Experimental Study on Parameters of Hardening Soil Model with Small Strain Stiffness for Muddy Silty Clay and Silty Sandy Soil in Yangtze Floodplain Area

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Abstract. To obtain test parameters for the hardening soil model with small strain stiffness (HSS) of the muddy silty clay and silty sandy soil in the Nanjing Yangtze River floodplain, the following tests were conducted: consolidation test, consolidated-drained triaxial test, triaxial loading and unloading test, resonant column test and other common geotechnical tests. These test parameters included reference tangent modulus Eoedref, reference secant modulus E50ref, reference unloading and reloading modulus Eurref, failure ratio Rf, etc. A standardized process for determining HSS model parameters of certain types of soils, including relevant laboratory tests procedures, calculations, and analyses, has also been discussed. The test findings conclude with a summary of the model parameters for the typical strata in the Yangtze River floodplain region, providing engineering reference value for the deep foundation excavation and other relevant projects in the Yangtze River floodplain area.

Keywords: Hardening Soil Model With Small Stain Stiffness · Triaxial Test · Oedometer Test · Resonant Column Test

1 Introduction

The soil layers in the Yangtze River floodplain area mainly consist of muddy silty clay in the upper floodplain and silty sand in the lower floodplain, which are widely distributed. These soils exhibit typical engineering properties such as low foundation bearing capacity, easy deformation after disturbance, and time-consuming for stability. In order to better understand the response behavior of these soils in engineering activities, samples of the muddy silty clay and silty sand in this area are collected and tested in the following three aspects:

- Physical property tests: natural gravity, liquid limit, plastic limit, plastic index;
- Mechanical property tests: consolidation test, consolidated-drained triaxial test, triaxial loading and unloading test;
- Dynamic property test: resonant column test.

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2 Hardening Soil Model with Small Strain Stiffness (HSS)

The HSS model could reflect the nonlinear behavior of soil stiffness in the small strain range, which is one of the constitutive models of soft soil widely utilized in the numerical analysis of geotechnical engineering. The HSS model contains 11 standard Hardening Soil (HS) model parameters and 2 additional small strain parameters [1], which are specified as follows: E_{50}^{ref} is the reference secant modulus, which is the secant slope corresponding to 1/2 destructive strength under the reference confining pressure; E_{oed}^{ref} is the compression modulus of consolidation test under reference confining pressure; E_{ur}^{ref} is the unloading and reloading modulus; m is the power index for stress-level dependency of stiffness; *c'* is the effective cohesion of soil mass; *φ'* is the effective internal friction angle of soil mass; *Ψ* is the dilatancy angle of soil mass; R_f is the failure ratio; v_{ur} is the Poisson's ratio for unloading and reloading; G_0^{ref} is the initial shear modulus under the reference confining pressure; $\gamma_{0.7}$ is the shear strain threshold; P^{ref} is the reference stress; K_0 is the coefficient of lateral earth pressure at rest.

3 Test Content and Results

3.1 Soil Samples

The sampling site is located at No. 201, Yanjiang Road, Gulou District, Nanjing City, Jiangsu Province. The samples, which consist of muddy silty clay and silty sandy soil, were collected using thin-wall sampling method and preserved by wax-sealing, as shown in Fig. 1. The muddy silty clay on site has a thickness of 11.5 to 17.6 m. It is grayish black in color, flow plastic, sensitive to touch, with a smooth breaking section and relatively uniform particle size, and contains a large amount of organic matter and some shellfish fragments. The thickness of silty sandy soil on site ranges from 18 to 23.4 m. These two corresponding soil layers are typical strata in Nanjing Yangtze River floodplain area.

Fig. 1. Thin-wall samples from Nanjing Yangtze River floodplain area.

3.2 Physical Property Tests of Undisturbed Soil Samples

The basic physical properties of undisturbed soil include natural water content (w), density (ρ), specific gravity (G_S), Atterberg limits, etc. The test methods shall be in accordance with the Standard for Soil Test Methods (GB/T50123–2019) [2].

Water Content Test. The natural water contents of soil samples were measured using the drying method. 15–30 g representative samples were dried in oven for 48 h with the temperature controlled at 60–70 °C during the test. The samples were weighed before and after the drying process, respectively. The calculation formula of natural water content (w) is as follows:

$$w = (m/m_d - 1) \times 100 \tag{1}$$

where w is water content of the soil sample (%), m is the mass of wet soil (g), m_d is mass of dry soil (g).

Density Test. The densities of soil samples were obtained using the cutting ring method. The soil sample was trimmed into a soil column which is slightly larger than the diameter of the ring sampler using a wire saw. Then the ring sampler lubricated with Vaseline was pressed vertically to cut the soil until the soil sample extended out of the ring sampler. The remaining soils at both ends were cut off and leveled. The ring sampler were weighed before and after, respectively. The density calculation formula is as follows:

$$\rho = (m_1 - m_2)/V \tag{2}$$

where ρ is the density of the soil sample (g/cm^3), m_1 is the total mass of wet soil and ring sampler (g), m_2 is mass of ring sampler (g), V is the volume of ring sampler (cm^3), which is 60 cm^3.

Specific Gravity Test. The specific gravities of soil samples were measured using the pycnometer method. Dry the pycnometer before filling it with a 100 ml pycnometer with 15 g of dried soil. Half-fill the bottle with pure water, then shake the pycnometer. The following day, place the bottle on the sand bath and boil it for at least an hour. Fill the pycnometer with pure water, and wait until the suspension on the upper part of the bottle is clear and the temperature of the suspension in the bottle is stable. Plug the bottle stopper to make excess water overflow from the capillary tube of the bottle stopper. Weigh the combined mass of the bottle, water, and soil after drying the water outside the bottle. Immediately after weighing, take a temperature reading of the water in the bottle. Check the total mass of the bottle and the water based on the relationship that has been drawn between the temperature and the total mass. The calculation formula is as follows:

$$G_S = \left(\frac{m_d}{m_1 + m_d - m_2}\right) G_{wt} \tag{3}$$

where G_S is the specific gravity of the soil particles, m_d is the mass of the dry soil (g), m_1 is the total mass of the bottle and water (g), m_2 is the total mass of the bottle, water and soil (g), G_{wt} is specific gravity of pure water at t °C, accurate to 0.001.

This test requires two parallel measurements and the allowable parallel difference is 0.02. The arithmetic mean should be used.

Atterberg Limits Test. The Atterberg limits tests were conducted with the FG-III liquid plastic limit test equipment, as shown in Fig. 2. The cone sinking depths in the soil at different water contents were measured, and the relationship diagram between the water content and the cone penetration depth is drawn, as shown in Fig. 3. The water content in the diagram corresponding to the cone penetration depth of 2 mm is the plastic limit water content, and the water content in the diagram corresponding to the cone penetration depth of 17 mm is the liquid limit water content. According to the plastic limit and liquid limit obtained from the test, the plasticity index of soil is defined as:

$$I_P = w_L - w_P \tag{4}$$

where I_P is the plasticity index, w_L is the liquid limits (%), w_P is the plastic limits (%).

The liquidity index of soil is calculated according to the following formula:

$$I_L = \frac{w - w_P}{I_P} \tag{5}$$

where I_L is the liquidity index, w is natural moisture content of soil (%).

Liquid limit, plastic limit and plasticity index comprehensively reflect clay content in soil. The plasticity index is higher when more clay is present. The liquidity index reflects the natural state of the soil. When the liquid index is greater than 1, the soil is in flowing state.

Fig. 2. Atterberg limits test.

The physical properties test results of muddy silty clay and silty sandy soil samples are shown in Table 1.

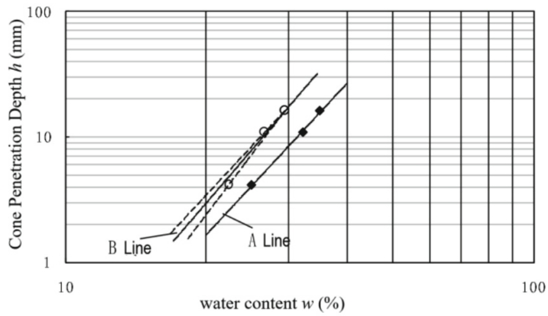


Fig. 3. Relationship between cone penetration depth and water content.

Table 1. Physical properties test results of undisturbed soil samples.

Soil sample	General parameters						Atterberg limits		
	Water content (%)	Density (g/cm^3)		Specific gravity	Void ratio	Plastic limit (%)	Liquid limit (%)	Liquidity index	Plasticity index
		Wet	Dry						
	w	ρ	ρ_d	G_S	e_0	w_p	w_L	I_L	I_P
Muddy silty clay	40.1	1.81	1.29	2.71	1.094	20.7	41.4	/	21
Silty sandy soil	31.2	1.81	1.38	2.69	0.952	/	/	/	/

3.3 Mechanical Property Tests of Undisturbed Soil Samples

Consolidation Test. The consolidation and compression characteristics of undisturbed soil mainly include the compressibility coefficient, compressibility modulus, consolidation coefficient and other characteristic parameters. The test method to determine these parameters should follow the steps specified in the Standard for Soil Test Methods (GB/T50123–2019) [2]. The consolidation pressure grades are 12.5, 25, 50, 100, 200, 400, 800 and 1600 kPa, respectively. The relationship between deformation and time under each consolidation pressure shall be measured, based on which the compression and consolidation characteristic parameters are determined.

The consolidation test was carried out on an oedometer with a sample area of 30 cm^2 and an initial height of 2 cm. The applied loads were 12.5, 25, 50, 100, 200, 400, 800, 1600 kPa, each of which lasted for 24 h. The compression deformation of the sample under each level of load was measured, the void ratio of the sample under each level of load was then calculated and the compression curve (e – p curve) was drawn in Fig. 4.

The initial void ratio of the sample is calculated as follows:

$$e_0 = \frac{\rho_w G_S (1 + 0.01 w_0)}{\rho_0} - 1 \tag{6}$$

where G_S is the specific gravity of soil particles, ρ_w is the density of water (g/cm^3), ρ_0 is initial density of the sample (g/cm^3), w_0 is the initial moisture content of the sample (%).

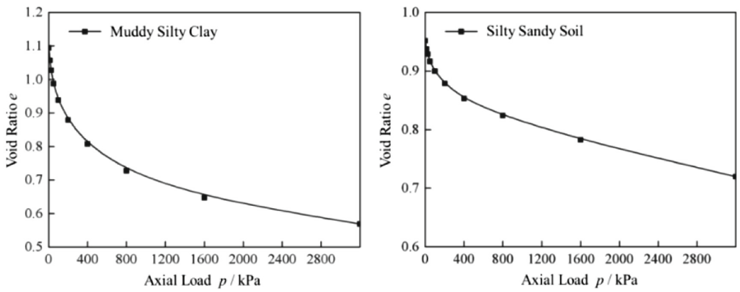
According to the compression curve ($e – p$ curve) obtained from the test, the compression coefficient and compression modulus were calculated. Generally, the compressibility of soil is evaluated by the compressibility coefficient within the pressure range from 100 to 200 kPa. The formula for calculating the compressibility coefficient and modulus is as follows:

$$a_v = \frac{e_i - e_{i+1}}{p_{i+1} - p_i} \tag{7}$$

where e_i is the void ratio under certain pressure p_i (kPa).

The compression modulus within the pressure range from 100 to 200 kPa is calculated using the following formula:

$$E_s = \frac{1 + e_0}{a_v} \tag{8}$$



(a) (b)

Fig. 4. Relationships between void ratio and load in consolidation test of: (a) Muddy silty clay; (b) Silty sandy soil.

The relationship between the axial load and the axial strain in the standard consolidation test of each sample is shown in Fig. 5. The axial deformation of each sample increases with the increment of the axial load applied. The trend of ε_d is consistent with the axial load p: the initial curve is relatively flat and the slope of the curve increases with the increment of the axial load applied. The muddy silty clay sample generated larger axial deformation than the silty sandy soil sample. The tangent slope of each curve at the load (p) of 100 kPa was calculated, which is the reference tangent modulus $E_{oed}{}^{ref}$ when the reference stress P^{ref} is 100 kPa. The consolidation test results are shown in Table 2.

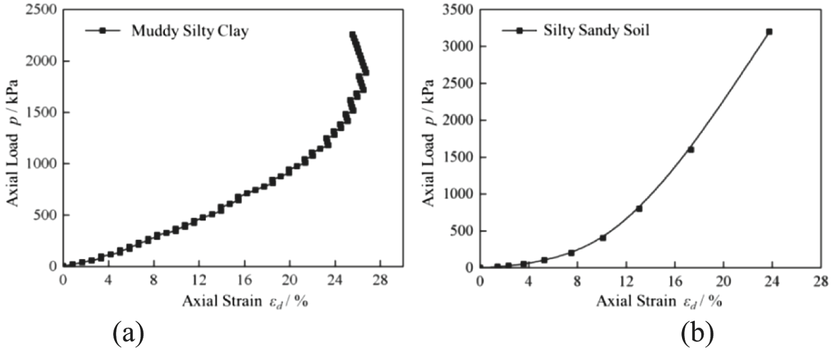


Fig. 5. Relationships between load and strain in consolidation test of: (a) Muddy silty clay; (b) Silty sandy soil.

Table 2. Consolidation test results of undisturbed soil samples.

Soil sample	Compressibility coefficient $a_{v1-2}\left(\mathrm{MPa}^{-1}\right)$	E_{oed}^{ref} (Mpa)	E_{s1-2} (Mpa)
Muddy silty clay	0.60	3.80	3.50
Silty sandy soil	0.22	8.93	9.09

Consolidated-Drained Triaxial Test. The reference secant modulus $E_{50}{}^{ref}$, failure ratio R_f and strength parameters of soil mass (c' and φ') were obtained from the consolidated-drained triaxial test, which is conducted with triaxial shear apparatus. The sample is 39.1 mm in diameter and 80 mm in height.

The consolidated-drained triaxial test is mainly carried out in four steps: (1) back pressure saturation: In this study, the back pressure saturation of soil samples was directly conducted under the condition that the confining pressure was always 5 kPa higher than the back pressure; (2) B value check: The back pressure remained unchanged while the confining pressure was increased by 30 kPa, and then the pore water pressure coefficient B was measured. If B value is greater than 0.95, the sample is considered to be saturated. Otherwise, the back pressure saturation should be continued until B value is greater than 0.95; (3) Consolidation: The samples were consolidated under effective isotropic confining pressure σ_3; (4) Shear: The shear rate was set to be 0.008 mm/min. The test was terminated when the strain value of the sample reached 20%. During the test, the drainage volume during consolidation, axial load, shear displacement, volume change and other parameters were measured. Relevant calculations were conducted per specification requirements.

As shown in Fig. 6, the consolidated-drained triaxial tests were conducted with confining pressure of 100 kPa to obtain the relationship between the deviatoric stress q and axial strain ε_d of the samples. At the beginning of the test, the deviatoric stress of each sample increased with the increment of the axial strain. When the axial strain reached a certain range, the deviatoric stress tended to be unchanged or slightly decreasing. The deviatoric stress value corresponding to the axial strain of 15% is taken as the failure value q_f and the deviatoric stress value corresponding to the stable section or peak of the curve is taken as the asymptotic value q_a. Then the failure ratio of each sample R_f can be calculated $\left(R_f = q_f/q_a\right)$. The slope of the line connecting the origin and the point corresponding to $0.5q_f$ is the reference secant modulus E_{50}^{ref} of the sample.

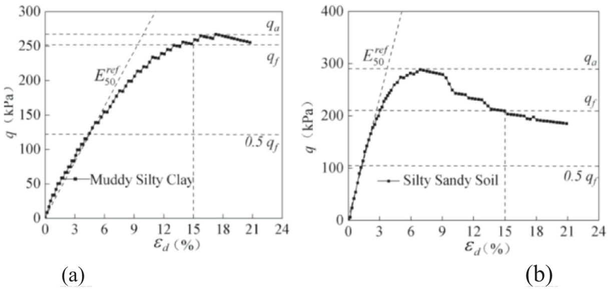


Fig. 6. Stress strain relationship curves from consolidated-drained triaxial tests for: (a) Muddy silty clay; (b) Silty sandy soil.

According to the obtained stress strain relationships under four different confining pressures of 100, 200, 300 and 400 kPa, four Mohr circles and corresponding failure envelope were drawn in Figs. 7 and 8, for muddy silty clay and silty sandy soil, respectively. The intercept and slope of each failure envelope are taken as the effective cohesion c' and effective friction angle φ'. The consolidated-drained triaxial test results are shown in Table 3.

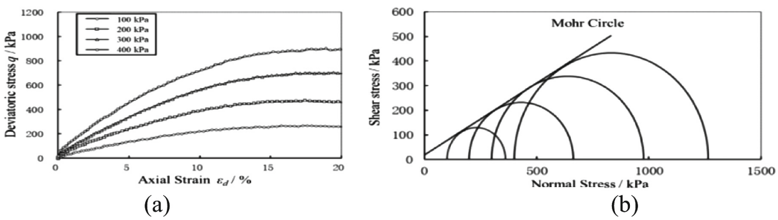
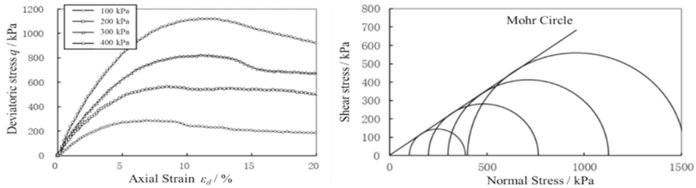


Fig. 7. (a) Stress strain relationship curves and (b) Mohr circles for muddy silty clay.

Triaxial Loading and Unloading Test. The consolidated-drained triaxial loading and unloading test is mainly carried out in four steps, the first three of which are the same as the consolidated-drained triaxial test mentioned in the previous chapter. The fourth step



(a) (b)

Fig. 8. (a) Stress strain relationship curves and (b) Mohr circles for silty sandy soil.

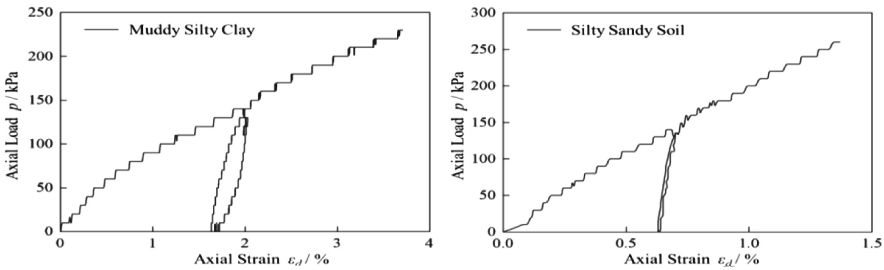
Table 3. Consolidated-drained triaxial test results of undisturbed soil samples.

Soil sample	E_{50}^{ref} (MPa)	R_f	Effective cohesion c' (kPa)	Effective friction angle φ' (degree)
Muddy silty clay	3.53	0.96	18.3	30.2
Silty sandy soil	6.9	0.73	1.6	35.4

is to apply axial loading, unloading and reloading to the sample. The load applied for the first time is about 40% of the expected failure deviatoric stress of the sample. When the load is added to the target value, the axial unloading shall be carried out immediately until the load is zero. Then the axial reloading shall be conducted until the load applied is about 60% of the expected failure deviatoric stress of the sample. The axial loading, unloading and reloading processes are all stress-controlled.

The consolidated-drained triaxial loading and unloading test can determine the reference unloading and reloading modulus E_{ur}^{ref} of soil. Figure 9 shows the stress strain relationship curves during consolidated-drained triaxial loading and unloading tests under the confining pressure of 100 kPa for muddy silty clay and silty sandy soil, respectively.

The deviatoric stress increased with the increment of axial strain at the initial stage of loading, and the curve is relatively flat; During the initial unloading stage, the axial strain slightly increased before decreasing, showing unloading rebound; During reloading process, the initial stress-strain curve is very steep, indicating that the axial strain changed little with the increment of deviatoric stress until the deviatoric stress reached the stress level before unloading. Then the stress-strain curve became relatively flat again and increased along the trend of the initial loading curve. In the process of unloading and reloading, the stress-strain curve showed a hysteresis loop. The average slope of the unloading and reloading curves, which connects the two endpoints of the hysteresis loop, serves as the reference unloading and reloading modulus E_{ur}^{ref} of the soil mass, as shown in Table 4.



(a) (b)

Fig. 9. Stress strain relationship curves during triaxial loading and unloading test for: (a) Muddy silty clay; (b) Silty sandy soil.

Table 4. Triaxial loading and unloading test results of undisturbed soil samples.

Soil sample	E_{ur}^{ref} (MPa)
Muddy silty clay	23.5
Silty sandy soil	44.8

3.4 Dynamic Property Tests of Undisturbed Soil Samples

Resonant Column Test and Data Analysis. GZZ-50 resonant column test equipment, which is shown in Fig. 10, has incorporated necessary data treatment in the instrument program based on the basic principle of resonant column test, and can output the test results directly. The relationships between confining pressure and maximum dynamic shear modulus, shear strain and shear modulus, shear strain and material damping ratio of remolded soil samples are obtained using resonant column tests.

(a) (b)

Fig. 10. Resonant column test instrument.

It should be noted that the analysis results using different fitting methods for the dynamic stress-strain relationships of soil are not exactly the same. In this study, values of shear modulus and material damping ratio are obtained using resonant column test within shear strain (γ) less than 10^{-4}, and fitted using Wang and Stokoe (2022) model [3] for shear strain (γ) greater than 10^{-4}.

Based on Menq (2003) [4], the relationship between the maximum dynamic shear modulus and the effective average consolidation stress can be described using following empirical formula:

$$G_{\max} = C \cdot P_a \cdot \left(\frac{\sigma_m}{P_a}\right)^n \tag{9}$$

where G_{max} is the maximum dynamic shear modulus, C is $G_{\max}/P_a$ value when $\sigma_m/P_a = 1$, n is the slope of the fitting line in log scale, P_a is the atmospheric pressure.

The vertical effective self-weight stress of muddy silty clay and silty sandy soil at the depth of sampling is 300 kPa and 400 kPa, respectively. According to Wang and Stokoe (2022) [3], the relationships between shear strain and shear modulus, shear strain and material damping ratio of uncemented soils can be described using Eqs. 10 and 11, respectively.

$$\frac{G}{G_{\max}} = \frac{1}{\left(1 + (\gamma/\gamma_{mr})^a\right)^b} \tag{10}$$

where both "a" and "b" are the curvature parameters which control the shape of the shear modulus reduction curve, and γ_{mr} is the modified reference strain at which $G/G_{\max} = 0.5^b$.

$$D = \frac{d \cdot (\gamma/\gamma_D)^c + D_{\min}}{(\gamma/\gamma_D)^c + 1} \tag{11}$$

where "d" is the practical upper boundary limit of the material damping ratio, "c" is the curvature parameter which controls the shape of the material damping increment curve, $D_{\min}$ is the material damping ratio of soil in the small-strain range and γ_D is the reference shear strain at which $D = (d + D_{\min})/2$.

According to the PLAXIS manual [1], the initial shear modulus G_0^{ref} of each layer under the reference confining pressure can be obtained using following formula:

$$G_0 = G_0^{ref}\left(\frac{c' \cot\varphi' - \sigma_3}{c' \cot\varphi' + P^{ref}}\right)^m \tag{12}$$

Dynamic Properties of Muddy Silty Clay and Silty Sandy Soil. Based on the data obtained from resonant column tests, the relationships between confining pressure and maximum dynamic shear modulus for muddy silty clay and silty sandy soil are shown in Table 5, and plotted in Fig. 11, respectively. The relationships between shear strain and shear modulus for muddy silty clay and silty sandy soil are plotted in Fig. 12, and the normalized shear modulus reduction curves are shown in Fig. 13. The relationships between shear strain and material damping ratio for muddy silty clay and silty sandy soil are plotted in Fig. 14. The small-strain stiffness input parameters are calculated and shown in Tables 5 and 6.

Table 5. Relationships between confining pressure and maximum dynamic shear modulus for undisturbed soil samples

Soil sample	Consolidation ratio K_c	Confining pressure (kPa)	G_{max} (MPa)	n	C
Muddy silty clay	1.0	100	36.01	0.739	354.9
		200	57.85		
		300	78.95		
		400	101.08		
Silty sandy soil	1.0	100	79.38	0.573	788.4
		200	115.72		
		300	147.78		
		400	175.64		

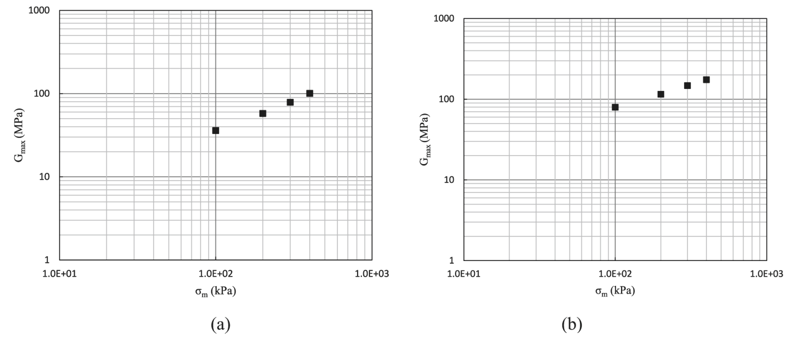


Fig. 11. Relationships between confining pressure and maximum dynamic shear modulus for: (a) Muddy silty clay; (b) Silty sandy soil.

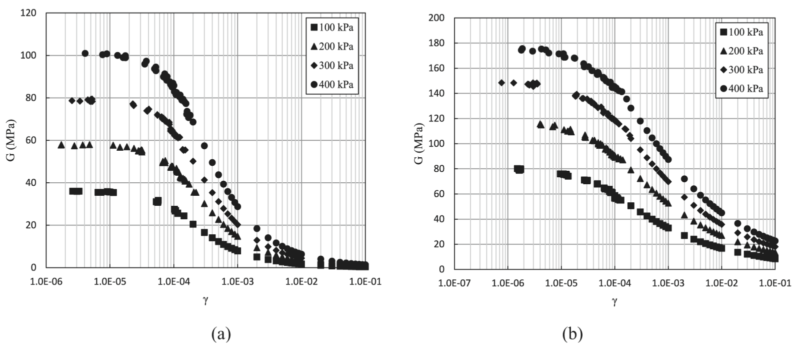
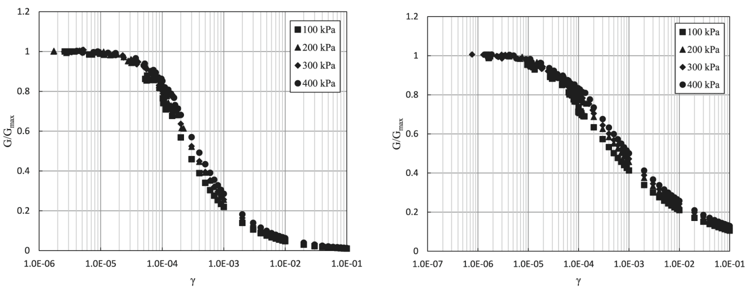
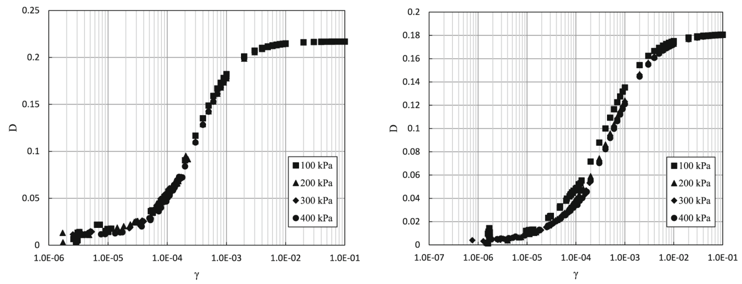


Fig. 12. Relationships between shear strain and shear modulus for: (a) Muddy silty clay; (b) Silty sandy soil.



(a) (b)

Fig. 13. Relationships between shear strain and normalized shear modulus for: (a) Muddy silty clay; (b) Silty sandy soil.



(a) (b)

Fig. 14. Relationships between shear strain and material damping ratio for: (a) Muddy silty clay; (b) Silty sandy soil

Table 6. Small-strain stiffness additional input parameters for undisturbed soil samples.

Soil layer	Confining pressure σ_m (kPa)	G_0 (MPa)	G_0^{ref} (MPa) ($\sigma_m = 100\,\text{kpa}$)	$\gamma_{0.7}$ (10^{-4}) ($\sigma_m = 100\,\text{kpa}$)
Muddy silty clay	300	37.04	47.44	2.9
Silty sandy soil	400	76.92	90.45	3.5

4 Summary

In this study, based on the data obtained from physical property tests, mechanical property tests, and dynamic property test, all input parameters for the hardening soil model with small strain stiffness (HSS) of the muddy silty clay and silty sandy soil in the Nanjing Yangtze River floodplain area have been determined. The summary of all input parameters for HSS model is shown in the Table 7. A standardized process for determining HSS model parameters of certain types of soils, including relevant laboratory tests procedures, calculations, and analyses, has also been discussed. These research results provide a basis for the numerical simulation of engineering practice, such as deep foundation excavation modeling, site response analysis, and other relevant engineering calculation in the Yangtze River floodplain area.

Table 7. Summary of input parameters for HSS model.

Parameter	Physical Significance	Unit	Muddy Silty Clay	Silty Sandy Soil	Methods of Acquiring
${E_{50}}^{ref}$	Reference secant modulus	kPa	3.53	6.9	Consolidated-drained triaxial test
${E_{oed}}^{ref}$	Compression modulus	kPa	3.80	8.93	Consolidation test
${E_{ur}}^{ref}$	Unloading and reloading modulus	kPa	23.5	44.8	Triaxial loading and unloading test
M	Power for stress-level dependency	/	0.61	0.56	Consolidated-drained triaxial test
c'	Effective cohesion of soil mass	kPa	18.3	1.6	Consolidated-drained triaxial test
φ'	Effective internal friction angle of soil mass	degree	30.2	35.4	Consolidated-drained triaxial test
Ψ	Dilatancy angle of soil mass	degree	0	5.4	Consolidated-drained triaxial test
R_f	Failure ratio	/	0.96	0.73	Consolidated-drained triaxial test
v_{ur}	Poisson's ratio for unloading and reloading	/	0.33	0.30	Consolidated-drained triaxial test
G_0^{ref}	Initial reference shear modulus	MPa	47.44	90.45	Resonant column test
$\gamma_{0.7}$	Shear strain threshold	10^{-4}	2.9	3.5	Resonant column test
P^{ref}	Reference stress	kPa	100	100	/
K_0	Coefficient of lateral earth pressure at rest	/	0.5	0.42	$K_0^{nc} = 1 - sin\,\varphi'$

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