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ハノイ市における繊維質材料混合流動化処理土の埋戻し地盤への適用に関する研究

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Suggestion of Methods for Estimation of Soil Dynamic Parameters in Conformity with Current Condition in Vietnam

5.1 INTRODUCTION

Ground-borne vibration from dynamic loading induced by moving train propagates in soil layers under shear wave. Ground is a complicated multilayered medium described by elaborated theoretical models and numerical analysis with considerably effort. Moreover, with condition of Vietnam at present, the shortage of data of soil dynamic parameters makes the problems become more complicated to simulate. Therefore the models which have been made sophisticated are difficult to apply due to indeterminacy of the parameters. Consequently it is necessary to determine the dynamic parameters of ground in order to solve the ground-borne vibration problems. Adequate information on dynamic soil properties, especially dynamic shear modulus and damping ratio, is essential for accurate computations of ground respond and soil-structure interaction problems.

5.2 SOIL DYNAMIC PARAMETER AND ESTIMATING METHODS

The dynamic parameters of ground influence considerably the results of problem. Moreover the parameters are not the same in a soil kind, in a soil layer at each position it is also different. the factors such as granular composition, structural form, thickness of stratum can amplify the amplitude of vibration of ground through the elastic parameters

such as elastic modulus E , Poisson's ratio ν , shear modulus G , damping ratio h . The parameters relate to compression wave velocity (v_p) and shear wave velocity (v_s) of soil, G , E , ν , by the following equations (Das, 1995; Kramer, 1996; Verruijt, 1994):

Shear modulus:

$$G = \rho \cdot v_s \quad (5.1)$$

Elastic modulus:

$$E = \frac{\rho \cdot v_s^2 (3v_p^2 - 4v_s^2)}{v_p^2 - v_s^2} \quad (5.2)$$

Poisson's ratio:

$$\nu = \frac{v_p^2 - 2v_s^2}{2(v_p^2 - v_s^2)} \quad (5.3)$$

The parameters can be determined by field and laboratory method presented in available literatures (Das, 1995; Kramer, 1996; Karl, 2005; Schneider et al., 1999). A representation of these test procedures is shown in Figure 1.

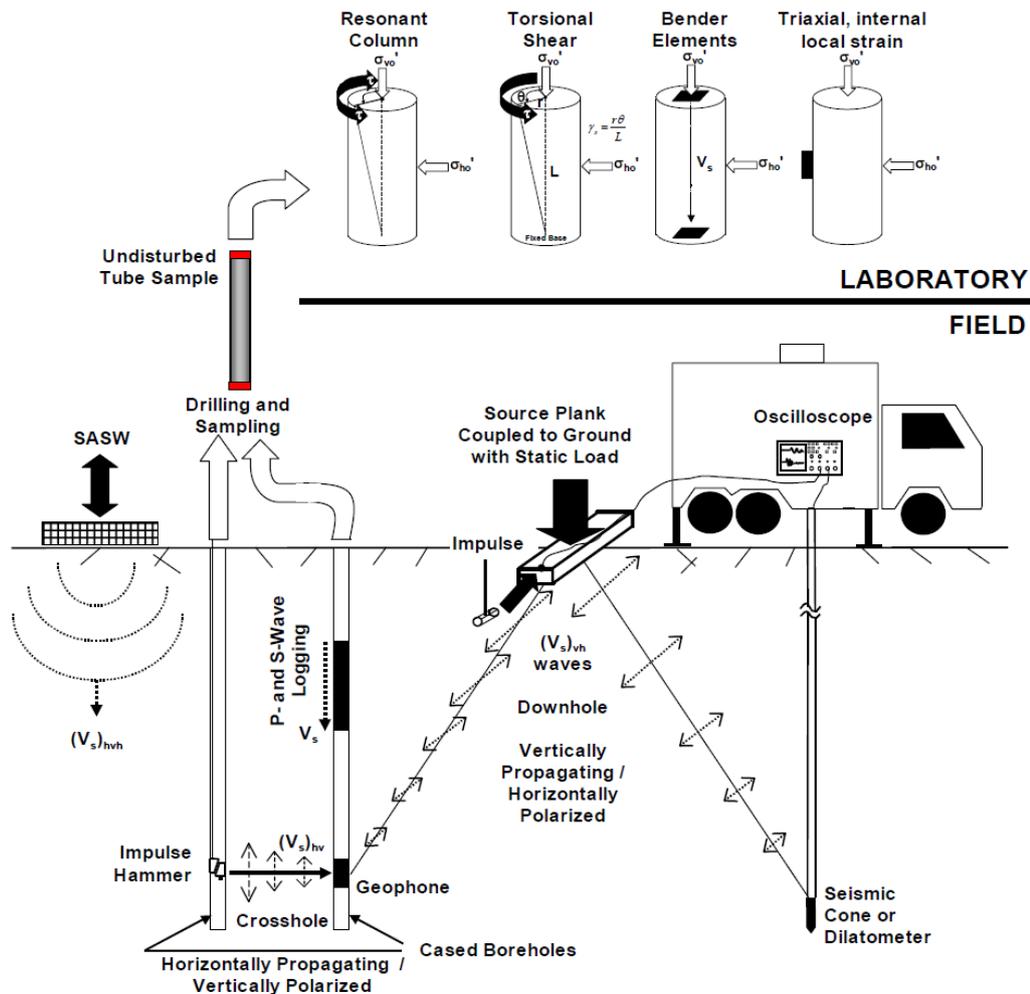


Figure 5.1 Field and laboratory methods for determining dynamic parameters

A wide variety of field and laboratory techniques are available in the world, each with different advantages and limitation with respect to different problems. Laboratory tests generally provide the most direct means of evaluating dynamic soil parameters for seismic analyses (Kavazanjian et al., 1997). These laboratory tests are classified as large strain range, medium range and small strain range test. Since the soil is nonlinear material it could be tested under wide range of strain.

The laboratory tests are carried out for the samples that represent a soil block. This is limitation of the tests due to the difficulty to simulate a condition which is similar to the in-situ situation such as stress, strain. Thus, at present in Vietnam no laboratory test may describe accurately the stain-stress relation of in-situ soils. Therefore, each test is applied to a specific circumstance. The field tests are more advantages than the laboratory tests that require the complicated equipment. Moreover the factors are close to actual conditions. However the cost for the tests is high.

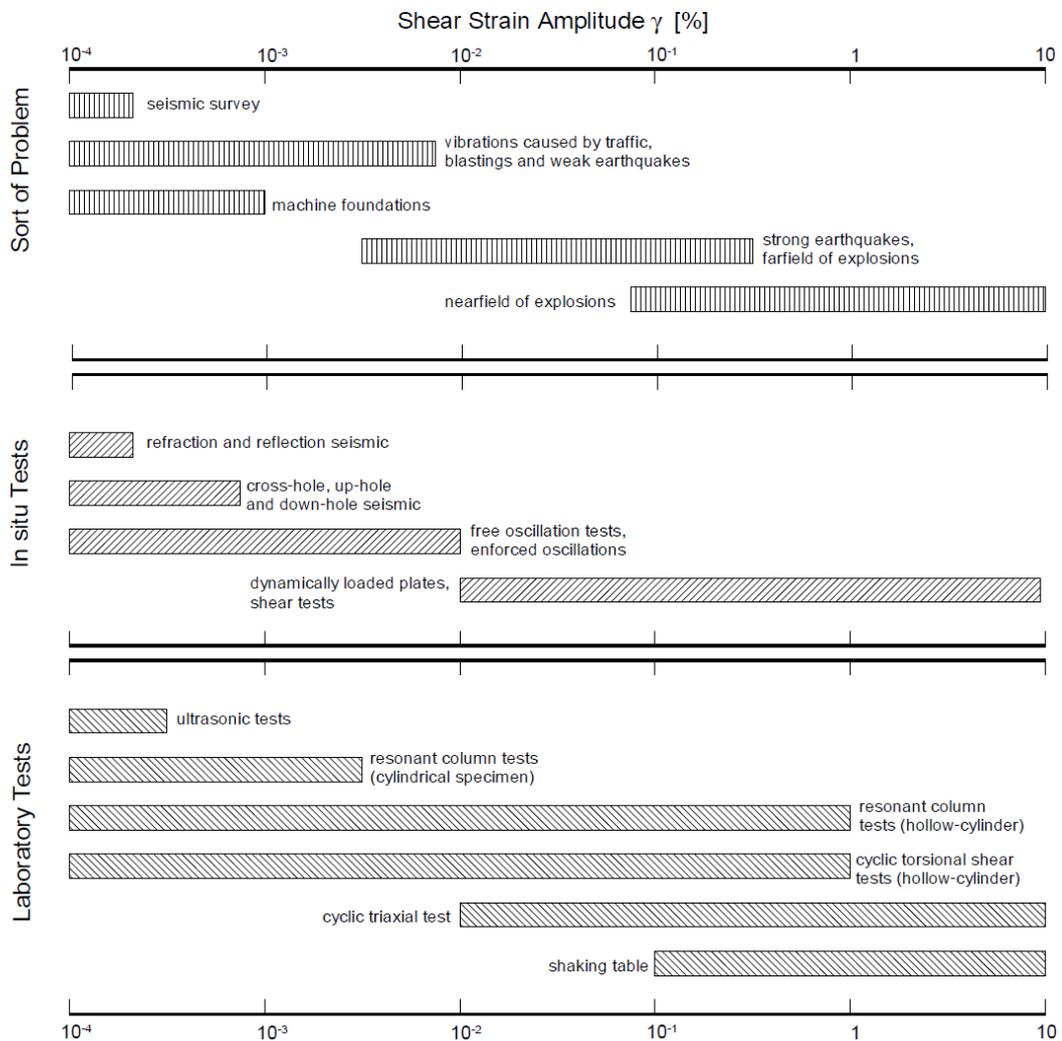


Figure 5.2 Overview of possible shear strain amplitudes

From the literatures review regarding to estimation of soil dynamic parameters (Das, 1995; Kramer, 1996; Karl, 2005; Schneider, 1999) it is found that the parameters is strongly depended on the shear strain amplitude γ , therefore when estimating the

parameters, the following three factors based on application region for dynamic tests of Karl (1999) should be considered (Figure 5.2):

- Application range of dynamic parameters in order to determine γ
- The suitable test kind for shear strain amplitude γ
- Possibility for performance of dynamic tests

From the analysis of methods determining the ground dynamic parameters, the procedures should be met the following requirements:

- Requirements of equipment and facilities for the tests
- Requirement of professional skill of engineer
- Requirement of experience of engineer

From the current situation of Vietnam, it is realized that:

- Due to the financial difficulties, almost project consider only in static problem without using the dynamic parameters
- From the point of view of commercial efficiency, expenses for investment of the dynamic tests is very expensive, so hardly laboratory can equip if they have no financial support from non-profit organization. For example, University of Transport Technology, Hanoi, Vietnam has been supported the finance of 6.5 million dollars by Japan International co-operation Agency (JICA) to invest the equipment for their laboratory but it has no apparatus for determining the dynamic parameters of soil specimen.
- In Vietnam, The standard of geotechnical investigation is for calculation and design of the static problems. Moreover the testing standards for the dynamic problems with fully legal corridor have not been promulgated to promote this field unless the special projects or works must refer to other standards from foreign country.

Therefore, it is necessary to select a suitable method for estimating the ground dynamic parameters in Vietnam. The objective of this chapter is to establish a procedure based on the methods of previous researches in the world for estimating the ground dynamic parameters which is suitable for current condition of Vietnam.

Recently, Andrus et al. (2003) have developed the procedures for estimating dynamic properties of soils in South Carolina of America. When site-specific shear wave velocity, v_s , are not available or need to be supplemented, an estimation of the shear wave velocity, v_s , can be made by the use of correlations with in-situ testing such as the Standard Penetration Test (SPT) or the Core Penetration Test (CPT). The procedures for correlating SPT and CPT results with shear wave velocity, v_s , have been summarized in design standard, Geotechnical Earthquake Engineering, South Carolina Department of Transportation (2008).

The unified formulas which express the dynamic shear moduli and the damping ratios in terms of maximum dynamic shear modulus, cyclic shear strain amplitude, mean effective confining pressure and soil's plasticity index have been proposed by Ishibashi, Zhang (1993). The formulas fitted experiment data reasonably well and could be conveniently utilized in dynamic analyses such as seismic ground respond and

soil-structure interaction problems. The formulas cover wide variety of soils ranging from sands to highly plastic clays.

5.3 ESTIMATION OF DYNAMIC SOIL PROPERTIES

5.3.1 Estimation of shear wave velocity, v_s , from SPT data

Recommended equations to estimate shear wave velocity, v_s , are based on standardized SPT blow count (N_{60}^*), depth (Z), Fines Content (FC), geologic age and location of deposit, and Age Scaling Factor (ASF). Equations for estimating shear wave velocity, v_s , are provided in Table 5.1.

Table 5.1 SPT (N_{60}^*) – Shear Wave Velocity, v_s , Equation for Sand (Andrus et al., 2003)

Fines Content, FC	Equation for predicting v_s (m/s)	Equation No
< 40%	$v_s = 72.9(N_{60}^*)^{0.224}Z^{0.130}ASF$	5.4
10% to 35%	$v_s = 72.3(N_{60}^*)^{0.228}Z^{0.152}ASF$	5.5
<10%	$v_s = 66.7(N_{60}^*)^{0.248}Z^{0.138}ASF$	5.6

N_{60}^* =blows/0.3m and Z =depth in meters, ASF =Age Scaling Factors.

Recommended age scaling factors (ASF) based on Andrus et al. (2003) are provided in Table 5.2.

Table 5.2 Recommended Age Scaling Factors (ASF) for SPT (Andrus et al., 2003)

Geologic Age and Location of Deposit	Fines Content, FC (%)	Age Scaling Factor, ASF	Database Range of Shear Wave Velocity, v_s (m/s)
Holocene Coastal Plain	< 40%	1.00	110 – 260
	10% to 35%	1.00	120 – 240
	< 10%	1.00	110 – 260
Pleistocene Coastal Plain	< 40%	1.23	150 – 270
	10% to 35%	1.08	160
	< 10%	1.28	150 – 270
Tertiary Coastal Plain Ashley Formation	< 40%	1.82	340
	10% to 35%	1.71	340
Tertiary Coastal Plain Dry Branch Formation	< 40%	1.59	330 – 350
	10% to 35%	1.48	330 – 350

FC =% passing #200 sieve

The procedures of estimating shear wave velocity, v_s , from SPT data are following:

1. Estimating FC(%) of soil layer at SPT depth
2. Calculating SPT blow count (N_{60}^*) and Age Scaling Factor (ASF) based on Fine Content (FC) at each depth of SPT (Z)
3. Estimating v_s , from (5.4), (5.5), (5.6) equation with FC(%) and correlation depth of SPT (Z)
4. Drawing relation between v_s and Z

A review of SPT calculated shear wave velocity relationships reveals that few relationships have been developed for clays. This is likely due to SPT blow counts (N) not being the appropriate test for cohesive soils, particularly since soft clays would have SPT blow counts that would be close to zero. This results in the shear wave velocity that also would be close to zero.

5.3.2 Estimation of shear wave velocity, v_s , from CPT data

Recommended equations to estimate shear wave velocity, v_s , for soils are based on CPT tip resistance (q_c), depth (Z), soil behavior type (I_c), geologic age and location of deposit, and Age Scaling Factor (ASF). Equations for estimating shear wave velocity, v_s , of soils provided in Table 5.3.

Table 5.3 CPT (q_c) – Shear Wave Velocity, v_s , Equations for Soils
(Andrus et al., 2003)

Soil Behavior Type, I_c	Equation for Predicting v_s (m/s)	Equation No
All value	$v_s = 4.63q_c^{0.342}I_c^{0.688}Z^{0.092}ASF$	5.7
< 2.05	$v_s = 8.27q_c^{0.285}I_c^{0.406}Z^{0.122}ASF$	5.8
> 2.6	$v_s = 0.208q_c^{0.654}I_c^{1.910}Z^{-0.108}ASF$	5.9

Recommended age scaling factors (ASF) depended on soil behavior type (I_c) and depth (Z) are provided in Table 5.4 (Andrus et al., 2003)

The soil behavior index, I_c depended on the normalized cone resistance, Q, and the normalized friction ratio, F, are computed using the following equation:

$$I_c = [(3.47 - \log Q)^2 + (1.22 + \log F)^2]^{0.5} \quad (5.10)$$

Where:

$$Q = \left[\frac{q_c - \sigma'_v}{P_a} \right] \cdot \left(\frac{P_a}{\sigma'_v} \right)^n \quad (5.11)$$

$$F = \left[\frac{f_s}{q_c - \sigma'_v} \right] 100\% \quad (5.12)$$

Where:

q_c – CPT Tip Resistance (kPa)

F_s – CPT Skin Resistance (kPa)

P_a – Reference Stress = 100 kPa = 1 atm

σ'_v - Effective Vertical or Overburden Stress (kPa)

n – Exponent ranging from 0.5 to 1.0 depended on computing procedure of soil behavior index.

Table 5.4 Recommended Age Scaling Factors (ASF) for CPT
(Andrus et al., 2003)

Geologic Age and Location of Deposit	Soil behavior Description	Soil Behavior Type Index, I_c	Age Scaling Factor, ASF	Database Range of Shear Wave Velocity, v_s (m/s)
Holocene Coastal Plain	All soils	All values	1.00	60 – 260
	Clean sand, silty sand	< 2.05	1.00	110 – 260
	Clay, silty clayey silt, silty clay	> 2.60	1.00	60 – 230
Pleistocene Coastal Plain	All soils	All values	1.23	450 – 1000
	Clean sand, silty sand	< 2.05	1.34	500 – 1000
	Clay, silty clayey silt, silty clay	> 2.60	1.16	450 – 1000
Tertiary Coastal Plain Ashley Formation	All soils	All values	2.29	230 – 540
Tertiary Coastal Plain Tobacco road Formation	All soils	All values	1.65	310 – 350
	Clay, silty clayey silt, silty clay	> 2.60	1.42	330 – 350
Tertiary Coastal Plain Dry branch Formation	All soils	All values	1.38	310 – 360
	Clean sand silty sand	< 2.05	1.33	310 – 360

The procedure for estimating the soil behavior index is following:

1. Calculate soil behavior index, I_c , using equation (5.10) with $n=1.0$ to have $I_c=I_{c1}$
2. If soil behavior index, I_c , is > 2.60, use computed I_c using $n=1.0$
3. If soil behavior index, I_c , is < 2.60, recalculate I_c using $n=0.50$
 - a. If the recalculated I_c is < 2.60, use computed I_c using $n=0.50$
 - b. If the recalculated I_c is > 2.60, recalculate I_c using $n=0.70$
4. End.

In brief, the estimating procedures of shear wave velocity, v_s , from CPT data are following:

1. Obtain q_c (kPa) and f_s (kPa) at depth, Z , of CPT
2. Calculate σ'_v (kPa) at depth, Z ,
3. Calculate soil behavior index I_c at depth, Z , using equation 5.10
4. Use Table 5.4 to obtain age scaling factor (ASF) from correlated I_c
5. Estimate v_s using 5.7, 5.8, or 5.9 from correlated I_c

5.3.3 Estimation of damping ratio

Damping ratio is a measure of energy dissipation and increases with increase in shear strain amplitude. The damping ratio increases slowly at a low strain level and then increases quickly with increasing strain amplitude. Damping is induced in soil due to the friction between the soil grains, viscous drag between the pore fluid and soil grains, and plastic deformation of the soil grains. The soil grains are very stiff and tend to keep their elasticity at high confining pressure, and thus very little energy dissipates through plastic deformation. In general, the damping ratio of soil is affected by a lot of factors (Ishibashi and Zhang, 1993; Karl, 2005; Zhang et al., 2005) such as effective confining pressure, void ratio, plastic index, stratum age, and loading condition. Therefore accurate estimation of the damping ratio considered all factor is a complex work. In 1993, Ishibashi and Zhang have proposed an equation for estimating the damping ratio of soil based on plastic index, effective confining pressure, and shear strain amplitude. The authors reanalyzed the available experimental data and an attempt is made to establish unified formulas for dynamic shear moduli and damping ratio to cover wide variety of soils ranging from sands to highly plastic clay.

The formula for estimation of damping ratio is expressed as following:

$$h = 0.333 \frac{1+e^{-0.0145 \cdot PI^{1.3}}}{2} \left[0.586 \left(\frac{G}{G_{max}} \right)^2 - 1.547 \left(\frac{G}{G_{max}} \right) + 1 \right] \quad (5.13)$$

where:

$$\left(\frac{G}{G_{max}} \right) = K(\gamma, PI) \bar{\sigma}_0^{m(\gamma, PI) - m_0} \quad (5.14)$$

PI: Index plastic of soil (%)

γ : Medium shear strain amplitude (%)

$\bar{\sigma}_0$: mean effective confining pressure

$$K(\gamma, PI) = 0.5 \left\{ 1 + \tanh \left[\ln \left(\frac{0.000102 + n(PI)}{\gamma} \right)^{0.492} \right] \right\} \quad (5.15)$$

$$m(\gamma, PI) - m_0 = 0.272 \left\{ 1 - \tanh \left[\ln \left(\frac{0.000556}{\gamma} \right)^{0.4} \right] \right\} e^{-0.0145 PI^{1.3}} \quad (5.16)$$

$$n(PI) = \begin{cases} 0.0 & \text{for } PI = 0 \text{ (sandy soil)} \\ 3.37 \times 10^{-6} PI^{1.404} & \text{for } 0 < PI < 15 \text{ (low plastic soils)} \\ 7.0 \times 10^{-7} PI^{1.976} & \text{for } 15 < PI \leq 70 \text{ (medium plastic soils)} \\ 2.7 \times 10^{-5} PI^{1.115} & \text{for } PI > 70 \text{ (high plastic soils)} \end{cases} \quad (5.17)$$

Using the above formulas to calculate the damping ratio for sandy soil and medium plastic soils with PI of 35 % in case of $\bar{\sigma}_0 = 10$ kPa and 400 kPa, respectively, the results are shown in Figure 5.3 and 5.4.

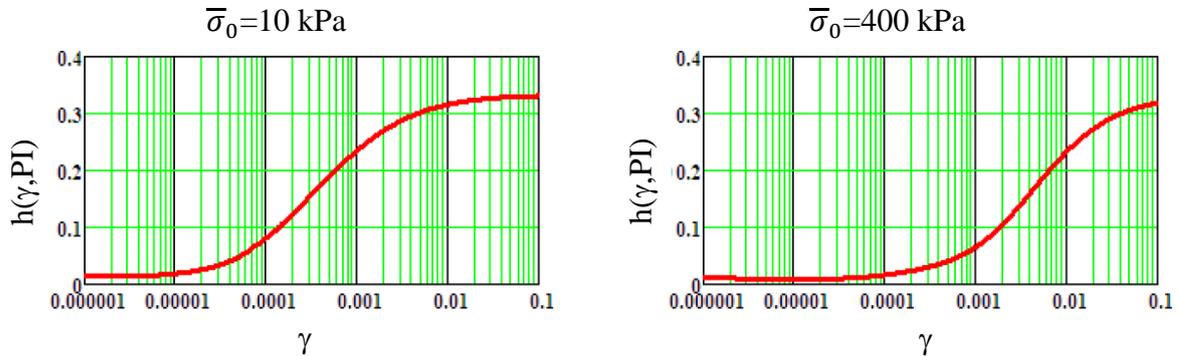


Figure 5.3 $h \sim \gamma$ relation of sandy soils (PI = 0 %)

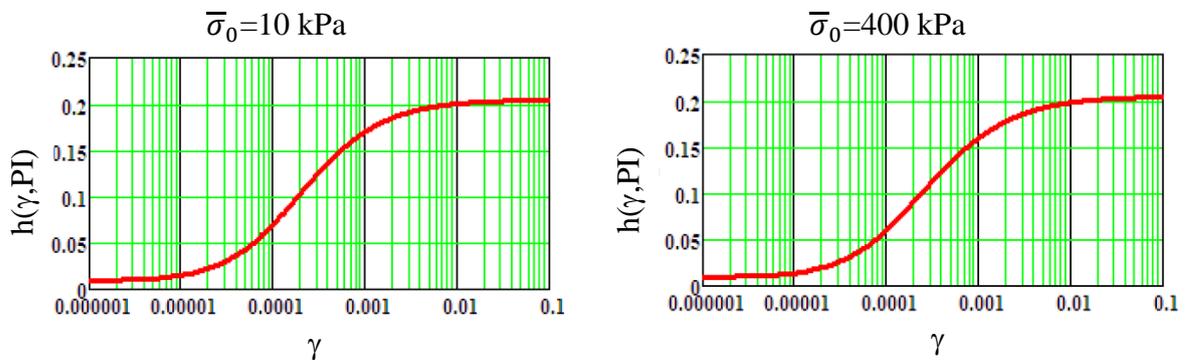


Figure 5.4 $h \sim \gamma$ relation of plastic soils (PI = 35 %)

5.4 ESTIMATION OF SOIL DYNAMIC PARAMETERS FOR METRO LINE No.3 IN HANOI CITY

Hanoi City is located in the center of North Vietnam. It is a metropolis developed in Red River Delta – one of great delta in Southeast Asia. This position is about 100 km upstream from the mouth of the Red River. The Red River flows from northwest to southeast through mountainous region, then branch off in plain area.

The flow direction of Red River is NW-SE caused by Red river fault system in glacial period. Sedimentation basin size is approximately 500 km-long, 50-60 km-wide and more than 3 km of the sedimentation thickness containing piled sediments up to glacial and Fourth period. Fourth period's sediment is sand and gravel, inserted with lens of silk or clay soil. The sedimentation layer in the Fourth period's after latest glacial period is composed of three layers. The order of accumulation is Vinh Phuc layer, Hai Hung layer and Thai Binh layer (Refer to Table 5.5).

Table 5.5 Geological classification
(Other Till et al., 2005)

Age (cal. kyr BP)	Epoch	Mathers et al. (1996) Mathers and Zalasiewicz (1999) (thickness)	Sediment character	Tran and Ngo (2000)	VN core (thickness)	GA Core (thickness)
0	Holocene	Thai Binh Fm. (ca.30m)	Gravel, sand and clay with an upward-fining character	Q_{IV}^3	Unit 2 (29.8m)	Unit 2 (36.6m)
3.7 4		Haihung Fm. (ca.30m)	sand	Q_{IV}^2		
7		Vinhphuc Fm. (ca.30m)	Gravels and clay with an upward-fining character	Q_{IV}^1	Unit 1 (5.2m)	Unit 1 (3.6m)
8.5 9.8 10 12	Latest Pleistocene			Q_{III}		

Hanoi metro No.3 is a part of the Hanoi metro rail system project, with starting point at Nhon town, Tu Liem district and ending point at Hanoi railway station. In general, the geology condition of the line is good and stable in the starting area and weak in the ending area. Especially, the area from station No6 to the ending point, around 5 m depth from ground, appear a very weak soil layer with thickness from 10 m to 12 m (layer 1: void ratio of 2.07; water content of 77 %; plastic index of 34 % and degree of saturation of 97.5 %).

In the geological survey report of the metro line No.3, currently there is no the data of the soil dynamic parameters. Moreover the soil layers in the area of the line are almost soft soil specifying as clay, sandy. Therefore, application of method of estimation of shear wave velocity, v_s , from SPT data is not reasonable in this case (SCDOT, 2010). Thus, to solve the problem this study used the method of estimation of shear wave velocity, v_s , from CPT data as aforementioned.

The procedures for estimation of the soil dynamic parameters from CPT data are performed as following:

1. Collect data from geological and stratum survey report
 - Estimation of stratum age
 - Estimation of borehole log and result of CPT
 - Physical properties of soil
2. Estimate shear wave velocity, v_s , for each position from CPT data with testing depth as 5.3.2

3. Assemble the shear wave velocities, v_s , of each soil layer
4. Make regression equation to estimate v_s for each soil layer with depth
5. Use the above equation with depth to estimate the average shear wave velocity, v_s , for each layer with average depth of each layer
6. Calculate v_p from v_s by equation: $\frac{v_p}{v_s} = \sqrt{\frac{2(1-\nu)}{1-2\nu}}$ with Poisson's ratio, ν , estimated according to instruction of Vietnamese standard "Poisson's ratio of soil in urban areas is from 0.1 to 0.5. Based on previous researches, the Poisson's ratio doesn't influence considerably the vibration of ground, therefore in the calculation it can be assumed as 0.4". This agrees with result of site survey reported by Trinh et al. (2003 and 2011).
7. Estimate the average damping ratio for each soil layer.

From the above procedures, calculation for 8 penetrated points from HX01 to HX08 was performed. After having the computed data of v_s of each soil layer it indicated that v_s vary linearly depended on the depth, Z . Therefore, the linear regression equation has been used to set up the relationship between v_s and depth, Z . The results of computing the shear wave velocities, v_s , at two typical boreholes HX02 and HX08 are shown in Figure 5.5 and 5.6, respectively. The results of estimating the equation of relationship between v_s and depth, Z , for each soil layer are shown in Table 5.6.

The results from Table 5.6 are used for computing the two typical positions of the metro line No.3. The position of Km 0+940 is between G1 station and G2 station, with minimum tunnel depth and the best geological condition. The position of Km 6+700 is between G7 station and ending point of the line, with maximum tunnel depth and the thickest weak soil layer. The results are shown in Table 5.7 and 5.8.

In order to verify the validity of the results of computing the shear wave velocity from CPT data, it needs to compare the geological conditions of urban delta area of Vietnam and research area of Andrus (2003) as following:

- In the above computing method, the CPT data from field test and the stratum age were used in the geological survey report of Hanoi city. The data is high reliable.
- The two compared areas are the delta with correlation about geographical location.
- The stratum age of Hanoi area is almost Holocene and Pleistocene (Till et al., 2005). On the other hand, beside Holocene and Pleistocene, the stratum of South Carolina includes the Tertiary. Thus, the two areas correlate in the stratum age.
- The shear wave velocity, v_s , propagating through saturated clay in Hanoi area is shown in the document issued in (Trinh et al., 2003) as 100m/s, which agree well with the results shown in Table 5.8 for layer 1 of saturated clay, $v_s=103.01\text{m/s}$. Thus, the results agree well with the measurement data.

From the aforementioned analysis, application of the suggested method for estimation of the dynamic parameters was found to be in conformity with actual condition of Vietnam at present, thus it showed the reliable results.

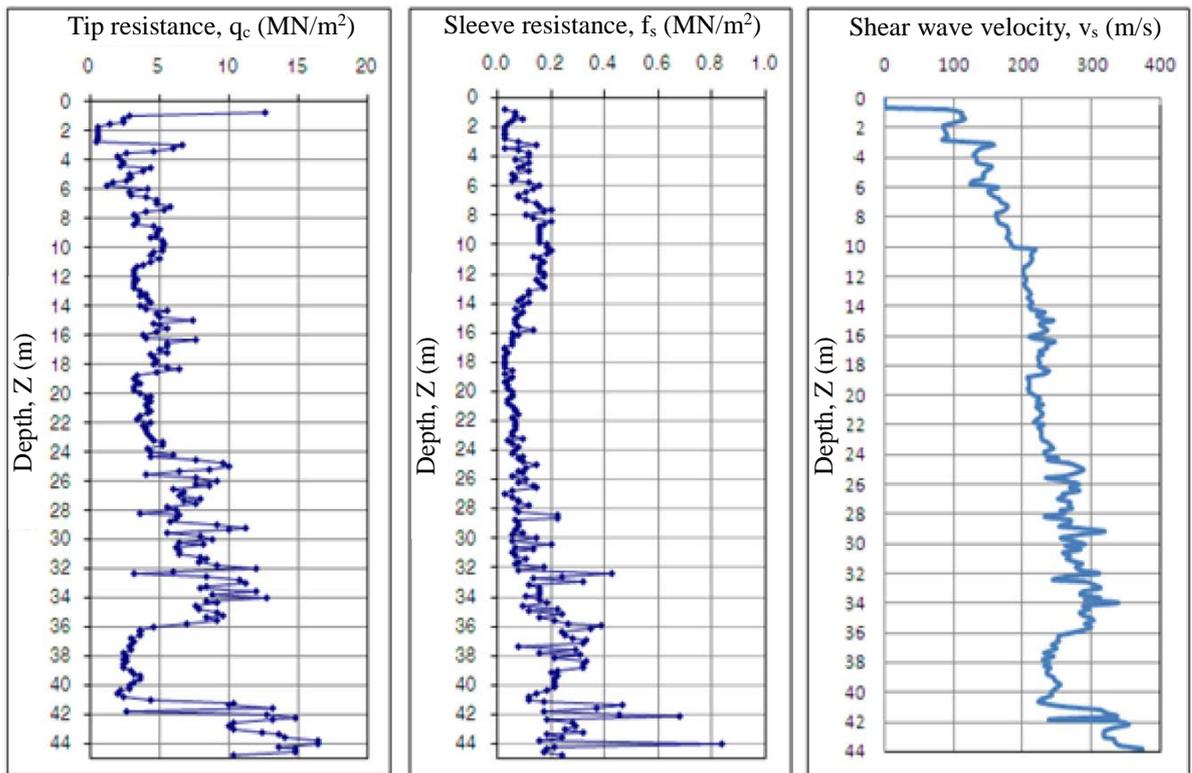


Figure 5.5 Computing result of shear wave velocity from CPT data at penetration point HX02 of metro line 03

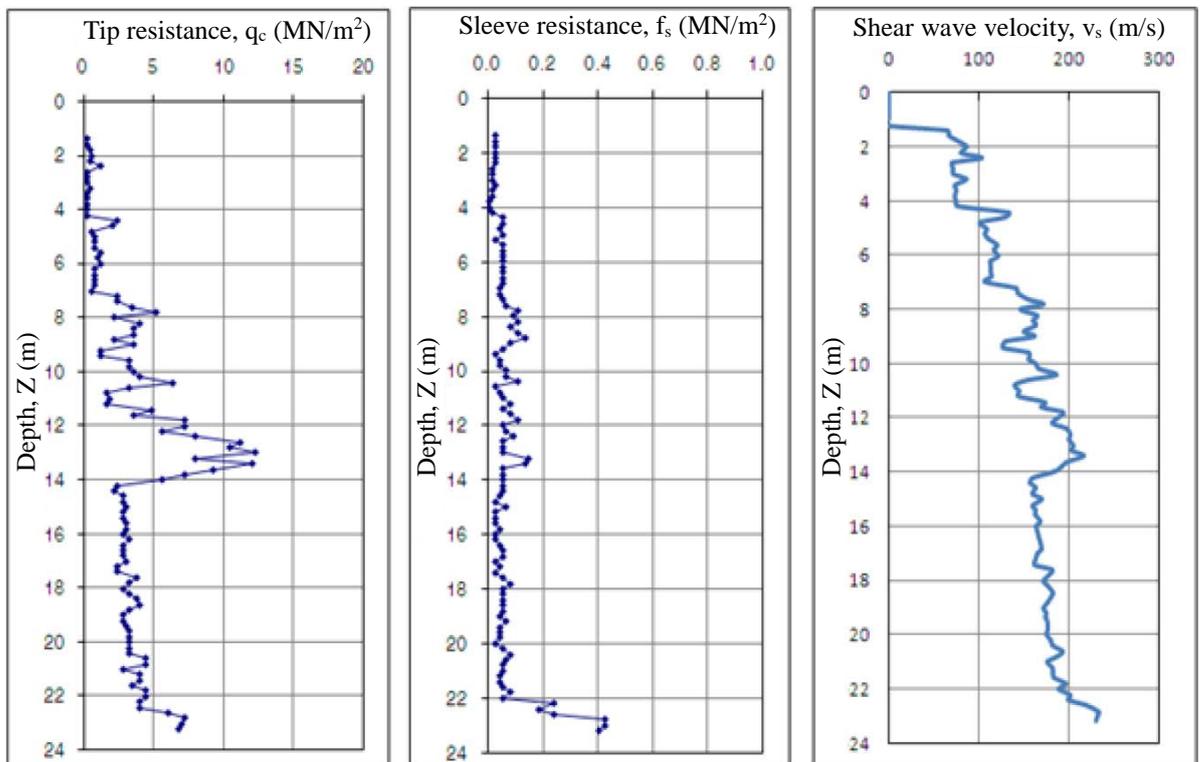


Figure 5.6 Computing result of shear wave velocity from CPT data at penetration point HX08 of metro line 03

Table 5.6 Estimation of shear wave velocity, v_s with depth of soil layers for metro line No.3 in Hanoi city

No	Layer	Soil layer Description	Regression equation, v_s (m/s)
1	F	Medium sand, gravel with clay, water content 22.4%	$v_s = \frac{20 + Z}{0.227}$
2	2	Silt clay, blackish grey, greyish blue, very soft to liquid,	$v_s = \frac{15.05 + Z}{0.249}$
3	1a	Clay, greyish blue, greyish brown, yellowish grey, moderately plastic, medium stiff	$v_s = \frac{4 + Z}{0.066}$
4	1b	Silty sand with little gravel, yellowish brown, greyish blue, saturation, water content 53%	$v_s = \frac{15 - Z}{0.173}$
5	2	Clay, greyish brown, greyish yellow with white, medium stiff	$v_s = \frac{25 + Z}{0.189}$
6	2a	Clayey sand, brownish grey, yellowish grey, fine to medium grained, medium dense, saturation	$v_s = \frac{9 + Z}{0.103}$
7	2b	Clay with gravel Laterite, yellowish grey, brownish grey, medium stiff	$v_s = \frac{10 + Z}{0.109}$
8	3	Silty sand, brownish grey, whitish grey, saturation, medium dense	$v_s = \frac{35 + Z}{0.238}$
9	3a	Silty sand, brownish grey, whitish grey, yellowish grey, saturation, dense	$v_s = \frac{28.38 + Z}{0.191}$
10	4	Clay, red-brown mottling, yellowish grey with whitish grey, stiff	$v_s = \frac{Z}{0.132}$

Table 5.7 Results of computing the shear wave velocity and damping ratio at Km0+940

Layer	Depth (m)	Calculation Result		
		v_s (m/s)	v_p (m/s)	h (%)
F	3.07	94.89	232.43	3.81
2b	7.29	139.27	341.14	5.91
2	17.27	197.25	483.16	7.70
3a	21.80	250.89	614.55	3.69
3	45.96	289.41	708.91	2.86
4	50.00	363.48	890.34	7.35

Table 5.8 Results of computing the shear wave velocity and damping ratio at Km6+700

Layer	Depth (m)	Calculation Result		
		v_s (m/s)	v_p (m/s)	h (%)
F	2.03	92.60	226.82	2.14
1a	4.79	106.42	260.67	3.10
1	16.41	103.01	252.32	7.83
1b	22.74	199.88	489.60	5.53
2	26.70	263.27	644.88	7.60
2a	31.31	369.03	903.84	6.24
3	48.48	315.46	772.72	2.35
4	50.00	374.39	917.06	7.08

5.5 SUMMARY

An analytical procedure for estimation of the soil dynamic parameters in conformity with current condition of Vietnam based on the methods of previous researches in the world has been suggested. The following conclusions were derived based on the calculated results.

The result from calculation of shear wave velocity based on CPT data for Metro line No.3 in Hanoi city agrees well with the measurement result reported in the literatures (Trinh et al., 2003 and 2011). Thus, the suggested methods were found to be reliable in conformity with current condition of Vietnam.

The estimation of the soil dynamic parameters based on CPT data with low cost is feasible at current situation in Vietnam.

The methods for estimation of the soil dynamic parameters based on SPT, CPT data bring a feasible research direction for evaluation and establishment of soil dynamic parameters in Vietnam in application for dynamic problems.

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