Dynamic Properties of Fine Grained Soils in South of Tehran

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ABSTRACT: *Shear modulus, damping ratio and shear wave velocity profiles are important input parameters in site response analysis. For seismic microzonation in south of Tehran, many field and laboratory studies were performed. Field investigations include seismic refraction, down-hole, SASW and SPT while laboratory tests encompass stress controlled cyclic triaxial and resonant column tests on undisturbed samples of low to medium plastic silty materials. This paper presents dynamic properties of fine grained soils in south of Tehran through field and laboratory studies. Based on field geoseismic investigations, new V_s-N(SPT) correlations for fine grained soils are presented. Also laboratory test results reveal that effective confining pressure at stage of consolidation has a remarkable effect on both strain dependent shear modulus and damping ratio of very low plastic soils but by increasing soil plasticity this effect disappears. Evaluation of plasticity effects on deformation properties shows that for PI<12, plasticity index does not have any outstanding effect on these properties, while by increasing PI, shear modulus ratio will increase and damping ratio will decrease.*

Keywords: Fine grained soils; Shear wave velocity; Shear modulus; Damping ratio; Confining pressure

1. Introduction

Tehran with a population of more than 8,000,000 is located in one of the most earthquake prone areas in Iran. Due to the political and socio-economical importance of Tehran, *IIEES* initiated in 1994, a detailed geotechnical microzonation studies for Tehran in two fields: site effects and liquefaction potential microzonation.

In the first step, all of the available geotechnical data were gathered. More than 400 boreholes and 50 deep wells data as well as some geoelecterical profiles for the studied area were collected and processed. Also based on the geoseismic investigations and collected data, a correlation between Standard Penetration Test (*SPT*) and Shear Wave Velocity (V_s) for all types of fine grained soils were developed consistent with Tehran's south geological conditions. In parallel with these investigations, the microtremor measurements were also performed to evaluate the natural period of the site [1, 2].

Since 1998 and in the framework of National Research Projects (*NRP*), the project is going on with two different plans. The first one, which is defined as a complementary study for south of Tehran, aimed to update the existing microzonation maps with a comprehensive field tests including drilling 26 boreholes, in-situ measurements of shear wave velocity through seismic refraction, down-hole and *SASW* methods, and dynamic laboratory tests. In the second plan, designed for North of Tehran, the preliminary microzonation maps will be provided.

In this paper the new V_s - *SPT* correlation and dynamic properties of fine grained soils, obtained from complementary field and laboratory studies in south of Tehran will be presented. In Figures (1) and (2), location of drilled boreholes and geoseismic investigations (including seismic refraction, down-hole and *SASW* methods) are demonstrated respectively.

2. Subsurface Conditions of Studied Area

The studied area mainly consists of sedimentary deposits of Quaternary era, which has been known as Tehran alluvial formation [3]. This formation often is a result of erosion and redeposition of former sediments, which has extended to the south as a young fan and generally consists of unsorted fluvial and river deposits. Both, the effects of climate processes and tectonic young activities caused a miscellaneous alluvium of type, thickness and grain size to be formed. Geotechnical observations of drilled boreholes, whose location has been shown in Figure (1), and also other previous studies reveal that sediments of northern and eastern parts of the studied area are mostly sand and gravel. These cemented coarse grained deposits (except at upper 5*m* band) has a high density and strength. The maximum depth of these deposits has been estimated to be 200*m* [4].

In the middle zone of region, both fine and coarse grained materials have deposited consequently. Furthermore, the deposits grain size decreases with distance from marginal elevations, so that southern part of area

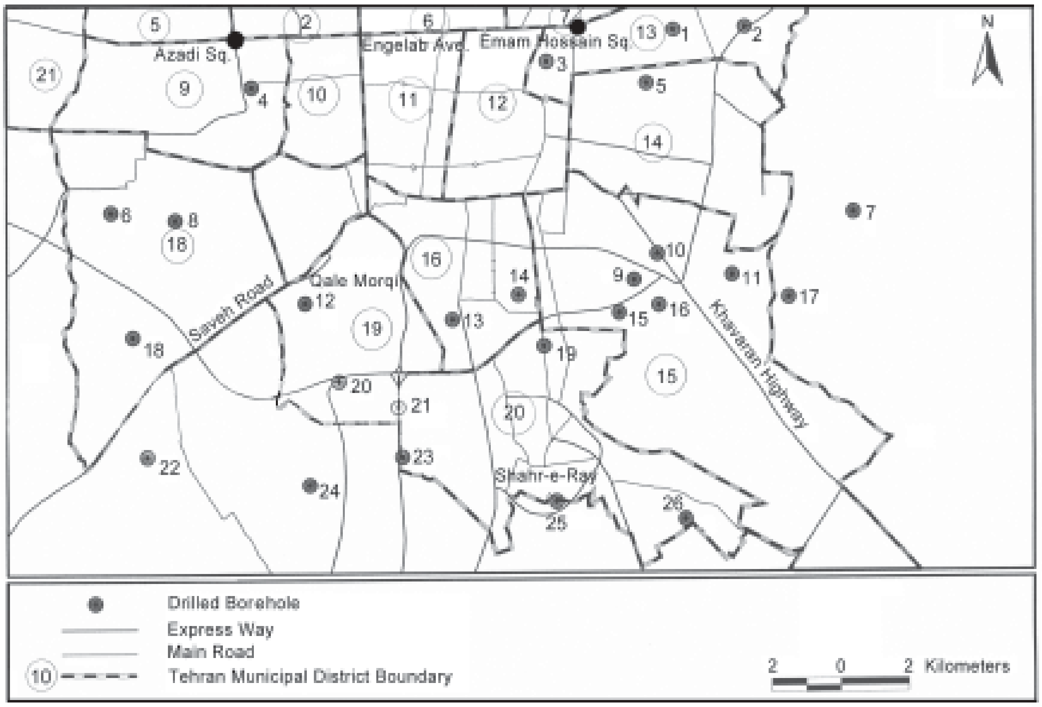

Figure 1. Location of drilled boreholes.

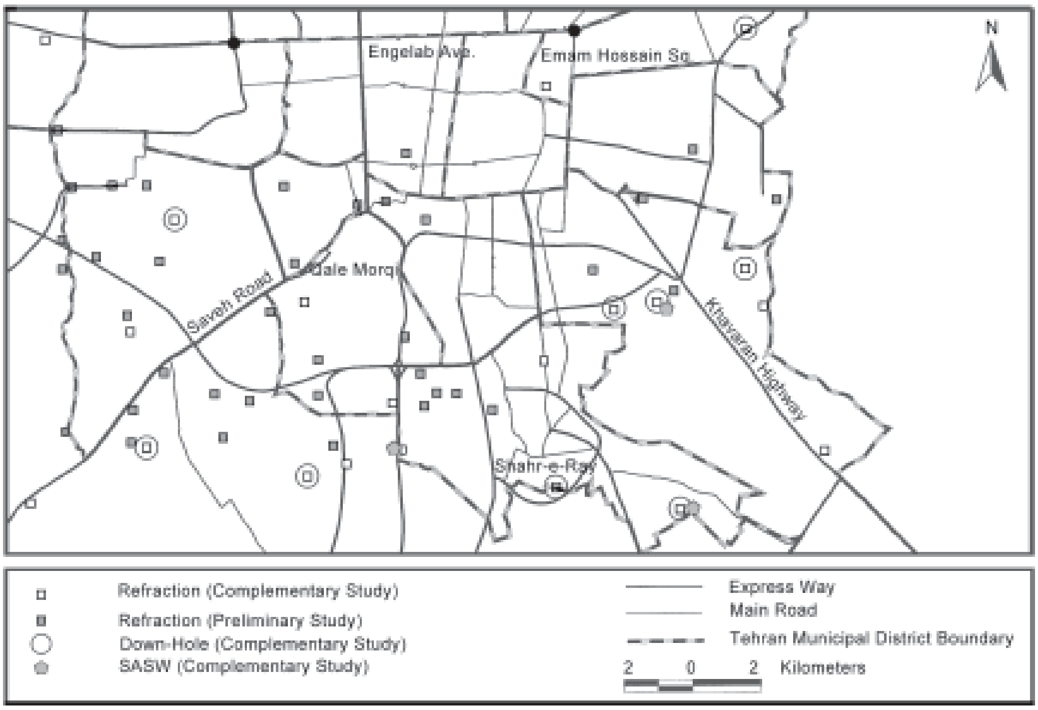

Figure 2. Location of geoseismic investigations.

comprises mostly of low plastic silty and clayey materials. This transformation from coarse to fine grain size is gradually throughout the region. The low plastic silty and clayey materials usually have a plasticity index, *PI* of less than 20 and a fine content (fraction <75 μm) of more than 75%. The maximum thickness of fine grained deposits has been estimated to be 150*m*.

3. Field Geoseismic Investigations

An accurate evaluation of seismic site dependant parameters (e.g. amplification factor and response spectra) needs a proper shear wave velocity profile. Nowadays, the most commonly used seismic methods for velocity logging are the cross-hole and down-hole (or alternatively up-hole). Seismic refraction is another technique which is largely used in determining dynamic properties of the underlying layers. Both the cross-hole and down/up–hole method rely on the measurement of body waves and produce fairly reliable data. However, they require drilling of one or more boreholes. This is practically a drawback for these techniques, as borehole drilling is often costly and time consuming. An alternative technique is *SASW* (Spectral Analysis of Surface Waves) which is based on the measurement of Raleigh waves on the ground surface.

More than 57 seismic refraction were performed in the framework of preliminary and complementary microzonation studies of south of Tehran. Also in the field investigations of complementary microzonation studies, 26 boreholes were drilled accompanied by *SPT*, meanwhile the seismic prospecting by down-hole method at 9 points were performed as well.

Furthermore, as a pilot study, different techniques of shear wave velocity measurement (including *SASW*) were compared at three separate locations. In *SASW*, common receiver midpoint geometry was used for source-receiver configurations [5].

4. Proposed Empirical Correlations for V_s- N(SPT)

Although shear wave velocity could be obtained directly from field investigations or laboratory testing of soil samples of studied area, it is not always economical. Indeed, when the direct measurement of shear wave velocity for soil layers is not available, the existing or developed correlations between *N* values of *SPT* and the shear wave velocity could be used. In this regard, many correlations were proposed as shown in Table (1) [6] whereas Jafari et al [7] and Baziar et al [8] were the first who gave such correlation for Tehran and Iran respectively.

All the aforementioned techniques of V_s measurement were used to develop as accurate as possible a new V_s-*N(SPT)* correlation for the top soil layers of Tehran alluvium. Figures (3) and (4) show the relationship between V_s and *N*-values of all available data. The non-corrected *SPT-N* values were used, since effective-vertical stress-corrected *N*-value (or N_1) is not an appropriate correlative variable to use in estimating V_s [9].

The correlations were developed using a simple linear regression analysis for existing database. Different categories were used to improve the accuracy, since in this case a certain range of void ratio is used [10, 11, 12, 13]. The following relationships are proposed between V_s (*m/s*) and corresponding N - SPT values in three different categories:

$$V_s = 27\,N^{0.73} \quad \textit{Clayey Soils} \qquad (1)$$

$$V_s = 22\,N^{0.77} \quad \textit{Silty Soils} \qquad (2)$$

$$V_s = 19\,N^{0.85} \quad \textit{Fine Grained Soils (Clayey \& Silty)} \qquad (3)$$

All of the obtained correlations of present study are plotted in Figure (5) together with other existing comparable correlations. As seen, the new proposed correlations are more closer to others than previous one proposed by Jafari et al [7]. Meanwhile a difference seen between existing and proposed correlations, specially for *N-SPT* values greater than 30. Even the data base was divided into two categories of the *N-SPT* values of less than 30 and more than 30, but again the exponent of *N-SPT* did not show any considerable difference with the ones proposed by Eqs. (1) to (3). Furthermore the quantity of processed data and procedure of *SPT* may be other causes of the difference. The specific geotechnical conditions of studied area are probably the main cause of difference seen between proposed correlations with the other ones. The soil geological passed history such as aging, overconsolidation, water table fluctuations are the main factors which could affect correlations considerably.

Also, data scattering shown in Figures (3) and (4) is mainly due to different methods of shear wave velocity measurement. Down-hole profile is a point measurement of the subsurface properties, meanwhile these properties are averaged in seismic refraction and *SASW* methods, along a horizontal distance extending approximately in 120*m*. In many geologic environments, there may be significant changes in the subsurface layers over this distance, and these can be reflected in seismic refraction and *SASW* techniques. Another reason is that fine layers, can be detected in *SASW* method, owing to use of wide frequency band of seismic signals, while thin layers and a low velocity layer within the layers of higher velocities can not be recognized in seismic refraction method.

5. Laboratory Testing of Field Samples

Due to possibility of getting intact samples in plastic fine grained soils, only silty and clayey soils were used for dynamic deformation tests. To have a better view on

Table 1. Some existing correlations between V_s -N (SPT).

Author(s)	Soil Type	Vs (m/s)
Kanai, et al (1966)	All	$Vs = 19\ N^{0.6}$
Shibata (1970)	Sand	$Vs = 32\ N^{0.5}$
Ohba & Toriuma (1970)	Alluvial	$Vs = 85\ N^{0.31}$
Ohta, et al (1972)	Sand	$Vs = 87\ N^{0.36}$
Ohsaki & Iwasaki (1973)	All	$Vs = 82\ N^{0.39}$
Ohsaki & Iwasaki (1973)	Cohesionless	$Vs = 59\ N^{0.47}$
Imai & Yoshimura (1975)	All	$Vs = 92\ N^{0.329}$
Imai, et al (1975)	All	$Vs = 90\ N^{0.341}$
Imai (1977)	All	$Vs = 91\ N^{0.337}$
Ohta & Goto (1978)	All	$Vs = 85\ N^{0.348}$
Ohta & Goto (1978)	Sands	$Vs = 88\ N^{0.34}$
Ohta & Goto (1978)	Gravels	$Vs = 94\ N^{0.34}$
JRA (1980)	Clays	$Vs = 100\ N^{1/3}$
JRA (1980)	Sands	$Vs = 80\ N^{1/3}$
Seed & Idriss (1981)	All	$Vs = 61N^{0.5}$
Imai & Tonouchi (1982)	All	$Vs = 97\ N^{0.314}$
Seed, et al (1983)	Sands	$Vs = 56\ N^{0.5}$
Sykora &Stokoe (1983)	Granular	$Vs = 100\ N^{0.29}$
Okamota, et al (1989)	Dilluvial Sands	$Vs = 125\ N^{0.3}$
Lee (1990)	Sands	$Vs = 57\ N^{0.49}$
Lee (1990)	Clays	$Vs = 114\ N^{0.31}$
Lee (1990)	Silts	$Vs = 106\ N^{0.32}$
Imai & Yoshimura (1990)	All	$Vs = 76\ N^{0.33}$
Yokota, et al (1991)	All	$Vs = 121\ N^{0.27}$
Jafari, et al (1997)	All	$Vs = 22\ N^{0.85}$

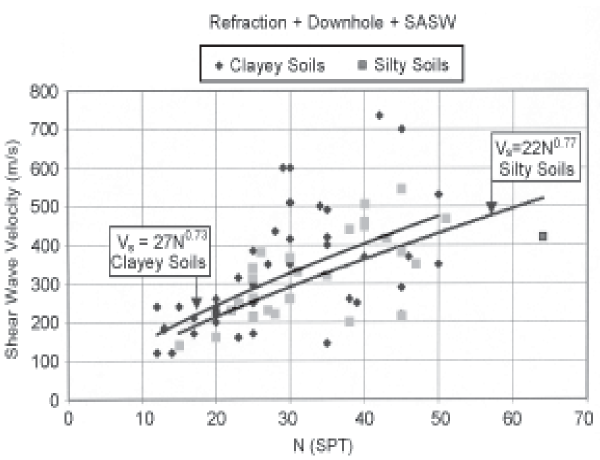

Figure 3. Correlation between V_s and N(SPT) for clayey and silty soils.

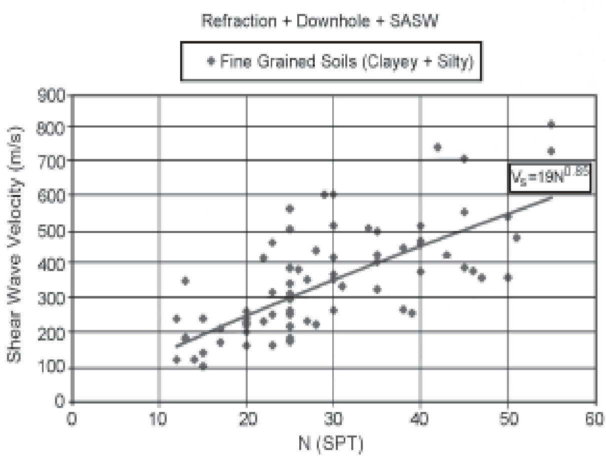

Figure 4. Correlation between V_s and N(SPT) for fine grained soils.

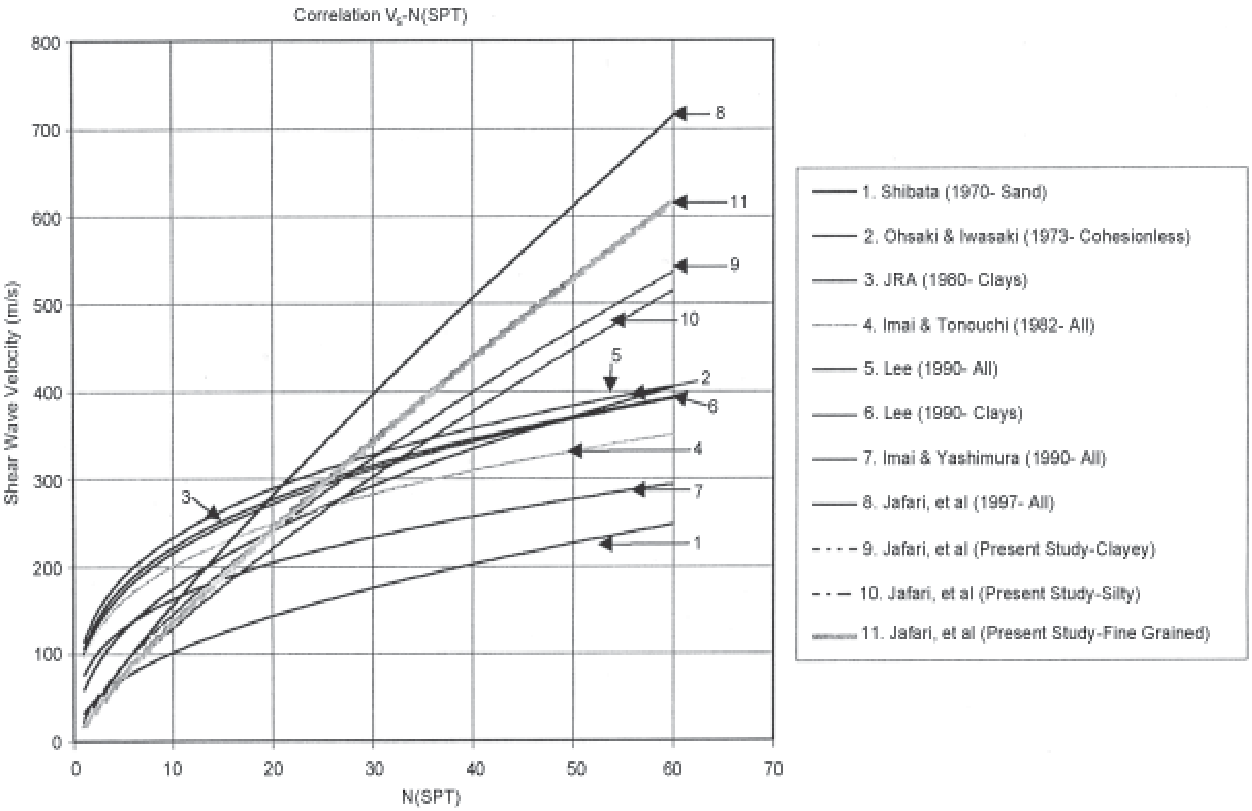

Figure 5. Comparison between proposed correlations for V_s and N(SPT).

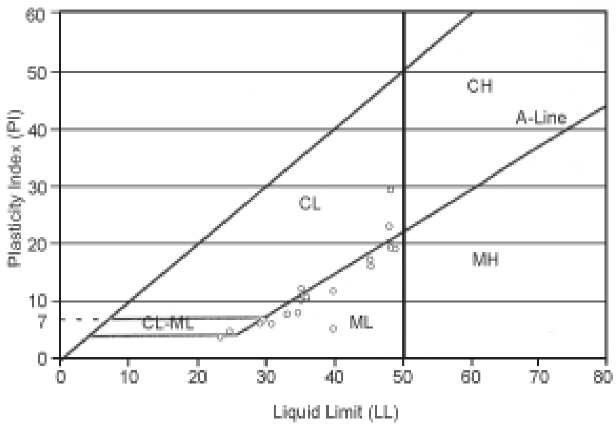

Figure 6. USCS classification of fine grained samples.

types of tested soils, the plasticity characteristics of all samples has been plotted (by hollow circles) on plasticity chart of Unified Soil Classification System (*USCS*) in Figure (6). This figure demonstrates that all the points are approximately adjacent to A-line. Most of the tested soils are *ML*, although a few samples of *CL* and *CL-ML* have been tested.

Laboratory tests on undisturbed samples included index tests, consolidation tests, cyclic undrained triaxial and resonant column tests. All undisturbed samples were obtained using 7.6*cm* outside diameter Shelby tube sampler or 10*cm* outside diameter *U*4 sampler. A specimen diameter of 5*cm* and height of 10*cm* was used for cyclic triaxial tests and for resonant column test a sample diameter of 7*cm* and height of 14*cm* was used. Totally 16 stress controlled cyclic triaxial and 11 resonant column tests were performed.

5.1. Consolidation Stress History

Study of geological history of Tehran sedimentary basin reveals that the encountering soils should be overconsolidated. This can be due to many factors. Severe and repeating season floods may cause many cycles of loading and unloading. Also other mechanisms such as water table fluctuations, desiccation during deposition, aging effects, cementation and upward motion of lower deposits due to Quaternary faults movements are the reasons for Tehran soils to be overconsolidated. Many consolidation tests were performed to determine the consolidation history of south of Tehran soils. The consolidation tests revealed that south of Tehran soils are generally lightly over-consolidated [14]. The results of these tests were used for calculation of at rest earth pressure (K_0), which is a function of overconsolidation ratio (*OCR*).

5.2. Cyclic Triaxial and Resonant Column Tests

In order to simulate site conditions, effective confining pressure, σ'_0, were selected based on in situ vertical effective stress, σ'_v, and stress history through equation

$= (1/3 + 2\,K_0/3)$. Tables (2), (3) and (4) present the boreholes number, samples depth, plasticity index, sample type, effective confining pressure and type of the test. In some tests on samples of similar depths (such as borehole No.13), different pressures were selected for dynamic deformation tests in order to see the effects of confining pressure.

The samples were divided into three groups based on their plasticity. Figure (6) obviously shows that $PI = 7$ is the boundary between *CL-ML* and *CL*. So the tested soils, in terms of their plasticity were divided into three groups: samples of very low plasticity ($PI < 7$), low plasticity ($7 < PI < 12$) and medium plasticity ($15 < PI < 30$). Placticity indexes of 12, 15 and 30 were selected for separation of groups, based on variation of samples plasticity.

All the samples were saturated using a combination of vacuum, de-aired water and a maximum back pressure of 200*kPa* . Saturation phase was continued until a Skempton B-value of at least 0.95 was reached. After saturation, samples were consolidated isotropically in one stage until primary consolidation was ended. Then resonant column and stress controlled cyclic triaxial tests were performed on specified specimens. The resonant column apparatus of *IIEES* geotechnical laboratory is a torsional type with fixed-free end conditions. Resonant column tests were done on samples with a shear strain range of about 10^{-6} to 10^{-4}.

In order to obtain dynamic deformation properties in wider strain range, stress controlled cyclic triaxial tests were performed. A frequency of 1*Hz* was used for applied sine loading, based on *ASTMD* 3999 [15]. At least 10 different amplitudes of cyclic stress was used in multi-stage cyclic loading tests for one test series to obtain the strain dependent shear modulus and damping ratio in the wide range of shear strain amplitude of about 10^{-5} to 10^{-2}.

Figures (7) through (12) show the results of dynamic deformation tests for very low, low and medium plastic materials. The amplitude of shear strain, indicated in the figures is obtained by converting the axial strain, in the cyclic triaxial tests through the relation: $\gamma = 1.5\varepsilon_a$ and for resonant column tests, shear strain amplitude is calculated at 80 percent of sample radius [16]. The shear modulus, G in cyclic triaxial tests was computed in 11^{th} cycle of loading according to Kokusho [17] who noted that the effect of the number of cycles practically disappears when the stress application is repeated for more than 10 cycles. The initial modulus, G_0 used for normalization of shear modulus, has been computed from resonant column test results in low levels of shear strain (approximately at 6×10^{-6}).

6. Discussion on Laboratory Test Results

Figures (7) to (12) can be used for evaluation of confining pressure effects on dynamic deformation properties of tested materials. Figure (7) clearly shows that for very low

Table 2. Testing plan for very low plastic samples (PI < 7).

Borehole No.	Sample Depth. (m)	PI	Sample Type	σ'_0 (kPa)	Test Type
26	8.4	6.6	ML	150	C.T.*
26	14.7	6.0	ML	200	C.T.
16	27.1	5.0	ML	300	R.C.**
14	18.6	3.7	ML	200	C.T. & R.C.
25	10.6	4.7	CL-ML	100	C.T.
13	21.8	5.9	ML	300	C.T.

* Cyclic Triaxial Test.
** Resonant Column Test.

Table 3. Testing plan for low plastic samples (7 < PI < 12).

Borehole No.	Sample Depth. (m)	PI	Sample Type	(kPa)	Test Type
26	26.7	7.8	ML	250	C.T.
16	20.5	11.5	ML	150	C.T.
25	24.3	7.6	ML	250	C.T. & R.C.
20	9.0	12.0	CL	100	R.C.
20	14.2	10.3	ML	150	C.T. & R.C.
20	17.3	10.5	ML	300	R.C.
12	6.6	10.0	ML	100	C.T.
12	10.7	10.6	ML	150	C.T.
12	10.7	10.6	ML	300	C.T.
13	12.7	10.6	ML	200	C.T.

Table 4. Testing plan for medium plastic samples (15 < PI < 30).

Borehole No.	Sample Depth. (m)	PI	Sample Type	(kPa)	Test Type
19	26.1	22.8	CL	250	R.C.
20	42.6	15.8	ML	450	C.T. & R.C.
20	49.5	17.0	ML	500	R.C.
13	42.2	19.0	ML	600	R.C.
13	46.0	18.8	ML	500	C.T. & R.C.
13	53.5	29.1	CL	450	C.T.

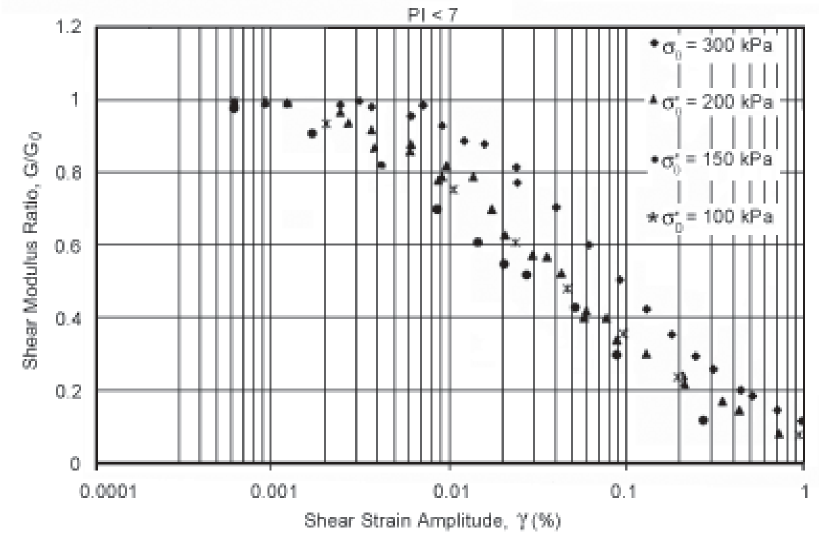

Figure 7. Shear modulus ratio of very low plastic soils at different confining pressures.

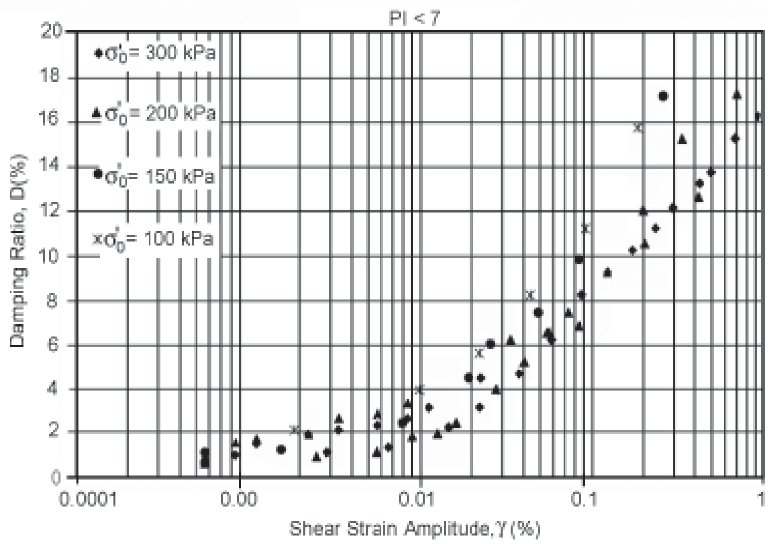

Figure 8. Damping ratio of very low plastic soils at different confining pressures.

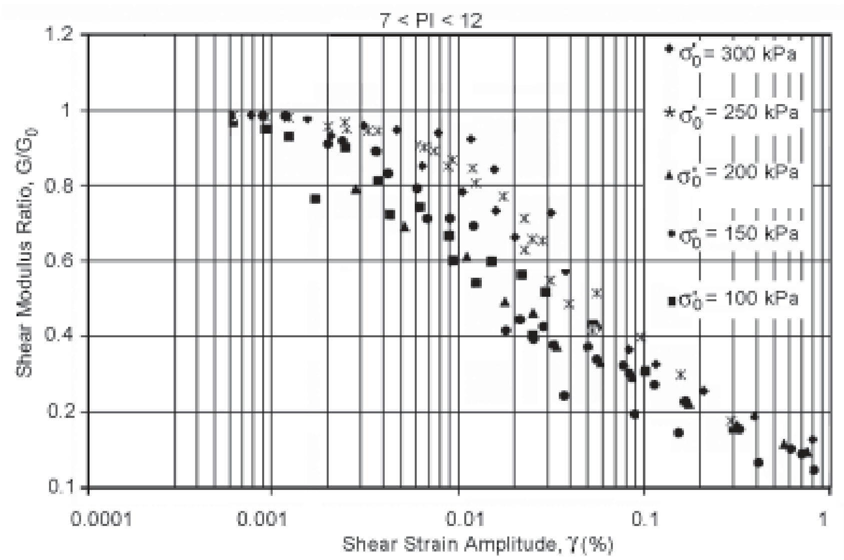

Figure 9. Shear modulus ratio of low plastic soils at different confining pressures.

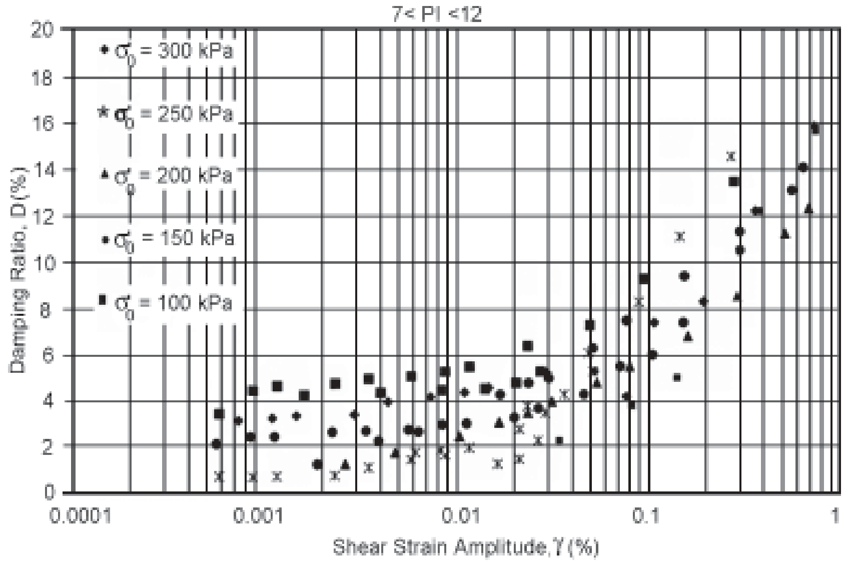

Figure 10. Damping ratio of low plastic soils at different confining pressures.

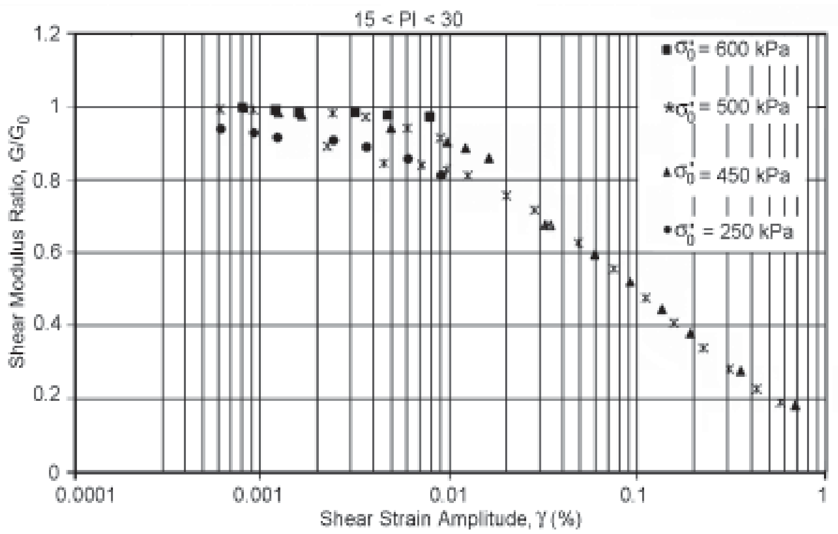

Figure 11. Shear modulus ratio of medium plastic soils at different confining pressures.

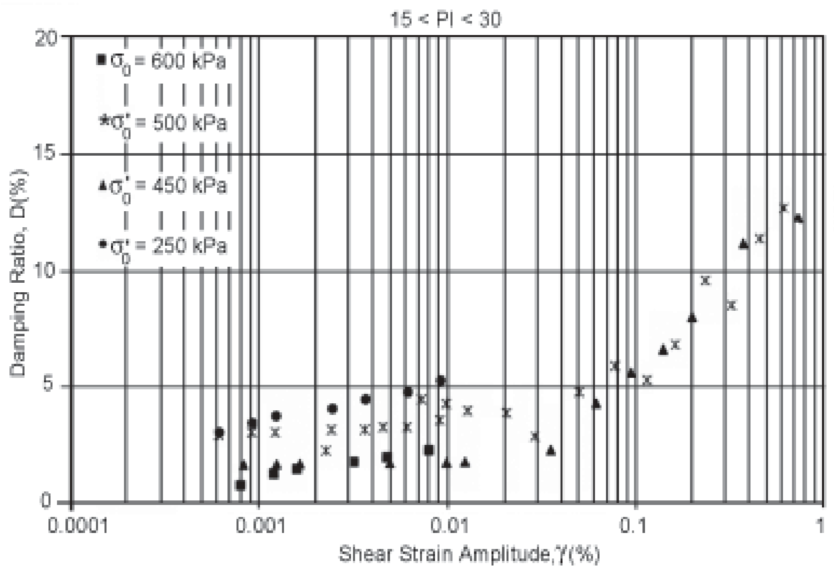

Figure 12. Damping ratio of medium plastic soils at different confining pressures.

plastic soils by decreasing confining pressure, shear modulus ratio will decrease. Since data for confining pressures of 100 and 150*kPa* (Table (2)) has been gathered only from one test, so it seems more data is needed to specify behavior at these confining pressures. Also this phenomena can be seen for damping ratio (Figure (8)) but with less effects of confining pressure. As seen, increasing confining pressure will cause damping ratio to decrease. Test results for low plastic soils have been plotted in Figures (9) and (10). Figure (9) implies that shear modulus ratio at confining pressures of 250 and 300*kPa* is more than other low ones, while there is not any remarkable effect on shear modulus at low confining pressures. Furthermore Figure (10) shows that values of damping ratio are approximately independent of confining pressures for low plastic soils. Behavior of medium plastic soils has been presented through Figures (11) and (12). According to these figures, no important effect of confining pressure on shear modulus and damping ratio can be seen, although it seems more data is needed. So it seems reasonable to assume that for the silty and clayey soils, the effects of confining pressure disappears by increasing soil plasticity.

In order to evaluate the effect of plasticity on behavior, all of the data have been plotted on one figure (Figure (13) for shear modulus ratio and Figure (14) for damping ratio). Figure (13) shows that shear modulus ratio of medium plastic soils is more than others, while it seems plasticity in the range of very low to low does not have any outstanding effect on shear modulus ratio. This trend can be seen in damping ratio (Figure (14)) in which damping ratio of medium plastic soils is less than others,

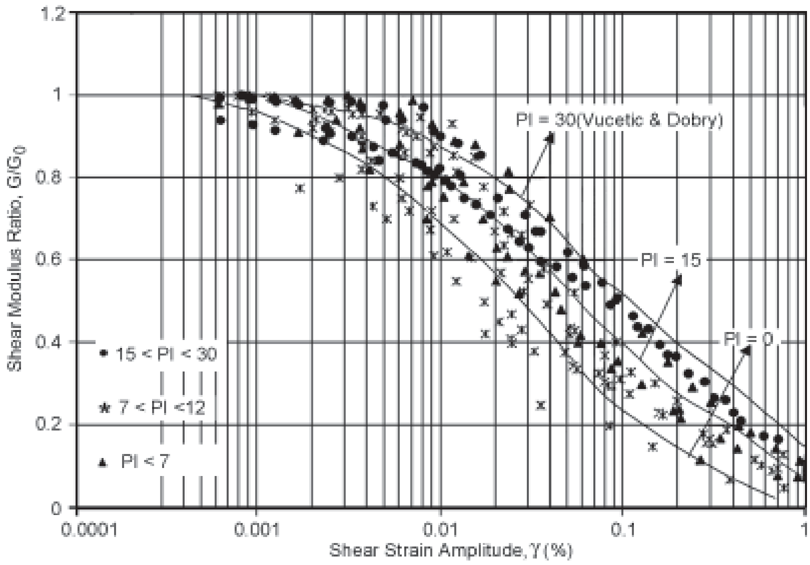

Figure 13. Shear modulus ratio of all tested materials.

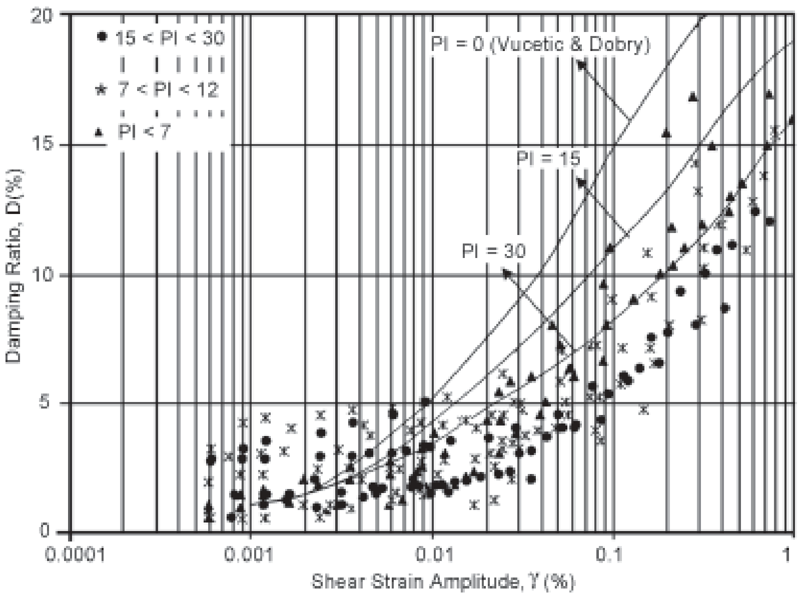

Figure 14. Damping ratio of all tested materials.

of course with more data scattering. It should be noted that confining pressure of medium plastic soils (Table (4)) is usually more than other soil types and so increase in shear modulus and decrease in damping ratio of this soils may be due to confining pressure to some extent.

Finally a comparison has been made with curves of Vucetic and Dobry [18], as shown in Figures (13) and (14) for plasticity indexes of 30, 15 and 0. Figure (13) shows that, test results for medium plastic soils ($15 < PI < 30$), nearly falls between curves of $PI = 15$ and $PI = 30$, although some scattering exists for lower plasticity indexes. Also, Vucetic and Dobry curves for damping ratio (as shown in Figure (14)) do not coincide with test results. This may be because of reference data for Vucetic and Dobry curves which mainly consists of test results on sands and clays while the material used in this study is mainly low plastic silt.

7. Conclusions

The south of Tehran alluviums consists of variety of soils from coarse to fine grained ones. An attempt has been made to evaluate dynamic properties of the fine grained soils through field geoseismic investigations and dynamic laboratory tests.

New -*N(SPT)* correlations were proposed in three categories of clayey, silty and fine grained soils. Although it seems that the existing scattering in data is due to different methods of shear wave velocity measurements. Also the new proposed correlations are more consistent with other existing correlations.

Analysis of the dynamic deformation tests, including stress controlled cyclic triaxial and resonant column tests suggests that

- ❖ Effective confining pressure at stage of consolidation has a pronounced effect on both strain dependent shear modulus and damping ratio of very low plastic soils ($PI < 7$) and increasing confining pressure will cause shear modulus ratio to increase and damping ratio to decrease;
- ❖ Effects of confining pressure on dynamic deformation properties disappears with plasticity index increasing into low and medium plastic soils;
- ❖ Shear modulus ratio of medium plastic soils is more than very low and low plastic one, while its damping ratio is lower;
- ❖ In the range of very low to low plastic soils, plasticity index does not have any important effect on shear modulus and damping ratio.

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