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DYNAMIC PROPERTIES OF STABILIZED SUBGRADE CLAY SOIL

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ABSTRACT

Expansive soils are fine-grained soils that can undergo a significant volume change due to the variation of water content. A change in the water content of weak subgrade material under the roadway is a cause for the damage of the road pavement. During an earthquake, the soft clay will lose the small shear strength it possesses which causes cracks and movements of the road. In the present study, soil samples were collected from Banda, Uttar Pradesh, India. The study area is at risk for seismic damage due to Himalayan frontal fault earthquakes. The soils samples are highly expansive in nature and used in road subgrade. The clay soil is treated with Rice Husk Ash and Portland cement slag. Strain control cyclic triaxial tests were carried out on the stabilized clay for different amplitudes of shear strain at frequency of 0.5Hz and 100kPa effective confining pressure. The damping ratios increase with increase in amplitude of shear strain and vary from 7-9% to 14-19% for γ= 0.4%to 1% respectively. The shear modulus decreases with increase in amplitude of strain. The modulus of degradation index decreases at a very fast rate for the first 50 cycles.

INTRODUCTION

Expansive soil has swelling and shrinkage nature and because of this nature it is problematic to civil engineering structures or construction. Each year, there are many billions of dollars in damages to civil engineering structures due to the problematic characteristics of expansive soil in many parts of the world, including USA, UK, Africa, India, etc.(Gourely et.al.1993;Sabat and Nanda 2011).

The behavior of clay soil is changing from place to place, depending on topography, weather conditions and in one site it dramatically changes from one season to the next due to the moisture content. Because of these factors, there is no simple solution to solve the problems related to expansive clay.

The swelling /shrinkage behavior of the clay soil decreases and its strength, durability and stuffiness increases when the soil undergoes chemical stabilization (Sherwood 1995; Prosniski and Bhattacharaya 1999).

Analysis of static loading behavior for the given site depends on field tests and laboratory tests and needs the deep technical knowledge of Geotechnical Engineers. However, another challenge for geotechnical engineers are the inevitable circumstances like earthquakes, blasting, machine vibration, pavement under heavy traffic loading etc.(Thammathiwat, A. and Chim-oye, W.2004) These occurrences are not predictable and studying these dynamic properties is not an easy task.

Some researchers have been studied the effect of cyclic frequency and strain amplitude on dynamic properties of soils.(Zhou et al.2001;Teachvorasinskun et al.2002;Jiang et al.2010) and chemically mixed soils (Hoyos et al.2004; Shafiee et al.2004)

For the last two decades, the seismic hazards in India have caused loss of life and damage of properties with different magnitudes of earthquake in different states such as Uttarkashi (M 7.0,1991),Jabalpur (M 5.8,1997),Chamoli(M 6.5,1999),Bhuj (M 7.6,2001) and Uttar Pradesh (M4.6,2008)(Hanumantharo,C.and Ramanaa,G.V 2008), and in addition to that, more than 20% of the total soil in India is expansive soil.

In the present study, clay soils were collected from Banda, Uttar Pradesh, India, where there is a threat of seismic damage due to Himalayan frontal fault earthquakes. The soil was mixed with locally available Rice-Husk Ash and Portland cement slag for different mix proportions and the optimum content (82.5% Soil-7.5 % Portland slag cement – 10% Rice Husk Ash) was chosen for static as well as cyclic tests. The variation of dynamic properties of the stabilized clay soil such as shear modulus, G, and damping ratio, D, and degradation index, δ,with number of cycles for different strain amplitude of γ= 0.4% to 1% at frequency of 0.5Hz and effective confining pressure of 100kPa have been studied.

MATERIALS

Soil

The expansive subgrade clay soil was collected from Banda Site, Uttar Prades, India. The laboratory test was carried out and the soil was classified as highly expansive soil (CH) according to Unified Soil Classification system (USCS) and A-7-6 according to American Association of State Highway and Transportation Officials' (AASHTO) classification system. The basic geotechnical properties of soil are given in Table 1. The grain size distribution of the clay soil is shown in Fig.1.

Table 1.Geotechnical properties of clay soil

Soil properties	**Values**
Grain size distribution	
Gravel (>4.75mm),%	0
Sand(4.75mm to 0.075mm),%	1.4
Silt(0.075mm to 0.002mm),%	55.4
Clay(<0.002mm),%	43.2
Specific Gravity	2.69
Atterberg Limit	
Liquid Limit (%)	53.4
Plastic Limit (%)	21.3
Plasticity Index (%)	32.1
Shrinkage Limit	11
Proctor compaction	
Maximum Dry Density(KN/m^3)	16.05
Optimum Moisture content (%)	21.25
Soil classification	
Percentage Passing (%) #200	98.6

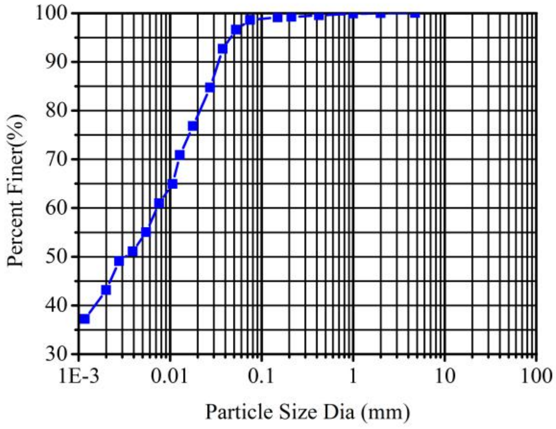

Fig.1. Grain size distribution curve for clay soil

Rice Husk Ash

The Rice Husk Ash(RHA) was collected from Kolkatta, India and was product of Rice Husk milled at circulating fluidized bed boiler at 470^0c temperature and Lancashirebiller at 67^0c.The RHA is defined as a pozzlanic material (ASTM C618-2012) due to its high amorphous silica content(81.88%) (Mehta 1986).The compositions of Rice Husk Ash are given in Table 2.

Table 2. Composition of Rice Husk Ash

Oxides	**Value (%)**
SiO_2	81.88
C	4.1
K_2O	1.3
Al_2O_3	0.88
CaO	0.82
MgO	0.48
Fe_2O_3	0.46
Na_2O	0.41
MnO_2	0.08
LOI	4.1

Portland Cement Slag

The Portland Cement Slag was collected fromAstrox Cement Private Limited, Surat, Gujarat, India. The high amount of CaO, 50.85%, reflects the Portland Cement Slag as a binder material. To identify and quantify the major and minor oxide elements present in the samples of obtained rice husk ash,X-Ray Fluorescence (XRF) analysis was carried out. The results are given in Table 3.

Table 3. Results of XRF on Portland Cement Slag sample

Oxides	**Value (%)**
CaO	50.85
S_iO_2	27.69
Al_2O_3	11.17
Fe_2O_3	2.93
SO_3	2.78
MgO	2.52
T_iO_2	0.66
Na_2O	0.61
K_2O	0.49
P_2O_5	0.13
S_rO	0.11

SPECIMEN PREPARATION

The soil-Rice Husk Ash-Portland cement Slag was thoroughly mixed with maximum dry density and optimum moisture for various mix proportions. Laboratory test was conducted on soil-Portland cement slag-Rice Husk Ash mixtures to determine the optimum mix content. Based on laboratory test the mix proportion of 82.5%Soil- 7.5% Portland cement slag-10%Rice Husk Ash have been used as the optimum mix.

The proctor curve for this optimum mixture is shown in Fig.2.The specimen used for static and cyclic tests was compacted in three layers having 50mm diameter and 100mm height mould with 98% of maximum dry unit.

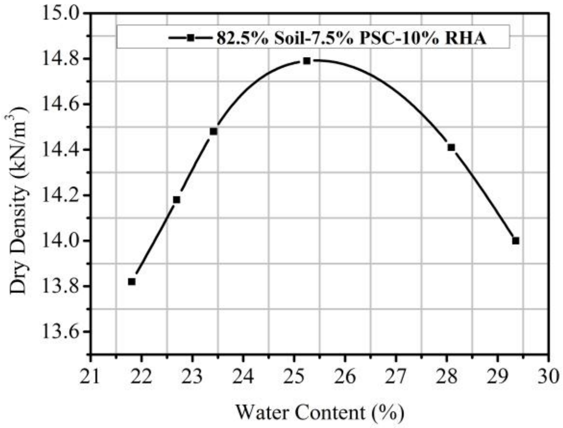

Fig.2.Proctor test for stabilized clay soil

TESTING PROGRAM AND PROCEDURE

Static tests

The static triaxial tests were conducted on stabilized clay soil of 50 mm diameter and 100mm height as per ASTM D4767-11. The soil sample was sheared at a cell pressure of 100kPa, 200kPa and 300kPa, with shear strain of 0.024mm/minute at a proving ring load of 2.5kN.

Cyclic tests

Strain-controlled cyclic triaxial tests were conducted on the stabilized clay soil having 50mm diameter and 100mm height with saturated, consolidated and undrained condition as per ASTM D3999-11.The cyclic triaxial equipment was shown in Fig.3.The stabilized clay samples were tested at strain amplitude of γ=0.4%,0.6%,0.8% and 1% with confining pressure of 100kPa and frequency of 30cycles/minute(0.5Hz)

Fig. 3. Cyclic triaxial equipment set-up

TEST RESULTS AND DISCUSSION

Static Test

The static triaxial tests were carried on the stabilized clay soil and the results are shown in Fig.4. The strain value at failure is about 2% to 2.7 % for confining pressure varying from 100 to 300kPa. The effective stress can be calculated from the deviator stress and pore pressure at failure. The maximum stress at failure on stress-strain diagram was taken as deviator stress at failure and the corresponding pore pressure at this point is taken as pore pressure at failure. The cohesion and angle of internal friction determined for stabilized clay from Mohr’s circle are 30kpa and 13^0respectively.

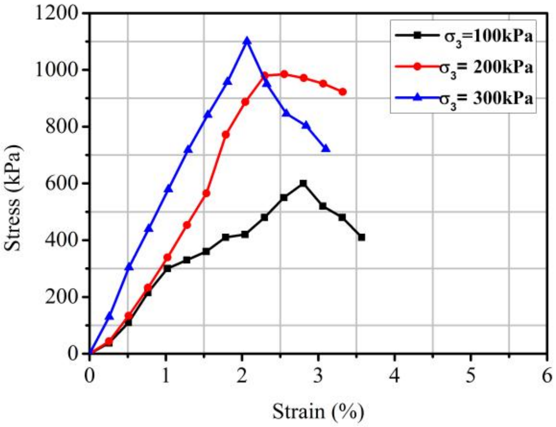

Fig.4.Stress-strain diagram for stabilized clay soil

Cyclic Test

The response behaviors of stabilized clay soil due to cyclic loading are explained in terms of shear modulus, damping ratio and degradation index in the following section.

VARIATION OF CYCLIC LOAD WITH TIME HISTORY

The variations of cyclic load with time history of stabilized clay soil have been plotted for different strain amplitudes. Typical diagrams for strain amplitude of 0.4 % and 1% are shown in Fig.5 and 6.

For the γ=0.4% strain amplitude the deviator stress decreases when cyclic loading proceeds. The load gradually decreased and then remained slowly constant up to 300 cycles as shown in Fig.5.

Similarly, γ=1% strain amplitude reduction in deviator stress will occur when loading proceeds up to 600000ms as shown in Fig.6.

Similar observation also noted for strain amplitude γ=0.6% and γ=0.8%

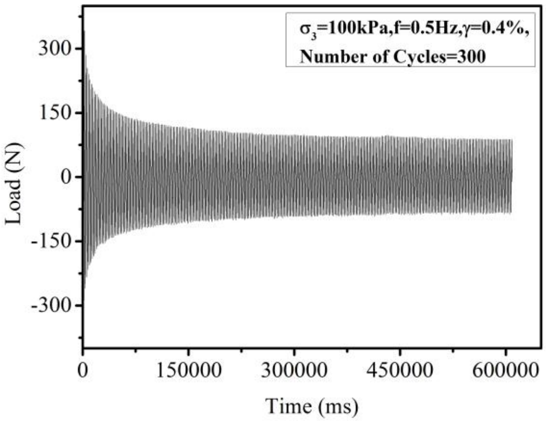

Fig. 5. Variation of cyclic load with time history at shear strain γ=0.4%

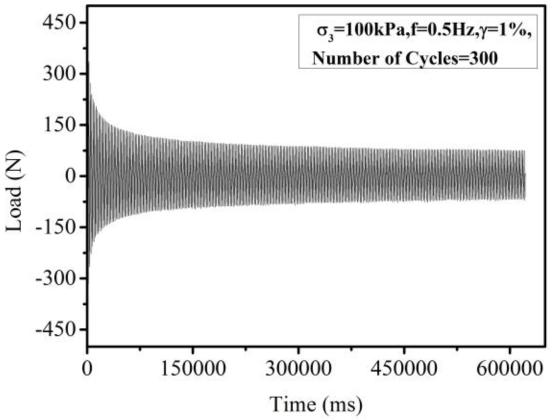

Fig. 6. Variation of cyclic load with time history at Shear Strain γ= 1%

VARIATION OF PORE WATER PRESSURE WITH NUMBER OF CYCLES

The variations of pore water pressure with the number of cycles of stabilized clay soil have been plotted for different strain amplitudes. Typical diagrams for strain amplitude of 0.4% and 1% are shown in Fig.7 and Fig.8

The pore water pressure built up at ten cycles and twelve cycles of loading for a shear strain of 0.4% to 1% respectively, and then increased up to a constant value as the number of cycles increased.

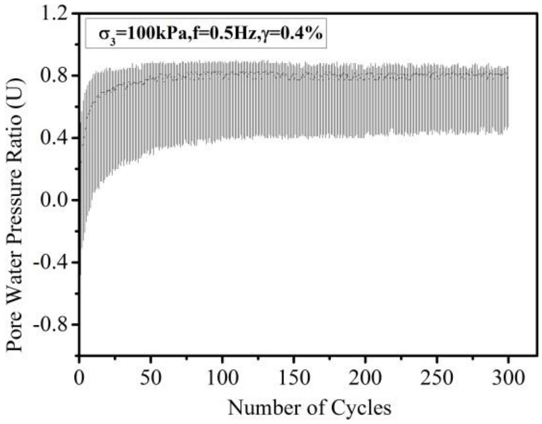

Fig. 7.Variation of excess pore pressure ratio with number of cycles at shear strain γ= 0.4%

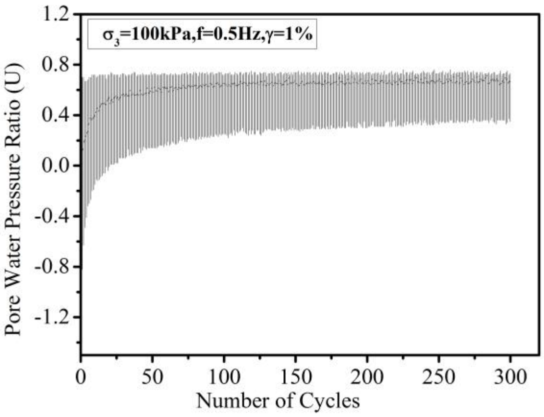

Fig. 8. Variation of Excess pore pressure ratio with number of cycles at shear strain γ=1%

VARIATION OF EFFECTIVE STRESS WITH NUMBER OF CYCLES

The variations of effective stress with number of cycles of stabilized clay soil have been plotted for different strain amplitudes. Typical diagrams for strain amplitude of 0.4% and 1% are shown in Fig. 9 and 10.

For a particular value of strain amplitude, as the cyclic loading progresses, the effective stress path moves towards the left reflecting an increase in pore pressure as shown in Fig.9 and 10.

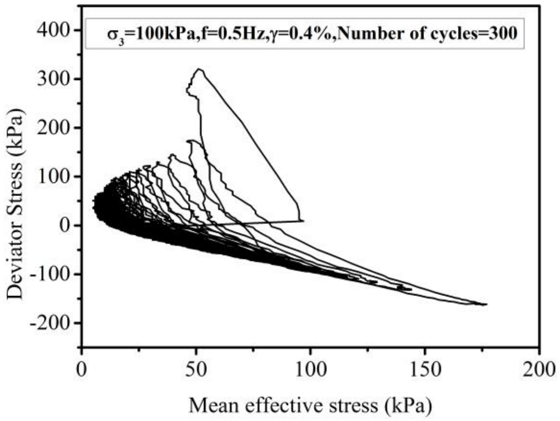

Fig. 9. Stress path for shear strainγ=0.4%

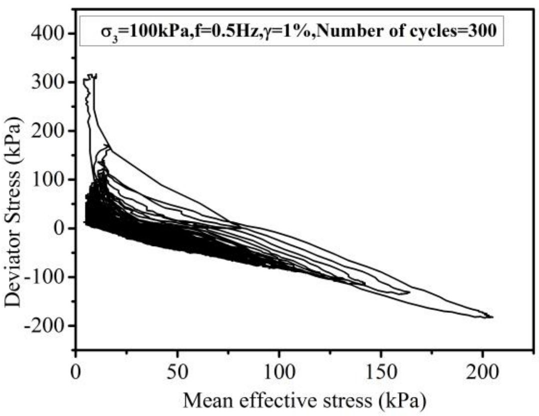

Fig. 10. Stress Path for shear strain γ=1%

VARIATION OF DEVIATOR STRESS WITH NUMBER OF CYCLES

The variations of deviator stress with number of cycles of stabilized clay soil have been plotted for different strain amplitudes. In this case, the data from the test was recorded after 10, 50,100,200 and 300 cycles of loading .Typical diagrams for strain amplitude of 0.4 % and 1% are shown in Fig. 11 and 12.

For γ=0.4%, the deviator stress decreases from 175kPa to 125kPa, 125kPa to100kPa, 100kPa to 85kPa and 85kPa to 75kP as number of cycles increases from 10 to 50, 50to100, 100 to 200 and 200 to 300 respectively. The reduction of deviator stress is about 57% with increase in number of cycles from 10 to 300 as shown in Fig.11

For γ=0.6%, the deviator stress decreases from 125kPa to 80kPa, 80kPa to55kPa, 55kPa to 40kPa and 40kPa to 38kP as number of cycles increases from 10 to 50, 50 to 100, 100 to 200 and 200 to 300 respectively. The reduction of deviator stress is about 68% with increase in number of cycles from 10 to 300

For γ=0.8%, the deviator stress decreases from 100kPa to 65kPa, 65kPa to55kPa, 55kPa to 50kPa and 50kPa to 39kP as number of cycles increases from 10 to 50, 50 to 100, 100 to 200 and 200 to 300 respectively. The reduction of deviator stress is about 61% with increase in number of cycles from 10 to 300

Similarly, γ=1%, the deviator stress decreases from 90kPa to 60kPa, 60kPa to 50kPa,50kPa to 45kPa and 45kPa to 37kPa as number of cycles increases from 10 to 50, 50 to 100, 100 to 200 and 200 to 300 respectively. The reduction of deviator stress is about 59% with increase in number of cycles from 10 to 300 as shown in Fig.12.

As the strain amplitude increases from 0.4% to 1%, the deviator stress decreases about 50% ,51.1%, 48.6% and 48.75% for number of cycles increases from 10 to 50, 50 to 100, 10to 200 and 200 to 300 respectively.

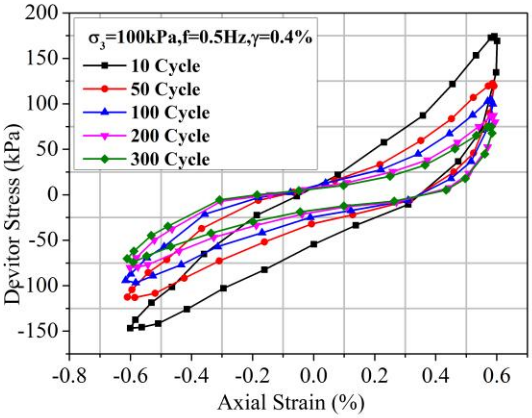

Fig. 11. Deviator stress vs. axial strain for Strain amplitude γ=0.4%

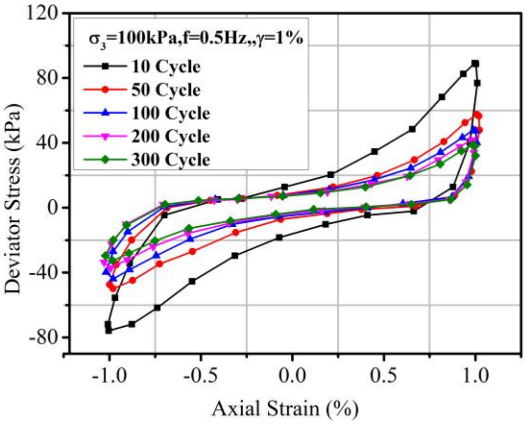

Fig 12.Deviator stress vs. axial strain for Strain amplitude γ=1%

VARIATION OF STRAIN AMPLITUDE WITH NUMBER OF CYCLES

The variations of strain amplitude with number of cycles of stabilized clay soil have been plotted for different strain amplitudes. In this case, the data from the test was recorded after 10, 50,100,200 and 300 cycles of loading Typical diagrams for strain amplitude of 0.4 % and 1% are shown in Fig. 13 and 14.

For tenth cycles, the deviator stress decreases from 175kpa to125kPa and 100kPa to 90kPa as strain amplitude increases from γ=0.4% toγ= 0.6% and γ= 0.8% toγ= 1.0% respectively..The stabilized clay sample carries more cyclic load on the tension side for strain amplitude γ= 0.4% and γ=0.6%, whereas for strain amplitude γ=0.8% and γ=1.0%, the cyclic load shared equally both on the tension and compression side.

Similar observations are noted for 50th, 100th, 200th and 300th cycles. For the 50th cycles, the deviator stress decreases from 125kPa to 80kPa and 65kPa to 60kPa as strain amplitude increases fromγ=0.4% to γ= 0.6% and γ= 0.8% toγ= 1.0%.respectively. For 100th cycles, the deviator stress decreases from 100kPa to 60kPa and 55kPa to 50kPa with increase in strain amplitude fromγ=0.4% toγ= 0.6% and γ= 0.8% toγ= 1.0% respectively. For 200th cycles, the deviator stress decreases from 85kPa to42kPa and 50kPa to 45kPa with increase in strain amplitude fromγ=0.4% toγ= 0.6% and γ=0.8% toγ= 1.0% respectively. For 300th cycles, the deviator stress decreases from 75kPa to 40kPa to and39kPa to 37kPa with an increase in strain amplitude fromγ=0.4% to γ= 0.6% and γ= 0.8% to γ= 1.0% respectively.

The percentage decrease in deviator stress is about 48%, 52%, 50%, 47% and 50.6% after 10, 50,100,200 and 300 cycles of loading for strain amplitude increases from 0.4% to 1% respectively.

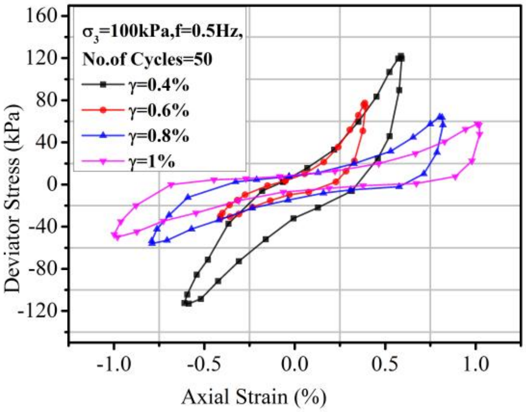

Fig.13.Variation of strain amplitude at 50 cycles

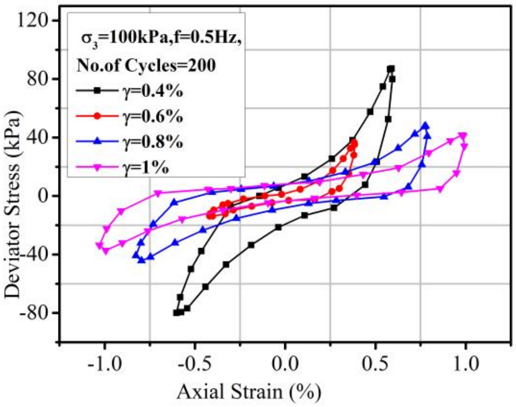

Fig.14.Variation of strain amplitude at 200 cycles

VARATION OF SHEAR MODULUS WITH SHEAR STRAIN

The variations of shear modulus with shear strain of stabilized clay soil have been plotted for different strain amplitudes. Typical diagrams for strain amplitude of 0.4 % and 1% are shown in Fig. 15 and 16.

The shear modulus decreases from 11000kPa to 1500kPa, 10900kPa to 4100kPa, 6107kPa to 1793kP and5000kPa to 1200kPa, with increase in shear strain from 0.598% to 0.63%, 0.896% to 0.946%, 1.208% to 1.33% and 1.512% to 1.61% for strain amplitude 0.4%,0.6%,0.8% and 1.0%, respectively.

The decrease of shear modulus is about 54.5% with increasing strain amplitude from 0.4% to 1%

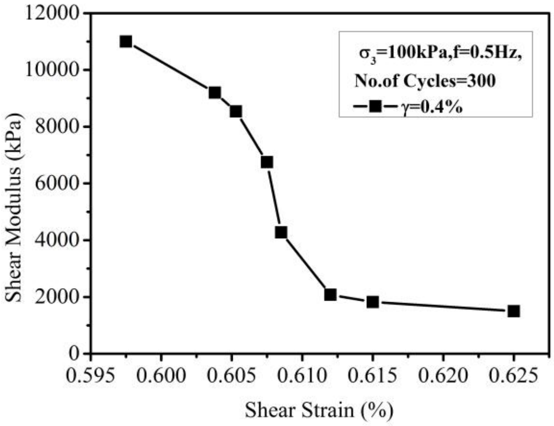

Fig. 15.Variations of shear modulus with shear strain for strain amplitude γ=0.4%

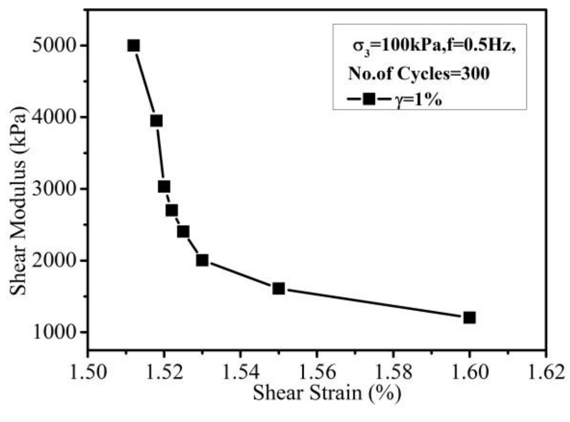

Fig. 16.Variations of shear modulus with shear strain for strain amplitude γ=1%

VARIATION OF DAMPING RATIO WITH SHEAR STRAIN

The variations of damping ratio with shear strain of stabilized clay have been plotted for different strain amplitudes . Typical diagrams for strain amplitude of 0.4 % and 1% are shown in Fig. 17 and 18.

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The damping ratio increased from 6.95% to 8.95%, 11.15% to 13.27%, 13.5% to 18.4% and 14.2% to 19.69%, with increase in shear strain from 0.598% to 0.63%, 0.896% to 0.946%, 1.208% to 1.33% and 1.512% to 1.61% for amplitude of strain 0.4%, 0.6%, 0.8% and 1.0%, respectively

The increase of damping ratio is about 54.5% with increasing strain amplitude from 0.4% to 1%

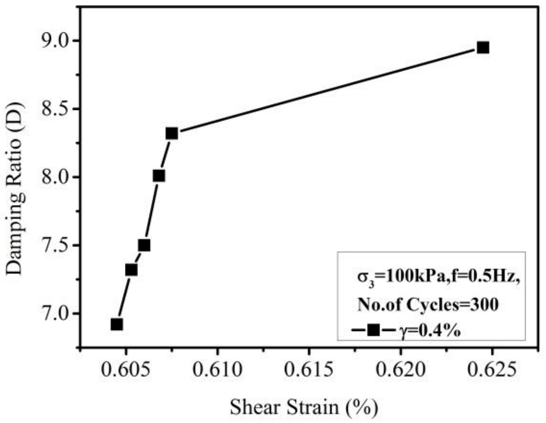

Fig. 17.Variation of damping ratios with shear strain for strain amplitude γ=0.4%

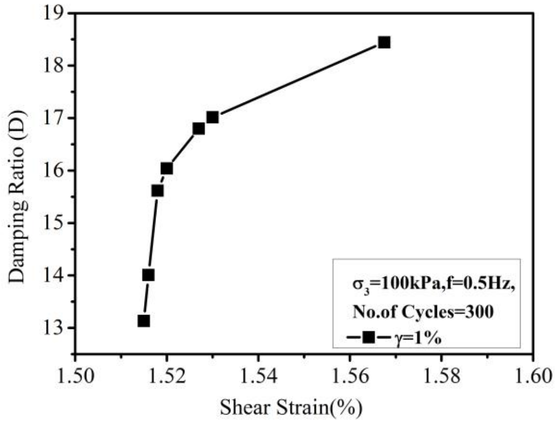

Fig. 18.Variation of damping ratios with shear strain for strain amplitude γ=1%

VARIATION OF SHEAR STRAIN WITH NUMBER OF CYCLES

The variations of shear strain with numbers of cycles of stabilized clay have been plotted for different strain amplitudes and shown in Fig.19.

The shear strain decreases gradually for 12 cycles and then remains constant until end of the cycle. The stabilized clay sample shows that the shear strain decreases with decrease in amplitude of strain. The decrease is about 62.5% with decrease in strain amplitude from 1% to 0.4%

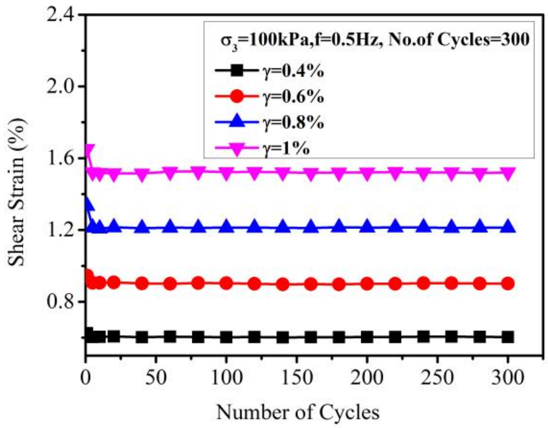

Fig. 19.Variation of shear strain with number of cycles

VARIATION OF SHEAR MODULUS WITH NUMBER OF CYCLES

The variations of shear modulus with numbers of cycles of stabilized clay have been plotted for different strain amplitudes and shown in Fig.20.

The shear modulus gradually decreases up to 200 cycles and then remains constant up to 300 cycles. The Shear modulus decreases from 11000kPa to 1500kPa, 10900kPa to 4100kPa to 6107kPa to 1793kP and5000kPa to 1200kPa with increase in number of cycle at strain amplitude 0.4%, 0.6%, 0.8% and 1% respectively

The decrease of shear modulus is about 54.5% with increasing strain amplitude from 0.4% to 1%

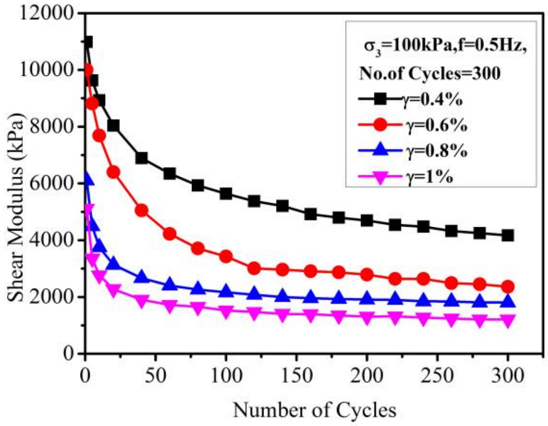

Fig. 20. Variations of shear modulus with number of cycles

VARIATION OF DAMPING RATIO WITH NUMBER OF CYCLES

The variations of damping ratio with number of cycles of stabilized clay have been plotted for different strain amplitudes and shown in Fig.21

The damping ratio decreases with increase in number of cycle up to 300 cycles. The damping ratio decreases from a 8.95% to 6.95%,13.27% to 11.15%,18.4% to 13.5% and19.69% to 14.2% with increasing the number of cycle at strain amplitude 0.4% ,0.6%,0.8% and 1% respectively

The stabilized clay sample shows that the damping ratio value decrease with increase in strain amplitude and the increase is about 54.5% for the strain amplitude varying from 0.4% to 1%.

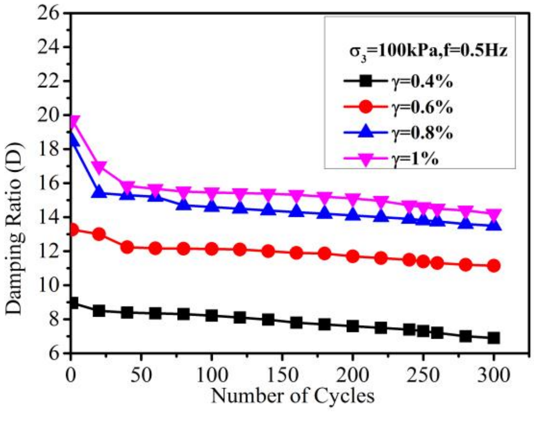

Fig. 21.Variation of damping ratios with number of cycles

VARIATION OF DEGRADATION MODULUS WITH NUMBER OF CYCLES

The variations of degradation index with number of cycles of stabilized clay have been plotted for different strain amplitudes and shown in Fig.22.

Degradation index, δ,is defined as the ratio of secant modulus in the Nth cycle to secant modulus in the first cycle (Idriss et al.1978).. The degradation index decreases at a fast rate for first 50 cycles and then it is almost constant or the decrease is negligible. The degradation index decreases from 1% to 0.38%, 1% to 0.32%, 1% to 0.27% and 1% to 0.23% with increasing the number of cycle at strain amplitude of 0.4% ,0.6%,0.8% and 1% respectively

The percentage decrease in degradation index is about 39% for strain amplitude varying from 0.4% to 1%.

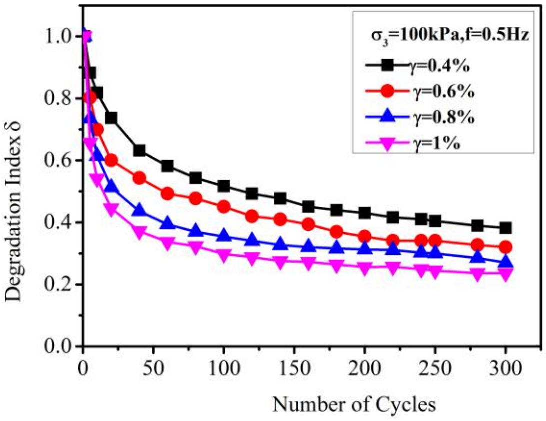

Fig. 22.Variations of degradation modulus with number of cycles

CONCLUSION

A clay soil was collected from a earthquake threat area due to Himalayan frontal and stabilized with the locally available waste materials such as Rice Husk Ash and Portland cement slag

A laboratory testing program was conducted on Portland cement slag-rice Husk Ash-Clay soil mixtures to determine the optimum mix content. Based on laboratory test the mix proportion of 82.5%Soil+ 7.5% Portland Slag Cement+10%Rice Husk Ash have been used as the optimum mix.

Cyclic triaxial tests were carried out to study the variation of dynamic properties of the stabilized clay soil such as shear modulus, G, damping ratio, D, and degradation index, δ,with number of cycles for different strain amplitude of γ= 0.4% to 1% at frequency of 0.5Hz and effective confining pressure of 100kPa.

The pore water pressure built up at ten cycles and twelve cycles of loading for a shear strain of 0.4% to 1% respectively, and then increased up to a constant value as the number of cycles increased.

The shear modulus decreases with increase in number of cycle for all applied strain amplitudes. The decrease of shear modulus is about 54.5% with increasing strain amplitude from 0.4% to 1%.

The damping ratio decreases with increase in number of cycle up to 300 cycles. The damping ratio value decreases with increase in strain amplitude and the increase is about 54.5% for the strain amplitude varying from 0.4% to 1%.

The degradation index decreases at a fast rate for first 50 cycles. The percentage decrease in degradation index is about 39% for strain amplitude varying from 0.4% to 1%.

This variation of dynamic propertied of stabilized clay with strain amplitudes may be helpful for design of subgrade clay soil.

REFERENCES

ASTM Designation: C618. [2012].”Standard Specification for coal fly ash and raw or calcined natural pozzolan for use in concrete”

ASTM Designation: D3999. [2011].”Standard Test Methods for the Determination of the Modulus and Damping Properties of soils using the Cyclic triaxial Apparatus”, Annual Book of ASTM standards, Vol.04.

ASTM Designation D4767. [2011].”Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils”

Gourley,C.S.,Newill,D. and Shreiner,H.D.[1993].”Expansive soils”.TRI’s research strategy,Proc.,1st Int.Symp.on Engrg. Characterstics of Arid Soils.

Hanumantharo,C.and Ramanaa,G.V.[2008].”Dynamic Soil Properties for microzonation of Delhi,India”, J.Earth Syst.Sci. 117 S2,pp.719-730.

Hoyos, L.R.Puppala,A.J,and Chainuwat,P. [2004]. “Dynamic Properties of Chemically Stabilized Sulphate Rich Clay”, J. Geotech. Geoenviron.Engrg.,Vol. 130, No.2, pp.153-162.

Idriss, I.M.,Dobry,R.,and Singh,R.D. [1978]. “Nonlinear Behavior of Soft Clay during Cyclic Loading”, J. Geotech. Engrg. Div., Vol. 104, No.12, pp.1427-1447.

Jiang , M.,Cai,Z.,Cao,P.,and Liu,D.[2010]. “Effect of Cyclic Loading Frequency on Dynamic Properties of Marine Clay”, Soil Dyn. and Earthquake, Geotech. Special Publication, GeoShangi International Conference. ASCE, No.201, pp. 240-245.

Metha,P.K.[1986].”Concrete Structure, properties and materials”, Prentice Hall, Englewood Cliffs, New Jersey.

Prosniski, J. and Bhattacharaya, S. [1999].”Effectiveness of Portland cement and lime in stabilizing clay soils”, Transportaioin Research Record No.1652, Transportaion Research Board, Washnigton ,DC.

Sabat, A.K. and Nanda, R.P. [2011]. “Effect of Marble Dust on Strength and Durability of Rice-Husk Ash Stabilized Expansive Soil”, Inter. J. Civ. Str. Engrg., Vol 1, No.4, pp.939-948.

Shafiee, A., Fafari, M. K., and Jalali, N.[2004].”Cyclic Behavior of Composite Clays”,13th World conference on Earthquake Engrg. Vancouver, B.B, Canada, No.462.

Sherwood, P. T.[1995].”Soil Stabilization with cement and lime”,HMSO,Pub.Center.pp14-55.

Teachvorasinskun, S.,Thongchim, P., and Lukkunaprasit, P.[2002]. “Shear Modulus and Damping of Soft Bangkok Clays”, Canadian Geotech. J., Vol. 39, No. 5, pp. 1201-1208.

Thammathiwat, A. and Chim-oye, W. [2004]. “Behavior of Strength and Pore Pressure of Soft Bangkok Clay Under Cyclic Loading”, Thammasat Int .J. Sc. Tech., Vol.9, No. 4, pp. 21-28.

Zhou,J. and Gong, X. N.[2001].”Strain degradation of saturated clay under cyclic loading”, Canadian Geotech. J., Vol.38, No.2 pp.208-212.