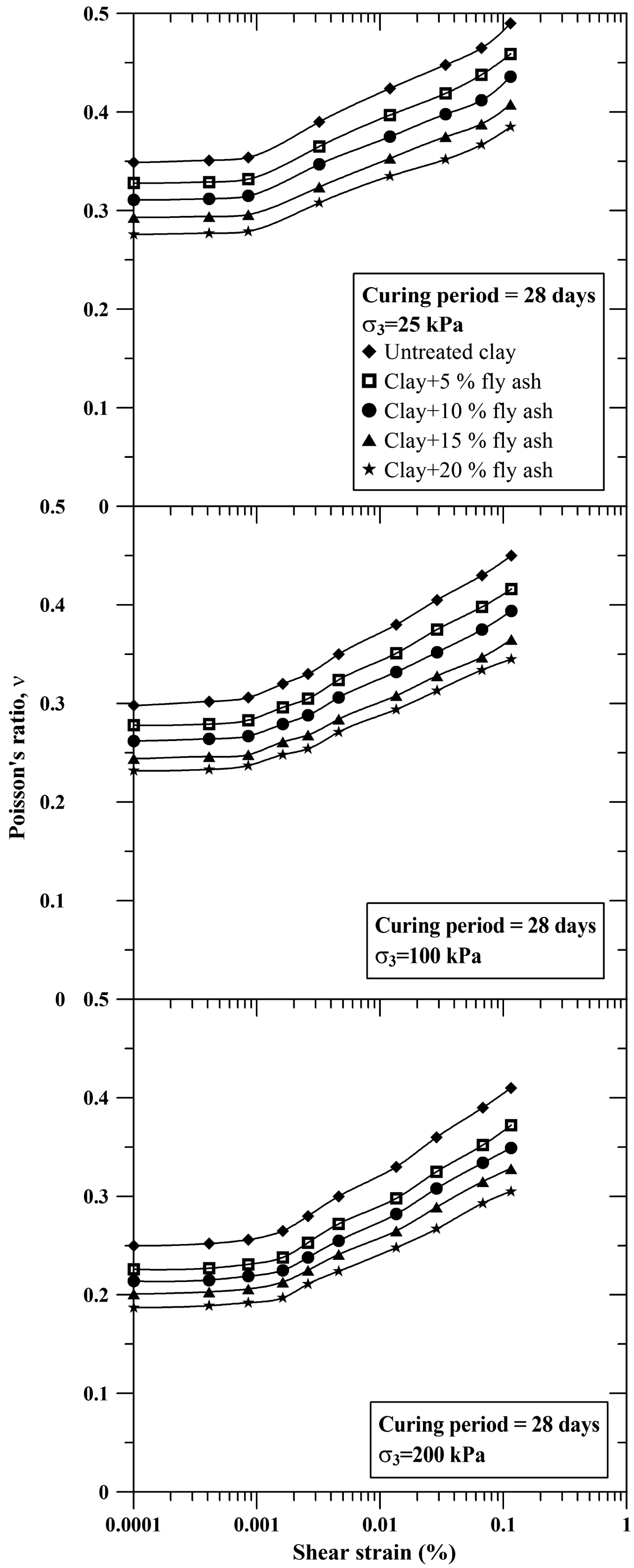

Fig. 7. Variation of Poisson's ratio (ν) with shear strain for a 28-day curing period

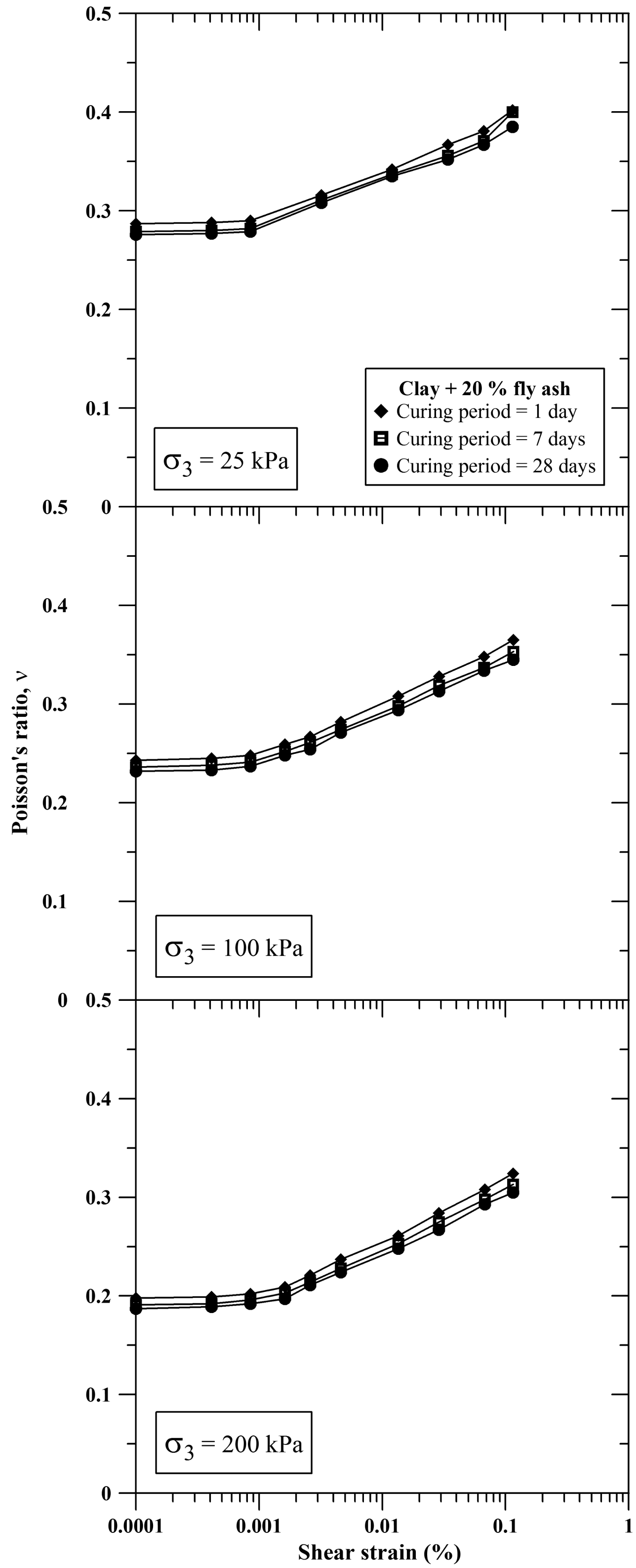

Fig. 8. Variation of Poisson's ratio (ν) with shear strain for different curing periods

can be noticeable for a given shear modulus degradation ratio. A general increasing trend of the Poisson's ratio with a decrease in the shear modulus degradation ($G/G_{\max}$) ratio can be noticed. The Poisson's ratio varies between a range of 0.20–0.35 for low-strain (10^{-4}%) shear modulus ratio to a range of 0.30–0.48 for high-strain (10^{-1}%) shear modulus ratio. It can be further observed that at lower values of confining pressures, fly ash contents, and curing periods, the Poisson's ratio scatters towards the upper values. However, for higher values of confining pressures, fly ash contents, and curing periods, the values are closer to the lower side. Fig. 9 may be used to select the Poisson's ratio for a given range of confining pressures, stabilizer dosage, and curing intervals.

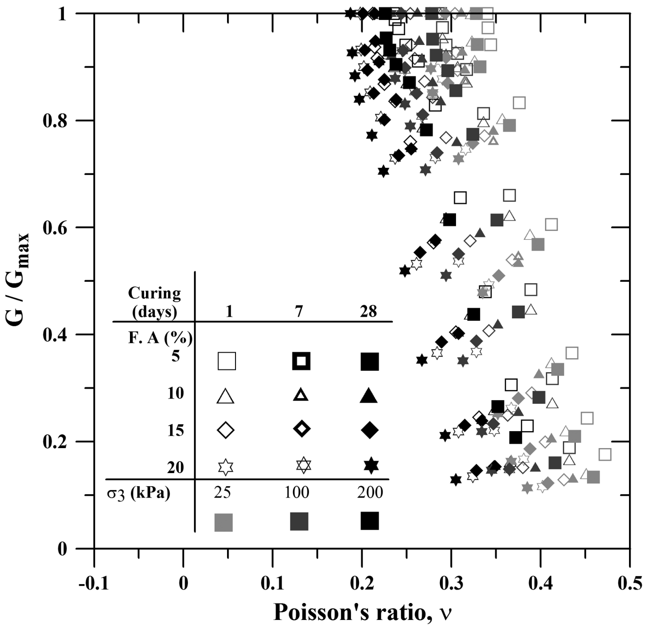

Fig. 9. Variation of Poisson's ratio with normalized shear modulus

Fig. 10(a) presents the variation of small-strain shear modulus (G_{max}) with an increase in the confining pressure for 28-day cured specimens prepared at different fly ash dosages. It is observed that the small-strain shear modulus (G_{max}) gradually increases with the increase in the confining pressure as well as an increase in the fly ash percentage. Fig. 10(b) gives the variation of the small-strain damping ratio (D_{min}) with confining pressure for different dosages of fly ash treated specimens cured for 28 days. The small-strain damping ratio (D_{min}) is determined at a shear strain of 10^{-4}%. It is observed that the damping ratio decreases rapidly with an increase in the confining pressure as well as fly ash content. However, at higher confining pressure (200 kPa), the reduction in damping ratio is minimal. It is well understood that the highly compacted and cemented specimens exhibit low damping properties. Fig. 10(c) presents the variation of small-strain Poisson's ratio (ν_{min}) with an increase in confining pressure for different dosages of fly ash for the 28-day curing period. The small-strain Poisson's ratio (ν_{min}) is also determined at shear strain of 10^{-4}%. There is a reduction in small-strain Poisson's ratio (ν_{min}) with an increase in the confining pressure as well as fly ash content. The reduction in Poisson's ratio is from 0.35 to 0.25 and 0.28 to 0.19 for 5 and 20% fly ash treated specimens, respectively.

Overall, the increase in small-strain shear modulus (G_{max}) and reduction of damping ratio (D_{min}) and Poisson's ratio (ν_{min}) with the increase in confining pressure is due to increased number of particle–particle bonds with increase in the confining pressure, which provides higher resistance to the specimen against deformation (Mitchell 1976).

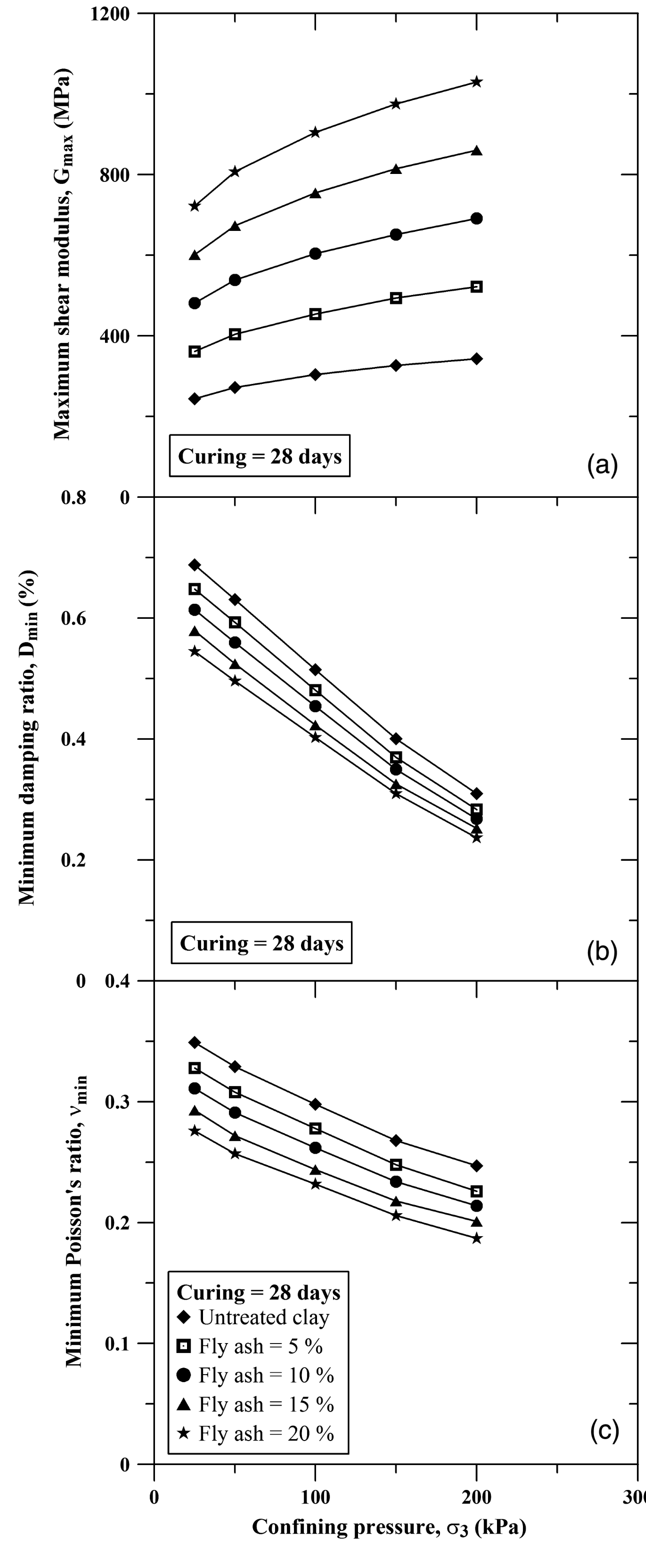

Fig. 10. Variation of small-strain dynamic properties with confining pressure: (a) shear modulus (G_{max}); (b) damping ratio (D_{min}); (c) Poisson's ratio (ν_{min})

X-Ray Diffraction (XRD) Analysis

Fig. 11 exhibits the XRD results of untreated and treated samples (fly ash content of 10 and 20%) cured for 28 days. The presence of montmorillonite clay minerals confirms the expansive nature of the soil. A notable change is not observed in the peaks of montmorillonite minerals with treatment. However, there are additional formation of other hydration compounds because of stabilization. When the specimen is treated with 10% fly ash, a sufficient number of calcium hydroxide [$Ca(OH)_2$] peaks are observed. The $Ca(OH)_2$ is a hydration product, which is formed because of a reaction of calcium oxide (CaO) of fly ash with water (H_2O). Calcium hydroxide further reacts with silica present in the soil to form cementitious products. Peaks of calcium silicate hydrate (C─S─H) compounds are also observed; however, the number of peaks obtained is less for the samples treated with 10% fly ash. When the specimens are

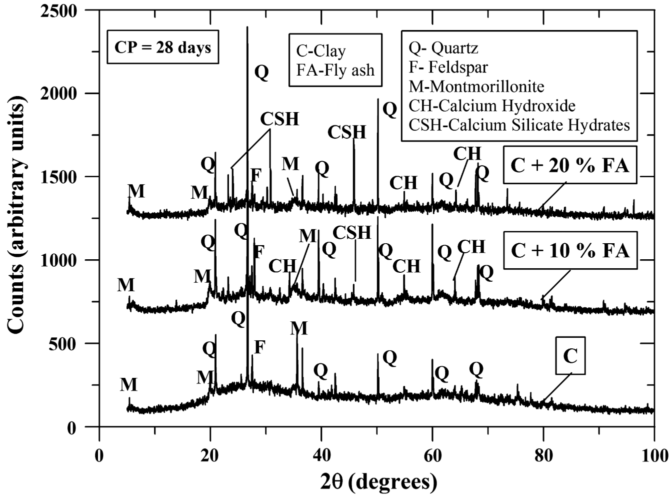

Fig. 11. XRD results of samples for a 28-day curing period

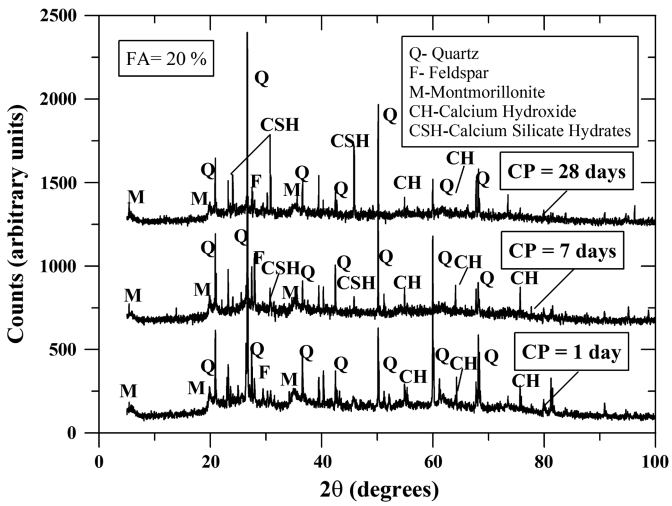

Fig. 12. XRD results of samples treated with 20% fly ash for different curing periods

treated with 20% fly ash, higher numbers of C─S─H peaks are observed. This is because with the increase in fly ash dosage, the pH of the specimen increases, which helps in the formation of C─S─H compounds. Unreacted calcium hydroxide $[Ca(OH)_2]$ compounds can also be found in the specimens treated with lower dosages of fly ash and can be effectively utilized by the addition of the alkalies, like sodium hydroxide or calcium carbide, to the mix (Phetchuay et al. 2014). Fig. 12 gives the XRD results of samples treated with 20% fly ash for different curing periods. It is observed that no C─S─H compounds are found in the samples cured for 1 day. At higher curing periods, peaks of C─S─H compounds are observed. This proves that the formation of pozzolanic compounds (C─S─H) is time dependent and it gives long-term strength to the specimens.

Design Application

It is evident that the adverse effects of the expansive soils can be controlled with various stabilization techniques. It is noticed that the dynamic properties of expansive soils can be improved with appropriate stabilizers. To visualize the influence of stabilization on the design of machine foundations subjected to vertical vibrations, a design example is demonstrated. It is anticipated that the resonance of the soil-foundation system can be controlled by stabilizing the soil. This will lead to an economical design of machine foundations.

Lysmer and Richart's (1966) elastic half space method has been adopted to design a typical machine foundation subjected to vertical vibrations. The main assumptions of this method are that the soil is homogeneous, isotropic, and elastic with shear modulus (G) and Poisson's ratio (ν). It should be noted that the damping automatically enters into the solution of elastic half space method. In this method, the following steps are adopted for the design of a foundation:

1. Calculation of equivalent radius from the length and width of the foundation

$$r_o = \sqrt{\frac{BL}{\pi}} \tag{8}$$

where r_o = equivalent radius; L = length of the foundation; and B = width of the foundation.

2. Calculation of mass ratio from the weight of the machinery and foundation block, unit weight, and Poisson's ratio of the soil

$$B_Z = \left(\frac{1-\nu}{4}\right)\left(\frac{W}{\gamma r_o^3}\right) \tag{9}$$

where B_z = mass ratio; W = weight of the machine and the foundation; ν = Poisson's ratio of the soil; γ = unit weight of the soil; and r_o = equivalent radius.

3. Calculation of resonant frequency of the foundation-soil system from the shear modulus of the soil, unit weight of the soil, equivalent radius, and mass ratio

$$f_{mr} = \left[\left(\frac{1}{2\pi}\right)\sqrt{\left(\frac{4Gr_o}{1-\nu}\right)\frac{\gamma}{W}}\right]\sqrt{1-2\left(\frac{0.425}{\sqrt{B_Z}}\right)^2} \tag{10}$$

4. Check for no resonance condition

$$\text{Factor of safety} = f_{mr}/f_o > 2 \tag{11}$$

where f_o = operating frequency of the machine.

If the previous condition is not satisfied, one of the following alternatives can be adopted:

1. Increase the size of the footing; or
2. Improve the strength of the soil by means of stabilization; or/and
3. Provide an appropriate damping system.

The following design problem is analyzed to show the application of the data obtained from the present study (stabilization of the expansive soil). The weight coming from the foundation and the machine is assumed as 800 kN. The operating frequency of the machine is assumed as 1,500 cpm. A rectangular footing of size 4×3 m is considered. The strains coming to the soil from a machine foundation are usually in the range of 10^{-4} to 10^{-3}% (Chowdhury and Dasgupta 2008). Hence, for conservative design, a strain value of 8×10^{-3}% is considered. The shear modulus and Poisson's ratio values of the soil are considered corresponding to 8×10^{-3}% strain and 25 kPa of confining pressure. The 25 kPa confining pressure represents a typical foundation depth of 1.5 m. Table 4 gives the factor of safety obtained for different dosage of fly ash corresponding to the 28-day curing period. It is observed that the factor of safety of the soil-foundation system has increased gradually with fly ash content to reach a limiting value of factor of safety of two at a fly ash percentage of 15%. It shows

Table 4. Factor of Safety of the Machine Foundation

Parameter	Untreated soil	5% FA	10% FA	15% FA	20% FA
Shear modulus (MPa)	233.85	325.15	420.48	535.65	612.48
Poisson's ratio	0.354	0.332	0.315	0.296	0.279
Equivalent radius (m)	1.95	1.95	1.95	1.95	1.95
Mass ratio	1.03	1.08	1.13	1.17	1.22
Resonant frequency of foundation-soil system (cpm)	2,114.02	2,442.32	2,735.98	3,038.56	3,192.67
Factor of safety	1.41	1.62	1.82	**2.02**	**2.13**

Note: Bold values represent the final results.

that for a safe, economical, and sustainable design of machine foundations, an appropriate soil stabilization method can be adopted.

Conclusions

Based on the fly ash stabilization of moderately expansive soil, the following major conclusions can be drawn from the current research:

1. With increase in dosage of fly ash, the optimum moisture content increases, and the maximum dry density decreases. The increase in optimum moisture content is because of the increase of fines content and corresponding increase in specific surface area. The presence of fly ash, which has a lesser specific gravity, causes the reduction in the maximum dry density.
2. The liquid limit and plastic limit of the expansive soil increase, whereas the plasticity index of the soil decreases with an increase in the fly ash content. The increase in liquid limit and plastic limit is because of flocculation and the conglomeration of the clay particles, which increases the water holding capacity and hence the liquid limit and plastic limit of the soil. However, the increase in plastic limit is greater than the liquid limit, causing a corresponding decrease in the plasticity index of the soil.
3. A considerable reduction in the free swell index of the expansive soil is observed with an increase in the dosage of fly ash. Because of flocculation, the specific area and water affinity are greatly reduced, which causes reduction in the free swell index of treated soils.
4. With an increase in fly ash dosage, the normalized modulus degradation ($G/G_{\max}$) ratio decreases. Generalized upper and lower bound modulus degradation curves have been proposed to estimate the $G/G_{\max}$ of any treated clayey soils.
5. With an increase in the confining pressure and the fly content, the damping ratio (D) decreases. As the confining pressure and the degree of cementation of the soil increases, the soil becomes more rigid, and particle slippage and particle rearrangement considerably reduce, resulting in a decreased damping ratio of the soil.
6. Curing has practically no influence on the normalized shear modulus degradation and damping ratios of the treated specimens.
7. There is a reduction in Poisson's ratio (ν) with an increase in the confining pressure as well as fly content of the treated soils. With the increase in confining pressure and fly ash content, the stiffness of the specimen increases, which gives higher resistance to the specimen deformation.

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