Indexed in
Scopus Compendex and Geobase Elsevier,
Geo-Ref Information Services-USA, List B of Scientific
Journals, Poland, Directory of Research Journals

International Journal
of Earth Sciences
and Engineering

www.cafetinnova.org

ISSN 0974-5904, Volume 09, No. 05

October 2016, P.P.1907-1915

Initial Shear Modulus of Sand-Fines Mixtures

B G SHIVAPRAKASH AND S V DINESH

Department of Civil Engineering, Siddaganga Institute of Technology, Tumkur, Karnataka, INDIA

***Email:** bgshivaprakash@gmail.com, dineshsv2004@gmail.com*

Abstract: This paper presents the dynamic properties of sand-fines mixtures based on the results of undrained cyclic triaxial tests. The sand was obtained from Sabarmati River, and three types of fines F1, F2, and F3 were obtained from locally available fine grained soils i.e. Suddha soil, Black cotton soil, and Bentonite soil. Sand-fines mixtures of wide range of plasticity from non-plastic to highly plastic mixture with PI= 86 were prepared using the fines passing 75µ size. The initial shear moduli of all sand-fines mixtures are examined in terms of fines content and conventional plasticity index (IP). In the present investigation, a term equivalent plasticity index (IP*) has been defined by considering the plasticity characteristics of soil passing 2 mm size IS sieve on the same lines of [16]. There is better correlation between the initial shear modulus of sand-fines mixtures and equivalent plasticity index (IP*) compared to conventional plasticity index (IP). An empirical equation for predicting initial shear modulus is proposed based on equivalent plasticity index and mean-effective stress.

***Keywords:** Dynamic properties; Shear modulus; Plasticity index, equivalent plasticity index; Sand-fines mixture*

1. Introduction

Determination of dynamic soil properties is difficult but extremely important for solving geotechnical earthquake engineering problems. Dynamic soil properties include initial shear modulus, shear modulus, and damping variations with shear strains. Field evaluation of dynamic soil properties are mainly based on the estimation of shear wave velocity at low strain level. The low strain level is that at which the soil shows higher stiffness and lower damping, and stress-strain behavior is linear. The strain less than 0.001% is considered as low strain [1]; [2]; [3]. Laboratory based evaluations help in the estimation of a true range of dynamic soil properties at varying shear strain levels. The various laboratory tests used to investigate low strain (<0.001%) dynamic properties of soil are: Resonant column test, Hollow cylindrical torsion test, Ultrasonic test, and piezoelectric bender element test. For large strains (>0.001%) dynamic properties: cyclic triaxial test, cyclic direct shear test, and cyclic torsional shear tests are commonly used. Various parameters such as relative density, confining pressure, soil plasticity, strain amplitude, frequency and magnitude of cyclic loading influence the dynamic soil properties. The initial shear modulus is useful to compute shear modulus degradation curves form the input parameters for ground response analysis and response of structures due to the cyclic loading arising from earthquake and wave effects etc... [4] Proposed empirical relationship to estimate the initial shear modulus for clean sands based on the laboratory tests. There are many experimental investigations for determining initial shear modulus of sand and several of them have proposed predictive relationships for determining initial shear modulus as shown in Table-1. These relationships are largely based on void ratio and confining pressure [5], [6], [7], [8], [9], and [10]. There are also several investigations [11], [12], [13], [14] for obtaining the initial shear modulus of clays and several empirical relationships for initial shear modulus. The available relationships are useful for sands and clays.

There are very limited studies for the determination of initial shear modulus of natural soils which consists of mixtures of sand, silt, and clay in various proportions. [15] & [16] have conducted studies on sand-clay mixtures and proposed empirical equations for determining initial shear modulus of sand-clay mixtures over a wide range of plasticity. [15] have also proposed empirical relationships applicable for sand-clay mixtures with plasticity index (PI) < 30 and clay mixtures with PI >30.

[16] has proposed an empirical relationship applicable for a wide range of plasticity and they have considered the plasticity of mixtures in terms of an equivalent plasticity index (IP*) is given by Eq. (1). There are very few studies about the dynamic properties sand-fines mixtures; hence, there is need for the investigation of dynamic properties of wide range of soils including sand-silt and sand-clay mixtures.

$$G_0 = 4{,}000.(IP^*)^{-0.7}(\sigma_m^{'}) \qquad (1)$$

Where IP*= equivalent plasticity index which is obtained by conducting consistency limit tests on soil passing 2mm sieve; $\sigma_m^{'}$ = mean effective confining pressure.

[16] has suggested equivalent plasticity index (IP*) based on conventional plasticity index (IP) and a reduction factor 'R_F' obtained from grain size data of

soil passing 425 micron sieve and original particle size distribution data

$$IP^{*} = IP \times R_F \quad (2)$$

$$R_F = \frac{Pc_a}{Pc_b} \quad (3)$$

Where Pc_a = clay content from the grain size distribution curve having particles less than 2mm (%) and Pc_b = clay content from the grain size distribution curve having particles 0.425mm (%).

In the present work, the reduction factor 'R_F' has been computed considering only the original grain size distribution curve.

In the present work a series of undrained, stress controlled cyclic triaxial tests was carried out on remolded soil samples to determine the dynamic properties of sand-fines mixtures corresponding to 0.02% to ≤1% shear strain levels. The initial shear modulus values for shear strain at 1x 10^{-4}% are computed from empirical relationship proposed by [16]. Using this data, the modulus reduction curves are obtained and the effect of fines content and plasticity index on initial shear modulus have been discussed.

2. Materials and Experimental Method

For the present investigation, river sand was obtained from the banks of Sabarmati River at a depth of 3m. The sand used for investigation is designated as S1 and it has specific gravity of 2.63, shape of the grain is sub-rounded and it is classified as SP as per the I S Soil classification. Three types of fine grained soils were selected from local sources, namely Suddha soil, Black cotton soil and Bentonite soil having PI=19, 46, 105 respectively. The grain size distribution of fine grained soils and that of base sand are shown in figure.1 and the geotechnical properties are shown in Table-2. These fine grained soils were sieved through 75 micron IS sieve and the fines less than 75μ size were used to prepare sand-fines mixtures. The fines obtained from these soils are designated as F1, F2 and F3 respectively. The fines F1, F2 and F3 belong to SM, MH and CH as per IS classification which is same as [17]. They are chosen to ensure sand-fines mixtures of wide range of plasticity are considered.

2.1. Sand-fines Mixtures

Sand-fines mixtures were prepared by mixing base sand and different percentages of F1, F2 and F3 fines and the properties of sand-fines mixtures with different percentage of fines are shown in Table-3.The mixtures have been designed to obtain sand-fines mixtures covering a wide range of plasticity. The liquid limit varies from 22 to 135 and PI varies from non-plastic to 86. Figures 2, 3 and 4 show the grain size curves for base- sand and sand-fines mixtures with various percentages of fines.

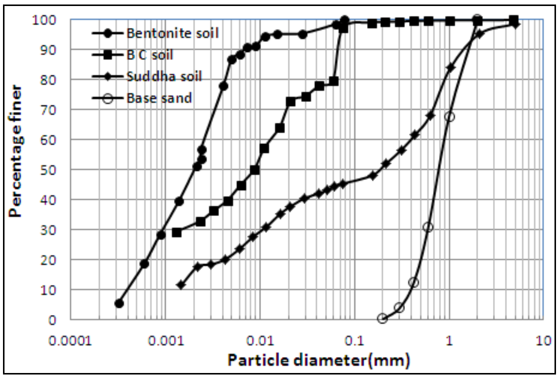

Figure 1 *Grain size distribution curves of Base sand, F1, F2 and F3 fines*

Table 1: *Relationships to estimate maximum/ initial dynamic shear modulus for sands based laboratory tests*

Type of Sand & Reference	Empirical relationship	Reference strain	Valid units $\bar{\sigma}_0$	Valid units G_{max}
Ottawa sand/ Hardin (1965)	$G_{Max} = \frac{(32.17 - 14.8e)^2}{(1+e)} \bar{\sigma}_0^{0.5}$ $\bar{\sigma}_0 > 2000$ psf $G_{max} = \frac{(22.52 - 10.6e)^2}{(1+e)} \bar{\sigma}_0^{0.5}$ $\bar{\sigma}_0 < 2000$ psf	2.5 x 10^{-5}	Psf	psi
Ottawa Sand / Drnevich and Richart (1970)	$G_{Max} = \frac{(32.17 - 14.8e)^2}{(1+e)} \bar{\sigma}_0^{0.5}$	<10^{-5}	psf	psi
Clean Sands/Hardin and Drnevich (1972)	$G_{Max} = 1230 \frac{(2.97 - e)^2}{(1+e)} \bar{\sigma}_0^{0.5}$	<2.5 x10^{-5}	psi	psi
Clean Sands /Iwasaki and Tatsuoka (1977)	$G_{Max} = 900 \frac{(2.97 - e)^2}{(1+e)} \bar{\sigma}_0^{0.4}$	10^{-6} 10^{-5} 10^{-4}	kg/cm^2	kg/cm^2

	$G_{Max}=850\dfrac{(2.97-e)^2}{(1+e)}\bar{\sigma}_0^{0.44}$ $G_{Max}=700\dfrac{(2.97-e)^2}{(1+e)}\bar{\sigma}_0^{0.5}$			
Monterey No.0 Sand/ Drnevich (1978)	$G_{Max}=1230\dfrac{(2.97-e)^2}{(1+e)}\bar{\sigma}_0^{0.5}$	$<10^{-5}$	Psi	psi
Clean sand / Hardin (1978)	$G_{Max}=\dfrac{625}{(0.3+0.7e^2)}p_0^{0.5}\bar{\sigma}_0^{0.5}$	10^{-5}	any units	Same as $\bar{\sigma}_0$
Toyoura Sand and Gifu Sand/ Kokusho (1980)	$G_{Max}=840\dfrac{(2.17-e)^2}{(1+e)}\bar{\sigma}_0^{0.5}$	10^{-5}	kg/cm^2	kg/cm^2
Monterey No.0 Sand/ Chung et al (1984)	$G_{Max}=\dfrac{523}{(0.3+0.7e^2)}p_0^{0.52}\bar{\sigma}_0^{0.48}$	10^{-5}	any units	same as $\bar{\sigma}_0$
Monterey No.0 Sand/ Saxena and Reddy (1987)	$G_{Max}=\dfrac{428.2}{(0.3+0.7e^2)}p_0^{0.426}\bar{\sigma}_0^{0.576}$	$<10^{-5}$	any units	same as $\bar{\sigma}_0$

Table 2: *Properties of Base Sand (S1) and Fine grained soils*

S.N	Properties	Sand	Fine grained soils		
		S1	**Suddha soil**	**B.C soil**	**Bentonite soil**
1	Specific gravity	2.63	2.65	2.77	2.82
2	Gravel (%)	--	0	0	0
3	Sand (%)	100	56	3	0
4	Silt (%)	--	26	65	52
5	Clay (%)	--	18	32	48
6	Co-efficient of uniformity(Cu)	2.26	--	--	--
7	Co-efficient of Curvature (Cc)	0.95	--	--	--
8	Maximum void ratio (e_{max})	0.65	--	--	--
9	Minimum void ratio (e_{min})	0.35	--	--	--
10	D_{50}(mm)	0.833	--	--	--
11	D_{10}(mm)	0.427	--	--	--
12	Maximum dry density $\rho_{d\,max}$(g/cc)	1.95	--	--	--
13	Minimum dry density $\rho_{d\,min}$ (g/cc)	1.59	--	--	--
14	Group symbol (IS classification)	SP	SC	MH	CH
15	Shape of grain	Sub rounded	--	--	--
16	Liquid Limit	--	46	80	160
17	Plastic Limit	--	27	34	55
18	Plasticity Index	--	19	46	105

Table-3: *Properties of Sand-fines mixtures*

Sample	Sand-Fines	FC (%)	G_S	w_L	w_p	IP	W_{L*}	W_{P*}	IP*	classification
S_1F_1 Mix.	S1F1-15	15	2.633	22	-	0	22	-	0	SM
	S1F1-20	20	2.634	24	16	8	24	16	8	SC
	S1F1-25	25	2.635	28	15	13	24	14	10	SC
	S1F1-35	35	2.637	38	20	18	35	20	15	SC
S_1F_2 Mix.	S1F2-30	30	2.672	48	23	25	19	8	11	SC-CH
	S1F2-40	40	2.686	59	24	35	29	14	15	SC-CH
	S1F2-50	50	2.700	62	25	37	39	17	22	SC-CH
	S1F2-60	60	2.714	65	25	40	49	20	29	SC-CH
S_1-F_3 Mix.	S1F3-30	30	2.687	98	33	65	42	21	21	SC-CH
	S1F3-40	40	2.706	110	35	75	65	23	42	SC-CH
	S1F3-50	50	2.725	123	42	81	78	25	53	SC-CH
	S1F3-60	60	2.744	135	49	86	91	26	65	SC-CH

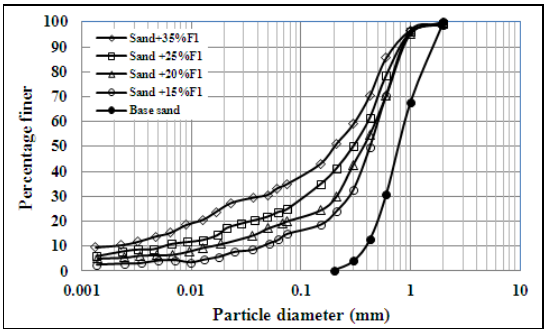

Figure 2 *Grain size curves of sand and sand-fines (S1F1) mixtures*

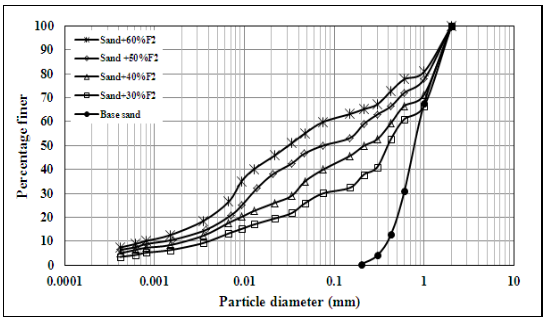

Figure 3 *Grain size curves of sand and sand-fines (S1F2) mixtures*

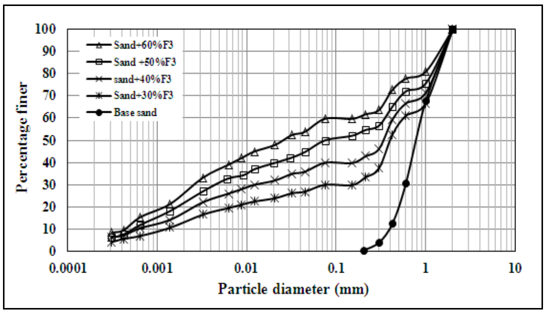

Figure 4 *Grain size curves sand and sand-fines (S1F3) mixtures*

2.2. Sample Preparation and Testing Procedure

A split mould of size 50 mm diameter and 100mm height along with rubber membrane glued to the inner surface with O rings was used for preparing the soil sample. Porous stones and filter papers were placed at the top and the bottom of the sample. The soil samples were prepared by dry deposition method (Ishihara) [14].The sand and fines portions were divided into 10 equal parts. Each of these sand and fines parts were thoroughly mixed and poured into mould gently using spout. The mixture was dry deposited with zero height free fall to prevent the segregation of particles such that the sand particles and fines roll down slowly**.** Each layer was tamped gently by giving 1-4 blows by wooden mallet of weight 144grams with amplitude of 50mm to impart energy of 1.43 x 10^{-4}N-m/m^3 to compact the mixture to the particular density. The above procedure was repeated for all the layers. Water was allowed slowly into the triaxial cell from the tank and filled up to the top of triaxial cell. CO_2 gas was passed through the soil sample for approximately one hour to remove the air voids. Then, deaired distilled water was passed through the soil sample until 600 ml of water (nearly 3 to 4 times of the volume of the sample) flows out from the sample at a velocity of 1x 10^{-4}cm /sec for S1+F3 type of fines such that the laminar flow condition prevail and segregation of fine particles is prevented. Then a cell pressure (100kPa) and back pressure (90kPa) were applied to the soil sample, under an effective confining pressure of 10 kPa and the sample was saturated by back pressure technique such that the Skempton's 'B' factor i.e. B= $\Delta u/\Delta\sigma$ to be not less than 0.96 to ensure saturation. Thereafter, Cell pressure (190kPa) and back pressure (90kPa) were applied for consolidation of sample at confining pressure of 100 kPa by keeping the drainage valves open. The volume change was recorded and the samples were isotropically consolidated at 100kPa confining pressure till the end of Casagrande's primary consolidation. The volume change (ΔV), and change in height (Δh) of the sample after consolidation were recorded by the sensitive volume change measuring device and LVDT respectively. To measure the displacements at lower strain level (2 x 10^{-2}% to 0.1%) proximity sensors were used. Beyond 0.1% strain, displacements were measured by LVDT. The deviator load was measured by a submersible load cell of 1kN capacity. The final volume and final height at the end of consolidation were computed and used for calculations. The reported results are based on relative density after primary consolidation. The sample was subjected to constant amplitude sinusoidal cyclic deviator stress at a frequency of 0.1Hz as per [19]. [21], [16] reported that the effect of frequency of loading on G/G_0 was negligible. Under repeated application of deviator load the sample is subjected to both compression and extension resulting in the development of pore pressure. The deviator load, sample deformation, and pore pressure were recorded accurately by a computer controlled data acquisition system. There were 128 data points in each loading cycle.

2.3. Experimental Programme

Table-4 shows the details of experimental Programme of stress controlled cyclic triaxial tests for sand-fines mixtures prepared from F1, F2 and F3 fines in various proportions. The initial and final void ratios at the end primary consolidation of the sample are shown in Table-4.The shear modulus is calculated using cyclic triaxial test data by the following Eq. (4)

$$G = \frac{E}{2(1+\mu)} \tag{4}$$

Where, μ is the Poisson's ratio taken as 0.5 for saturated undrained specimens [20].

Table-4: *Experimental Programme*

S.N	Sand-fines mixture	Con. pressure KPa	Freq.of loading Hz	Type of test	Initial void ratio(e_0)	Final void ratio at the end of consolidation(e_c)
1	S1F115%	100	0.1	Stress	0.615	0.60
2	S1F120%	100	0.1	controlled	0.621	0.602
3	S1F125%	100	0.1	cyclic triaxial	0.634	0.606
4	S1F135%	100	0.1	test	0.645	0.604
5	S1F230%	100	0.1	Stress	0.726	0.703
6	S1F2 40%	100	0.1	controlled	0.750	0.712
7	S1F250%	100	0.1	cyclic triaxial	0.760	0.706
8	S1F260%	100	0.1	test	0.770	0.708
9	S1F330%	100	0.1	Stress	0.825	0.804
10	S1F3 40%	100	0.1	controlled	0.855	0.801
11	S1F350%	100	0.1	cyclic triaxial	0.859	0.806
12	S1F360%	100	0.1	test	0.868	0.813

2.4. Initial Shear Modulus

The shear moduli of sand-fines mixtures are shown in figures 5, 7, and 9 correspond to a mean effective principal stress of 66.7kPa at a frequency of 0.1Hz for a wide range of single amplitude shear strain from 1×10^{-4} to 1.0%. The curves in these figures represent the hyperbolic relationship proposed by [4] and are given by

$$\frac{G}{G_0} = \frac{1}{1+\gamma_{SA}/\gamma_r} \qquad (5)$$

Where G = shear modulus; G_0 = initial shear modulus corresponding to a strain level of $\gamma_{SA}=1\times10^{-5}$%; and γ_r = reference strain, in the present study reference strain was considered at G/G_0 =0.5[22] The shear modulus at shear strain level equal to 1×10^{-4} is referred as initial shear modulus G_0, was obtained from Equation -1 [16] The shear strain was calculated using the Eq. (6) given below

$$\gamma = \frac{2(1+\mu)}{E}\tau \qquad (6)$$

Figure.5 shows shear modulus versus single amplitude Shear strain for the base sand with various percentages of F1 fines. The shear modulus obtained from cyclic triaxial test and the initial shear modulus (G_0) computed from the empirical relation [16] for the sand-fines mixtures are shown as open and solid symbols respectively. The initial shear modulus is considered at strain level of 1×10^{-4}. Using this data the modulus reduction curves as per the [4] are shown in Figure 6. Figures 5, 7 and 9 show shear modulus versus single amplitude shear strain for the base sand with various percentages of F1, F2 and F3 fines respectively. The results are in good agreement with the general trend of decrease in shear modulus for the soils with increase in fines and plasticity at constant confining pressure.

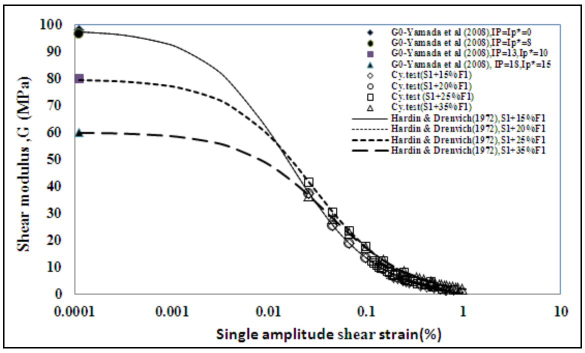

Figure 5 *Shear modulus versus single amplitude shear strain for S1+F1 mixtures*

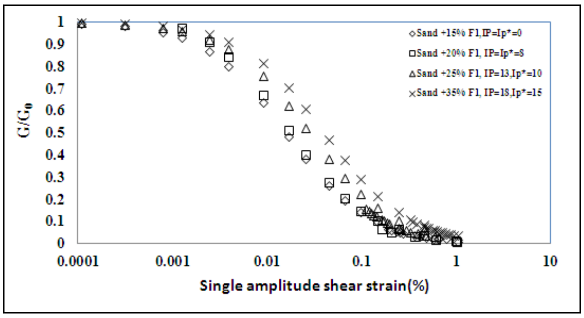

Figure 6 *G/G_0 versus single amplitude shear strain for S1+F1 mixtures*

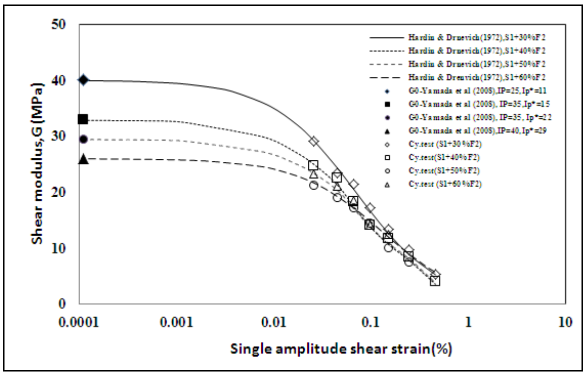

Figure 7 *Shear modulus versus single amplitude shear strain of S1+F2 mixtures*

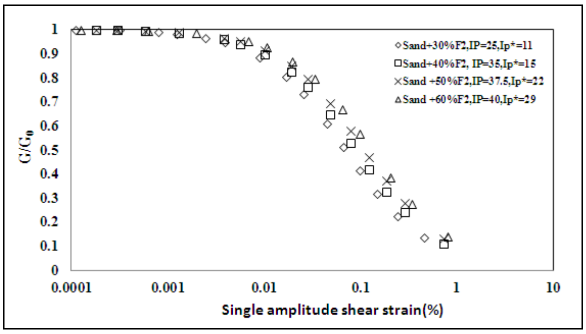

Figure 8 *G/G_0 versus single amplitude shear strain for S1+F2 mixtures*

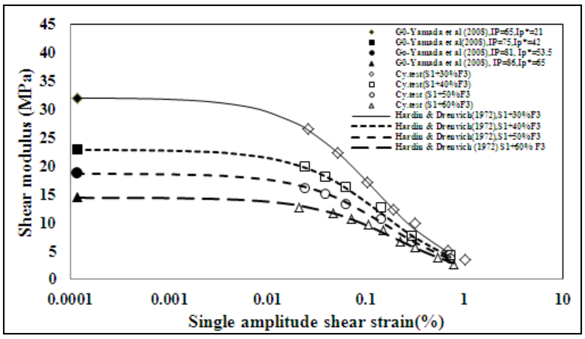

Figure 9 *Shear modulus versus single amplitude shear strain of S1+F3mixtures*

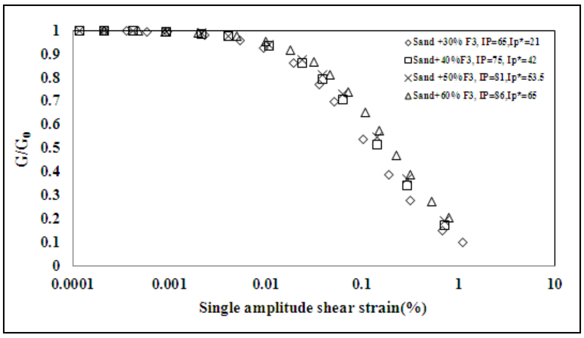

Figure 10 *G/G_0 versus single amplitude shear strain for S1+F3 mixtures*

The magnitude of decrease of shear modulus is significant with increase in fines content. For all sand-fines mixtures the samples show elastic behavior at low shear strain levels 5x10^{-4} Figure 6 shows variation G/G_0 versus single amplitude shear strain for sand with various percentages of F1 fines (15%, 20%, 25% and 35%) with IP=0, 8, 13, 18 respectively. G/G_0 for shear strain level 1×10^{-4} to 5×10^{-4} all the sand-fines mixtures behaves as elastic and there after the degradation begin at different strain levels. From figure 6 it is evident that the degradation depends on PI value, higher the PI lower the shear modulus degradation and the effect of PI is significant. At large strain levels (nonlinear range) the reduction in G/G_0 is higher at any shear strain level with increase in plasticity index of sand-fines mixtures, thereby soils of higher plasticity show increased stiffness when compared to low plasticity soils. Figures 8 and 10 shows shear modulus degradation (G/G_0) curves for sand-fines mixtures containing F2 and F3 fines respectively. Similar observations regarding the effect of plasticity index on modulus degradation could be made.

Figure 11 shows the initial shear modulus versus fines content for sand-fines mixtures considered in the present study. The initial shear modulus decreases with increase in fines content i.e. increase in plasticity index. Sand+ F1 fines shows higher initial shear modulus compared to Sand +F2 and Sand + F3 fines mixtures. For all sand-fines mixtures, the initial shear modulus follows linearly decreasing trend with increase in fines content. The decrease of initial shear modulus depends on the type of fines and no generalization could be made.

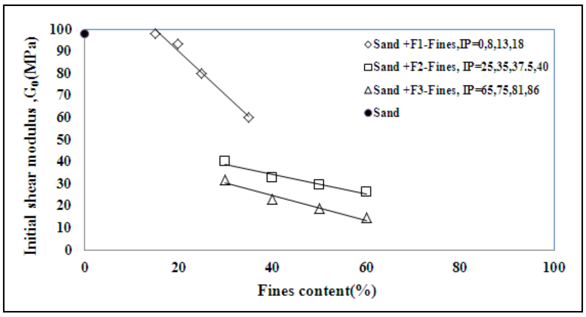

Figure 11 *Initial shear modulus versus fines content for sand with various percentages of F1, F2, and F3 fines*

In addition to conventional liquid and plastic limit tests to determine plasticity index IP, liquid and plastic limit tests were performed on soils passing 2mm size IS sieve on the lines of [16] to determine equivalent plasticity index (IP*). The Figure 12 shows the variation of plasticity index (IP) and equivalent plasticity index (IP*) versus sand content as solid and open symbols respectively. The results show that for each of the mixtures IP* is less than IP at any intermediate range of sand contents for sand-fines mixtures containing highly plastic fines when compared with sand-fines mixtures containing low plastic fines.

Considering the natural grain size curve of sand-fines mixtures (figure- 14) an empirical relation between IP* and IP has been obtained by regression analysis (r^2=0.939) as shown in Eq. (7)

$$IP^* = -14.5 + 0.541IP + 8.79R_F \qquad (7)$$

IP* = equivalent plasticity index; IP = conventional plasticity index; R_F = reduction factor, representing the ratio of percentage of particle size less than 0.425mm to percentage of particle size greater than 0.425mm upto 2mm from grain size curve. [16] was also proposed an empirical relationship to predict the equivalent plasticity index (IP*) based two grain size curve data and as given by Eq. (2). In the present investigation showed that IP* can also found by Eq. (7) using single grain size curve data.'

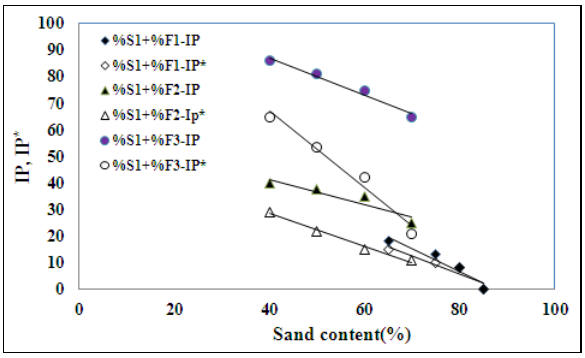

Figure 12 *IP, IP*versus percentage of sand content in sand-fines mixtures*

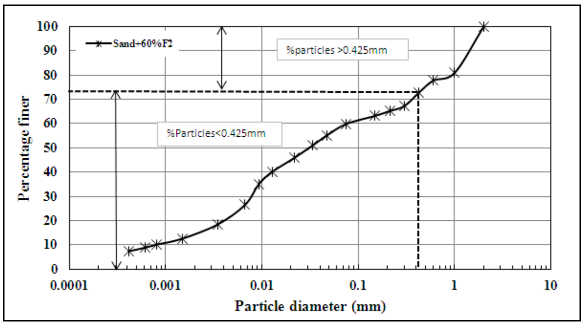

Figure 13 *Typical grain size distribution curve of S1+60%F2 fines used to choose % particles <0.425mm and % particles >0.425mm for prediction of equivalent plasticity index (IP*)*

Figure 14 shows the relation between the measured IP* versus predicted IP* using Eq. (7) for all sand-fines mixtures used in the present investigation and experimental data of [16].There is good correlation between the experimental IP* and predicted IP* for all the sand-fines mixtures and the data of [16]. There is a good correlation between experimental and predicted value with $r^2 = 0.939$

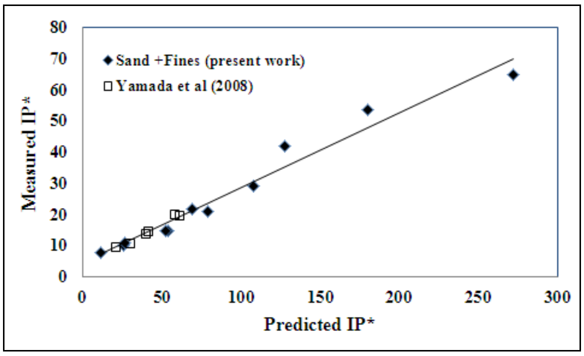

Figure 14 *Comparison between predicted and experimental value of IP* of sand-fines mixtures*

Figure 15 shows the initial shear modulus versus IP for sand-fines mixtures containing F1, F2 and F3 fines. Sand +F1 samples (IP=0 and 8) shows the same shear modulus values upto 20% of F1 fines, but further addition of F1 fines content shows decrease in shear modulus because of increase in PI in the sand-fines mixture. Similarly, sand-fines mixtures containing F2 and F3 fines, show decreased shear modulus with increase in plasticity index. However, sand-fines mixtures of different origin show different trend and the initial shear modulus does not correlate with conventional plasticity index.

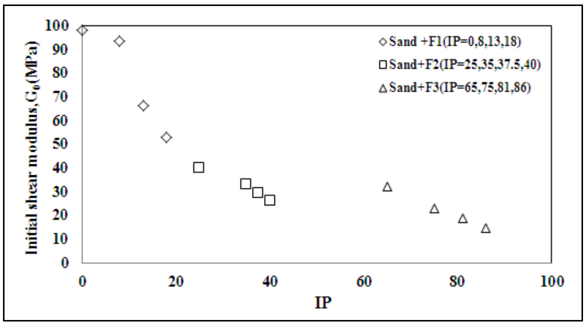

Figure 15 *Initial shear modulus (G_0) versus plasticity index, IP for sand with various percentages of F1, F2, and F3 fines*

Figure 16 shows the initial shear modulus versus IP* for sand-fines mixtures containing F1, F2, and F3 at various percentages of fines. The initial shear modulus of various sand-fines mixtures shows a decreasing trend and correlates well with the equivalent plasticity index (IP*). Therefore, the equivalent plasticity index shows better correlation with the initial shear modulus than conventional plasticity index (IP).

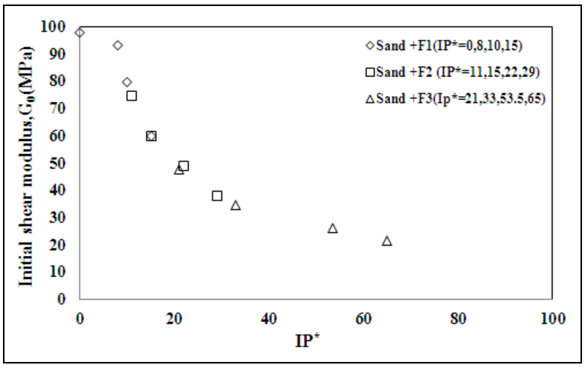

Figure 16 *Initial shear modulus (G_0) versus equivalent plasticity index, IP* for sand with various percentages of F1, F2, and F3 fines*

2.5. Prediction of Initial shear modulus (G_0)

For sand-fines mixtures containing plastic fines, the equivalent plasticity index (IP*) is less when compared with conventional plasticity index value depending on the sand content, the initial shear modulus (G_0) decreases with increase in fines content. Considering the equivalent plasticity index (IP*) which reflects the true physical characteristics of soil, a relation between the IP*and initial shear modulus (G_0) has been obtained as shown below

$$G_0 = 371(IP^*)^{-0.69}(\sigma'_m)^{0.006} \quad (8)$$

Figure 17 shows the plot of IP* from experiments versus initial shear modulus (G_0) from Eq. (8) for all sand-fines mixtures considered in the present investigation and experimental data of [16] .There is good correlation between the present studies G_0 and predicted G_0 from Eq.(8) for all the sand-fines mixtures and G_0 from [16]

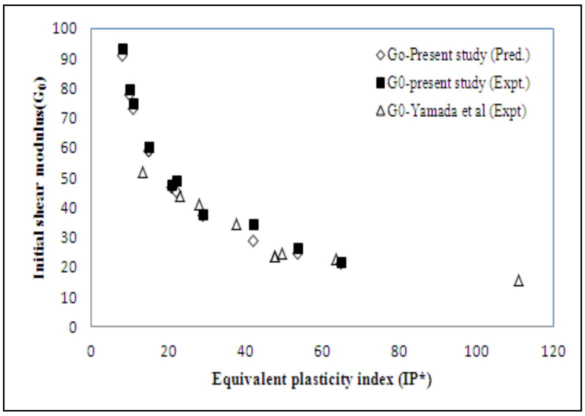

Figure 17 *Initial shear modulus (G_0) versus equivalent plasticity index, IP* for sand –fines mixtures*

2.6. Conclusions

This paper reports the results of cyclic triaxial tests and empirical equations for determining the shear modulus variation over a wide range of single amplitude shear strain for sand-fines mixtures containing various types of fines with a wide range of plasticity at confining pressure of 100kPa. The variation of initial shear modulus was analyzed in terms of fines content, plasticity index and equivalent plasticity index [16] was defined for sand-fines mixtures and equivalent plasticity index relates well with initial shear modulus. A simple empirical relationship (Eq.8) was proposed to determine the initial shear modulus based on equivalent plasticity index and confining pressure. The important findings of this investigation are presented below

- The initial shear modulus of sand-fines mixtures decreases with increase in fines content, however, the initial shear modulus does not show definite trend with reduction in fines content.
- The modulus degradation (G/G_0)of sand-fines mixtures decreases with increase in conventional plasticity index(IP)/ equivalent plasticity index(IP*) at large strain level (non-linear range)
- The initial shear modulus for sand-fines mixtures does not correlate well with the conventional plasticity index(IP); however, the initial shear modulus shows better correlation with equivalent plasticity index(IP*) than the conventional plasticity index(IP) over a wide range of plasticity; and is in full agreement with the results of [16].
- The initial shear modulus can be predicted based on mean effective confining pressure and equivalent plasticity index (IP*) which can be estimated from conventional consistency limits and grain size analysis.

3. Acknowledgements

The authors thank AICTE for funding the research project titled “Liquefaction potential of Sand-clay Mixtures” under RPS scheme No.: 20/AICTE/RIFD/RPS (POLICY-III) 31/2012-13

References

[1] Luna.R and H Jadi “Determination of Dynamic soil Properties using Geophysical Methods” Proceedings of the first international Conference on the application of Geophysical and NDT Methodologies to Transportation facilities and infrastructures, St. Louis, MO,2000

[2] Anderson, D.G and Richart, F.E. “Effects of straining on shear modulus of clays”. ASCE Journal of the Geotechnical Engineering Division, 102(9): 975-987,1976

[3] Stokoe, K.H and Lodde, P.F “Dynamic response of San Francisco Bay mud”. Proc. ASCE special conference on Earthquake Engineering and Soil Dynamics, Pasadena, California, Vol.2:940-959,1978

[4] Hardin, B.O and Drnevich, V. P. “Shear modulus and Damping in soils: Design Equations and curves.” Journal of the soil mechanics and Foundations Division, ASCE, Vol.98, No. SM7,1972

[5] Hardin, B.O and Richart, F.E.Jr “Elastic wave velocities in granular soils.” Journal soil Mechanics and Foundations Division, ASCE, Vol.89, No.SM1, pp.33-65,1963

[6] Hardin, B.O., and Black, W.L “Sand stiffness under various triaxial stresses” J. Soil Mech. and Found. Div., 92(2), 27-42,1966

[7] Seed, H.Bolten and Idriss, I. M “Soil moduli and Damping factors for Dynamic Response analysis,” Report No. EERC 70 – 10, earthquake Engineering Research center, University of California, Berkeley, California,1970

[8] Iwasaki, T. and Tatsuoka, F “Effects of grain size and grading on dynamic shear moduli of sand,” Soils and Foundations, JSSMFE, Vol. 17 No.3, pp. 19-35,1977

[9] Iwasaki, T., Tatsuoka,F., and Takagi, Y. “Shear modulus of sands under cyclic torsional shear loading.” Soils Foundations, JSSMFE, Vol.18, No.1, pp 39-5,1978

[10] Kokusho, T. “Cyclic triaxial test of Dynamic soil properties for wide strain range.” Soils and Foundations, JSSMFE, Vol. 20, No.2, pp. 45-60,1980

[11] Hardin, B.O., and Black, W.L. “Vibration modulus of normally consolidated clay.” Journal of soil mechanics. Foundations Division, ASCE, Vol.94 No.SM2, pp 353-369,1968

[12] Hardin, B.O and Black, W.L. “Closure to vibration modulus of normally consolidated clays,” Journal of soil Mechanics and Foundations Division, ASCE, Vol.95.no.10, pp.831-839,1969

[13] Marcuson, W. E and Wahls, H.E "Time effects on dynamic shear modulus of clays." J. Soil Mech. and Found. Div., 98(12), 1359-1373,1972
[14] Zen, K., Umehara, Y., and Hamada, K. "Laboratory tests and in-situ Seismic survey on vibratory shear modulus of clayey soils with various plasticities."Proc. Fifth Japanese Earthquake Engineering Symp. pp 721—728,1978
[15] Zen, K., Yamazaki, H., and Umehara, Y. "Experimental study on dynamic properties of soils for the use in seismic response analysis." Rep. of Port and Harbour Research Institute, Vol.26, No.1, Port and Harbour Research Institute, Yokosuka, Japan,1987
[16] Yamada, S., Hyodo, M., Orense, R., and Dinesh, S.V "Initial shear modulus of Remolded Sand-Clay Mixtures." Journal of Geotechnical Engineering, ASCE3, Vol.134, No.7, PP. 960-97,2008
[17] ASTM "Standard Test methods for liquid limit, plastic limit, and plasticity index of soils." ASTM D4318-05, Philadelphia,2005
[18] Ishihara, K."Soil behavior in earthquake Geotechnics." Oxford Engineering Science Series. Oxford University press,1996
[19] ASTM D 3999 "Standard Test Methods for Determination of the Modulus and Damping properties of soils Using the cyclic Triaxial Apparatus", American Society for Testing and Materials,1991
[20] Rollins, K.M., Evans, M.D., Diehl, N.B., and Daily, W.D., III. "Shear modulus and damping relationships for gravels." Journal of Geotch. and Geoenviron.Engrg., Vol.124, No.5, PP.396-405,1998
[21] Shibuya, S., Mitachi, T., Fukuda, F., and Degoshi, T "Strain rate effect on the shear modulus and damping of normally consolidated clay." Geotech. Test. J., 18(3), 365-375,2005
[22] Zhang, J., Andrus, R.D., and Juang, C.H. "Normalized shear modulus and material damping ratio relationships." Journal of Geotechnical and Geoenvironmental Engineering, Vol.131, No.4, pp 453-464,2005