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A Study on Parameters of HSS Constitutive Model of Soil from Ganzhou

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ABSTRACT: HSS is a kind of hardened soil model with small strain, in which 12 material parameters are needed in its constitutive model. Based on Ganzhou Rongjiang project, the parameters of HSS constitutive model of Ganzhou typical soil are determined by the geotechnical tests. Through ABAQUS, the constitutive relations of HSS model, MC model and MCC model are compared with the test results, which show that the triaxial stress-strain curve of HSS model is consistent with the triaxial stress-strain curve obtained from the test. The research results lay a technical foundation for the numerical simulation of Ganzhou soil.

Keywords: HSS model; Hardening with small strain; Mechanical parameters; Experimental analysis; Numerical simulation

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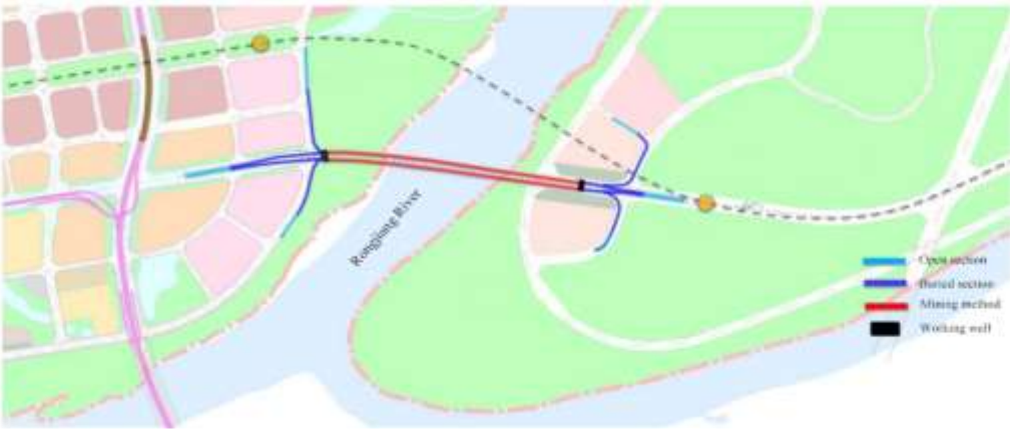
I. INTRODUCTION

There are 12 material parameters needed in the HSS constitutive model, some of which, such as cohesion, internal friction angle and so on, are the same as the existing molar-cullen constitutive model. However, most of the parameters such as the reference secant stiffness of 50% strength and the reference stiffness of unloading and reloading are not involved in other constitutive models. Therefore, it is necessary to obtain the 12 parameters of HSS constitutive model through relevant tests. The study of HSS constitutive parameters in Ganzhou has not been carried out at home and abroad. Therefore, it is the first attempt to determine the HSS constitutive parameters of Ganzhou through experiments.

For determining the 12 parameter values of HSS constitutive model, the following three types of tests are required: ①Basic geotechnical tests determine its density, pore ratio and other foundation parameters; ②Confined compression test determines its the reference lateral stiffness, stiffness stress level related power exponent and isotropic preconsolidation stress; ③Consolidated drained triaxial compression test determines reference secant stiffness, unloading / reloading reference stiffness, stiffness reference stress, unloading and reloading Poisson's ratio, cohesion, internal friction angle, expansion angle and failure ratio at 50% strength.

II. INTRODUCTION OF SOIL SAMPLES

The soil samples in this test are all taken from the deep foundation pit project of Rongjiang River under construction in Ganzhou. The specific location is shown in Figure 1.



1Fig.1 Location of Rongjiang Project

The sampling soil layer is silty clay layer, with a buried depth of $5.20m$. In the natural state, the

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water content is not high, and the soil is in a plastic state, with medium dry strength and medium toughness. The preliminary survey report shows that the natural density of soil is $1.91\times10^{3}kg/m^{3}$, the the natural water content is 26.50%, and the void ratio is 0.78. The sampling site is shown in Figure 2.

The on-site sampling includes undisturbed soil sample and disturbed soil sample. The sampling method of undisturbed soil sample is ring knife sampling. The specific operation is as follows: apply Vaseline to the inner wall of the ring knife to facilitate later demolding and soil sampling, clean and level the surface of the soil layer to be sampled, expose the soil layer to be sampled, and press the ring knife into the soil layer with the handle, and cut the surrounding soil while pressing downward until the soil in the ring knife is 3cm higher than the upper surface of the ring knife. Then use the soil trimming knife to level the upper and lower sides of the ring knife, and cover the ring knife cover. Two undisturbed soil samples are taken, as shown in Figure 3.



2Fig .2 Soil sampling site photo Fig .3 Two undisturbed soil samples3

For disturbed soil, 10kg of soil was taken from the site with a shovel, and then air dried, crushed, screened in the laboratory, prepared with water content to natural water content, and finally compacted to the corresponding density in a compactor. A total of 2 specimens for confined compression test and 2 specimens for triaxial compression test were prepared.

III. BASIC GEOTECHNICAL TESTS

In the basic geotechnical test, the natural density of soil is measured by ring knife method. The calculation of void ratio of soil is to measure soil moisture by drying method, and the relative density of soil is determined by specific gravity bottle method. Finally, the void ratio of soil is calculated according to the correlation.

The required test equipment includes: ring knife (the inner diameter is $70mm$, the height is 52mm, and the volume is $200cm^{3}$), electronic balance (the measuring range is 600g, and minimum graduation value is 0.1g), soil cutter, glass sheet, aluminum box, pycnometer, electronic thermometer, alcohol lamp, constant temperature oven (105-110 ℃), etc.

3.1 Test for natural density of soil

The specific operations of the ring knife method for measuring the natural density of soil are as follows:

① Use the electronic balance to weigh the mass m_1 of the empty ring cutter, which should be accurate to 0.1g

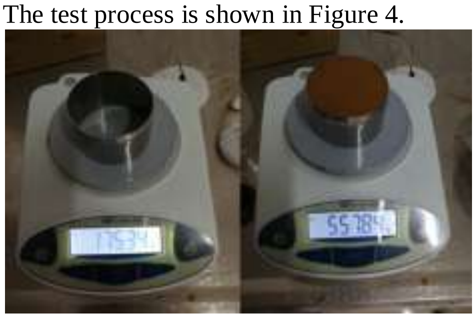
② Weigh the undisturbed soil sample and the ring knife together after erasing the soil on the outer wall of the ring knife, and get the mass m_2 of the ring knife plus soil, accurate to 0.1g

③Record the volumeV of the ring cutter

Then the natural density of soil is calculated according to formula 1.

$$\rho=\frac{m_2 - m_1}{V} \quad (1)$$

The test process is shown in Figure 4.


4Fig.4 Natural density weighing test

After two parallel measurements, the results are shown in Table 1. Taking the average of the two test results, the natural density of the soil sample is

International Journal of Advances in Engineering and Management (IJAEM)
Volume 3, Issue 6 June 2021, pp: 784-793 www.ijaem.net ISSN: 2395-5252

$1.911 g/cm^3$, the same as $1.911 \times 10^3 kg/m^3$.

1Table 1 Soil sample density test data

Serial number	Quality of Ring knife (g)	Volume of Ring knife (cm^3)	Mass of Clamp and soil (g)	knife Quality of Soil (g)	Density of Soil samples (g/cm^3)
1	175.3	200.0	557.8	382.5	1.913
2	175.0	200.0	556.9	381.9	1.909

3.2 Test for Soil Void Ratio

The specific operation of measuring water content of soil by drying method, measuring soil relative density by Pycnometer method, and then converting them to get soil void ratio is as follows:

(1) Use an electronic balance to weigh the mass, m_3, of the empty aluminum box to the accuracy of 0.01g;
(2) Take 10-20g soil sample from the ring knife, put it into the aluminum box, and weigh the total mass, m_4, of the aluminum box and soil block to the accuracy of 0.01g;
(3) Put the aluminum box into the oven, dry it for 10h, and then take it out after sealing and cool it to room temperature;
(4) Weigh the total mass, m_5, of the aluminum box and soil block to the accuracy of 0.01g;
(5) Take about 100g of the air dried soil block and grind it, and screen the air dried soil below 5mm;
(6) The sieved soil and the pycnometer were dried in an oven for 3 h, and then cooled to room temperature;
(7) Weigh the mass, m_6, of dry pycnometer to the accuracy of 0.01g;
(8)Weigh about 10g of dried soil, pour it into the pycnometer slowly with filter paper, and weigh the mass, m_7, of soil and pycnometer to the accuracy of 0.01g;
(9) Add half a bottle of pure water into the pycnometer, boil it for 1 hour and cool it, add the water to fill the pycnometer, plug the bottle tightly and wipe the surface water;
(10) Weigh the total mass, m_8, of pycnometer and water and soil mass to the accuracy of 0.01g, and measure the water temperature in the bottle to the accuracy of 0.5 ℃;
(11) After cleaning the pycnometer, fill it with boiled cool water, plug it tightly and wipe the surface water, and weigh the total mass, m_9, of pycnometer and water to the accuracy of 0.01g;
(12) Query and record the water density ρ_t corresponding to the temperature.

The water content, ω, of soil is calculated according to formula 2.

$$\omega = \frac{m_5 - m_4}{m_4 - m_3} \times 100\% \quad (2)$$

The relative density, G_S ,of soil is calculated according to formula 3

$$G_s = \frac{m_7 - m_6}{m_9 + m_7 - m_6 - m_8} \bullet \rho_t \quad (3)$$

The void ratio, e ,of soil is calculated according to formula 4

$$e = \frac{G_s \rho_w (1+\omega)}{\rho} - 1 \quad (4)$$

The test process is shown in figures 5 and 6.

5Fig .5 Water content test process

International Journal of Advances in Engineering and Management (IJAEM)
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6Fig.6 Relative density test process

Parallel tests were carried out for twice. The test results of soil moisture content are shown in Table 2, and the test results of soil relative density are shown in Table 3.

2Table 2 Test results fot water content of soil

Serial number	Quality of empty aluminium boxes (g)	Mass of aluminium box and soil (g)	Mass of aluminum box and soil after drying (g)	Water content of soil
1	7.75	33.87	28.07	0.2854
2	7.74	31.59	26.32	0.2836

3Table 3 Test results for relative density of soil

Serial number	Quality of pycnometer (g)	Quality of pycnometer and soil and water (g)	Water quality and soil mass (g)	Quality of water and pycnometer (g)	Density of water $(g/cm)^3$	Relative density of soil
1	31.33	42.91	90.41	83.17	0.999	2.67
2	31.34	41.64	89.49	83.10	0.999	2.63

If the average of the two test results is taken as the soil water content, the soil water content, ω ,is 28.45%, and the relative density, G_s, of soil is 2.649. The soil void ratio e = 0.779 is calculated by substituting formula (4), which is basically consistent with the data in the previous survey report.

3.3 Confined compression test

Confined compression test is that the standard specimen prepared by ring knife is put into the consolidation instrument, whose compression deformation by different loads under the condition of confined energy drainage. As a result, the relationship between vertical strain and vertical stress can be obtained to determine the reference lateral stiffness E_{oed}^{ref} , and m , the correlation power exponent of stiffness stress level. In addition, the $e-\log P$ curve can be used to determine the isotropic preconsolidation stress p_c.

The equipment required for the confined compression test include: lever type consolidometer (with weights, retaining rings, permeable stones and other accessories), ring knife, dial indicator (measuring range of 10 mm, minimum division value of 0.01 mm), timer, etc.

The main operations are as follows:

(1) Cut the soil samples for the test with ring knife, completing the preparation of the soil samples. After placing the steel retaining ring, the permeable stone and the filter paper in turn in the consolidometer, put the soil samples and the ring knife into the retaining ring to ensure that the installation is tight and free of voids, and the soil sample is covered with filter paper, permeable stone and transmission gland;

(2) Place the whole consolidation container in the center of the loading lever beam, and place the loading lever in the middle groove of the gland. Then install the loading equipment, apply 1kPa pre-pressure first to reduce the gap between the installed parts, and adjust the dial indicator reading to zero.

(3) The pressure of each level is applied according to the loading level of 12.5kPa, 25kPa, 50kPa, 100kPa, 200kPa and 400kPa. The vertical displacement value is measured and recorded 24h after the pressure of each level is applied

Then the curve relationship between vertical strain ε_{yy} and vertical stress σ_{yy} can be obtained, in which the vertical strain is calculated according to formula 5.

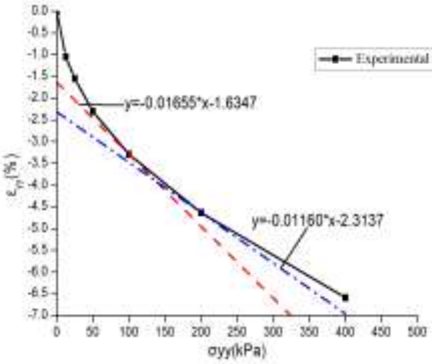
$$\varepsilon_{yy} = \frac{\Delta h}{h_0} \times 100\% \quad (5)$$

The total settlement value is Δh. The initial soil height is h_0. The vertical stress σ_{yy} is the load pressure value of each stage.

The samples and test process are shown in figure 7.

The curve relationship between the final vertical strain ε_{yy} and the vertical stress σ_{yy} is shown in figure 8.

7Fig.7 Samples and test process diagram of confinesd compression test



8Fig .8 Curve of vertical strain and vertical stress in confined compression

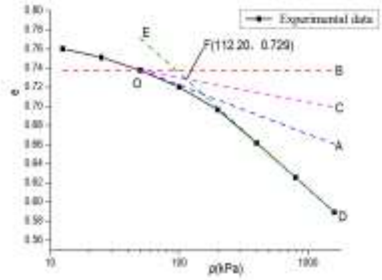
The reference lateral stiffness E_{oed}^{ref}, the same as the tangent stiffness when the vertical stress σ_{yy} is equal to the reference stress p^{ref}, which is the negative inversion of the tangent slope in the diagram. The reference stress p^{ref} is 100 kPa, so that the reference lateral stiffness E_{oed}^{ref} is 6042.197 kPa, The power exponent m can be determined by formula 6. In this test, $\sigma_{yy} = 200kPa$, $E_{oed}^{200} = 8617.690kPa$ .It's calculated that m equals to 0.512.

$$m = \log_{\frac{\sigma_{yy}}{p^{ref}}} \left(\frac{E_{oed}^{\sigma_{yy}}}{E_{oed}^{ref}}\right) \quad (6)$$

In addition to the curves of vertical strain ε_{yy} and vertical stress σ_{yy}, it is necessary to make e-log p curves to determine the Isotropic preconsolidation stress p_c, in which e can be calculated according to formula 7.

$$e = e_0 - \varepsilon_{yy}(1 + e_0) \quad (7)$$

A final e-log p curve is shown in Figure 9.



9Fig.9 $e - \log P$ Curve

International Journal of Advances in Engineering and Management (IJAEM)
Volume 3, Issue 6 June 2021, pp: 784-793 www.ijaem.net ISSN: 2395-5252

The pre-consolidation stress is determined by the following methods: find out the minimum curvature radius point O in the curve of $e - \log P$, make the tangent OA, horizontal line OB and bisector OC of angle AOB through O, extend the straight line at the end of the curve to be DE, DE and OC intersect at point F, and the pressure value p corresponding to point F is the pre-consolidation stress. In this test, the pre-consolidation stress is 112.20kpa, the buried depth of soil sample is 5.20m, and the natural density is $1.911 \times 10^3 kg/m^3$. The soil is over consolidated, and the over consolidation ratio OCR is 1.129.

3.4 Consolidated drained triaxial compression test

The consolidated drained triaxial compression test is to add confining pressure for consolidation under the condition of allowable drainage, and then load compression until failure of soil occurs. In this test, there is a process of unloading and reloading, and the complete stress-strain curve of loading, unloading and reloading soil can be obtained. The relationship between axial strain and volume strain can be recorded by switching on the volume strain measurement, and the stress circle and Mohr envelope diagram under different confining pressures can also be drawn[50].

The equipment required for consolidation drainage triaxial compression test includes: strain controlled triaxial apparatus (with volume change measurement system), sampler, saturator, rubber film, permeable plate, etc.

The main operations are as follows:

①Sample preparation: in this test, two samples were prepared, corresponding to the triaxial compression test when the confining pressure was 100kPa and 200kPa respectively. The standard triaxial sample (with diameter of 39.1mm and sample height of 80mm) was used for the soil sample, which was compacted in the compactor in five layers. The dry density and water content of the sample after control were kept the same as the undisturbed soil.

②The sample was completely soaked for 3 h by air extraction saturation method;

③Place the wet filter paper, the sample, the wet filter paper and the permeable plate on the base of the pressure chamber in turn, install the rubber membrane, and open the exhaust valve;

④Install pressure chamber cover, fill water and adjust dynamometer;

⑤Adjust the drain pipe and pore water pressure valve to the required confining pressure, and open the drain valve for consolidation for 1h;

⑥Adjust the contact between piston and sample, open the data acquisition interface of triaxial test, set the shear strain rate to $0.1\%/\min$ the same as $0.08mm/\min$, and return the index of dynamometer and displacement meter to zero;

⑦Turn on the motor to start shearing, load until the reading of dynamometer is about 300N, shut down the motor;

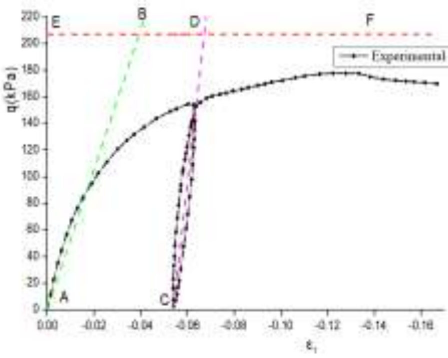
⑧Adjust the shear strain rate to $-0.1\%/\min$ which equals that the triaxial tension is $0.08mm/\min$. Start the motor to unload, load until the reading of dynamometer is about $10N$, and shut down the motor;

⑨Adjust the shear strain rate to $0.1\%/\min$, which equals that it compresses at a speed of $0.08mm/\min$. Start the motor to start reloading, when the peak load occurs, conduct 15-25% vertical strain again, shut down the motor, and the test is over.

The samples and test process are shown in figure 10.

10Fig .10 Samples and experimental process of triaxial compression test

The stress-strain curve obtained from the test is plotted. When the confining pressure is 100kPa, the complete stress-strain curve of soil loading, unloading and reloading is shown in Figure 11.



11Figure 11 Stress-strain curves of soil

In HSS constitutive model, the reference secant stiffness at 50% strength is the slope of secant AB, and the measured reference secant stiffness E_{50}^{ref} is 5274.934kpa. The unloading / reloading reference

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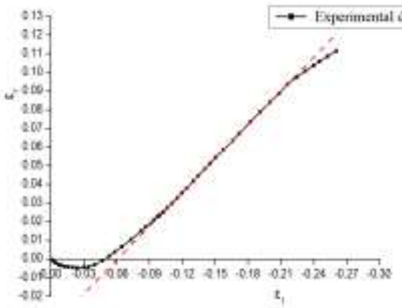
stiffness is the slope of line CD, and the measured unloading / reloading reference stiffness E_{ur}^{ref} is 17683.134kpa. The stiffness reference stress p^{ref} is set as 100kPa, and the progressive value of shear strength q_a is the corresponding stress value of line EF, which can be measured as 207.147kpa.

When the confining pressure is 100kPa, the volume strain and axial strain curve is shown in Figure 12.



12Fig.12 Volume strain and axial strain curve of soil

The unloading-reloading Poisson's ratio ν_{ur} is calculated by formula 4-8[9]。

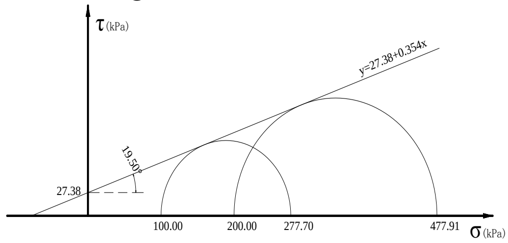
$$\upsilon_{ur} = \frac{1}{2} + \frac{\varepsilon_v}{2\varepsilon_1} \quad (8)$$

The shear expansion angle ψ is calculated by formula 9.

$$\psi = \arcsin \frac{\varepsilon_v - (1-2\upsilon)\varepsilon_1^e}{\varepsilon_v - 2\varepsilon_1 + (1+2\upsilon)\varepsilon_1^e} \quad (9)$$

υ nearly equals to ν_{ur} . ε_1^e is the vertical elastic strain value, which can be calculated by q/E_{ur}. It can be calculated that unloading-reloading Poisson's ratio $\nu_{ur} = 0.198$, and shear expansion angle $\psi = 2.56°$.

The stress circle and molar envelope diagram are shown in figure 13.



13Fig .13 Molar stress circle

The cohesion c of soil is 27.38 kPa, the internal friction angle is 19.50°.

The ultimate deviatoric stress q_f can be calculated to be 177.70 kPa according to formula 4, and the failure ratio R_f can be calculated to be 0.858 according to formula 4.

Above all, the calculation parameters of HSS constitutive model of Ganzhou silty clay are shown in Table 4.

4Form 4 Constitutive parameters HSS Ganzhou silty clay

Parameters	Value	Parameters	Value
ρ	1.911×10^3kg/m^3	c	27380Pa
E_{50}^{ref}	5274934Pa	φ	19.50°
E_{ur}^{ref}	17683134Pa	ψ	2.56°
E_{oed}^{ref}	6042197Pa	m	0.512
p^{ref}	100000Pa	e	0.779
p_c	112200Pa	R_f	0.858
ν_{ur}	0.198		

IV. SIMULATION VERIFICATION OF HSS CONSTITUTIVE MODEL BY TRIAXIAL COMPRESSION TEST

In ABAQUS, by writing subroutines, the HSS constitutive model is used to simulate the triaxial loading, unloading and reloading test of soil, and the simulation results are compared with the original Moore Coulomb model and modified Cambridge model in ABAQUS, so as to demonstrate the advantages and disadvantages of using HSS constitutive model.

4.1 Model analysis

In ABAQUS, the same scale model with triaxial compression test is established. The model is a cylinder with a bottom diameter of 39.1 mm and a height of 80 mm. The mesh size is 5.5mm, and the calculation model is shown in Figure 14.

The model constraint is the bottom constraint, and the displacement in Y direction is 0. The HSS constitutive model, Mohr Coulomb model and modified Cambridge model should be calculated

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in turn in the setting of material properties. The elastic modulus E in Mohr Coulomb model is calculated according to the initial soil stiffness E_0. The parameters of modified Cambridge model are calculated according to the geotechnical test in Section 3 and the research results of He Ping and Wang Weidong. The parameters of each constitutive model are set according to table 5.

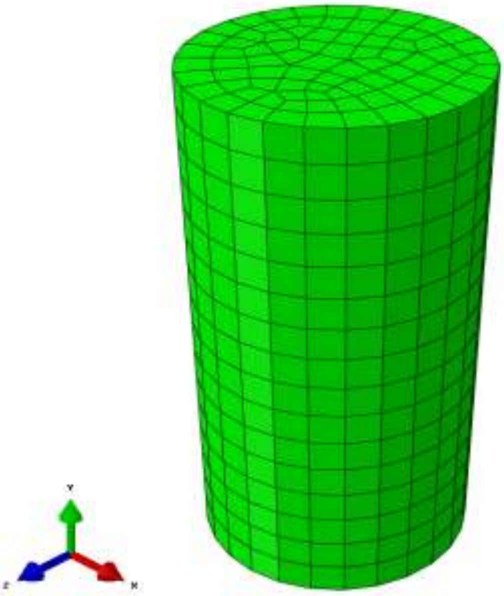

Figure 14 Computational Model Diagram

Table 5 Parameters of constitutive models

constitutive model	Model parameters					
HSS constitutive model	E_{50}^{ref} MPa	E_{ur}^{ref} MPa	E_{oed}^{ref} MPa	p^{ref} MPa	p_c MPa	ν_{ur}
	5.27	17.68	6.04	0.1	0.112	0.198
	c/MPa	φ	ψ	m	e	R_f
	0.027	19.5	2.56	0.512	0.779	0.858
Moore-Koulon Model	E/MPa	ν	c/MPa	φ	ψ	σ_t /MPa
	7.92	0.198	0.027	19.5	2.56	0
Modifyied Cambridge Model	M	κ	λ	β	K	
	1.116	0.05251	0.2258	1.0	0.800	

4.2 Analysis Step Settings

In the process of numerical simulation of triaxial loading, unloading and reloading experiment, there are five steps: 1 confining pressure loading; 2 initial loading; 3 unloading; 4 reloading; 5 internal force balance.

The analysis step of confining pressure loading mainly simulates the process of applying confining pressure in triaxial test. The type of this analysis step is Geostatic in-situ stress analysis, and the length of the analysis step is unit length 1. The method of simulating confining pressure loading is to apply 100 kPa uniform pressure on the top and side of the model.

After the confining pressure is loaded, the initial loading process will be simulated. The type of this analysis step is Dynamic Implicit analysis, and the length of the analysis step is unit length 1. The simulation loading method is to apply a velocity of 4.8 with Y-axis negative direction on the top surface of the model to simulate the loading process from the initial loading to the axial strain of -6%..

After the initial loading, the unloading process will be simulated. The type of this analysis step is Dynamic Implicit analysis, the length of analysis step is unit length 1, the simulation unloading method is to apply the velocity in the positive direction of Y axis on the top surface of the model, and control the final vertical stress close to 0, in which the velocity of Moore Coulomb model unloading is 1.8. The velocity of modified Cambridge model unloading is 0.488, and the velocity of HSS constitutive unloading is 0.688, which are used to simulate unloading to the final vertical stress unloading process close to 0.

4.3 Analysis of stress-strain curves

The stress-strain curves of three kinds of plastic constitutive models in the process of loading unloading reloading are compared with the experimental data, as shown in Figure 15.

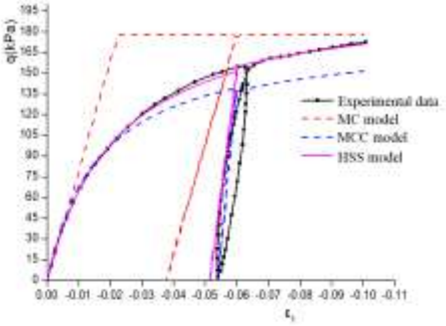

Fig .15 Comparison of simulated and experimental stress-strain curves

It can be seen from Figure 15 that in the initial loading stage, Mohr Coulomb model shows the characteristics of typical linear elastic-plastic model, and the unloading stiffness is consistent with the loading stiffness. HSS model and modified Cambridge model belong to nonlinear elastic-plastic model, the stress-strain relationship presents as a curve, and the unloading stiffness is greater than that of Mohr Coulomb model.

The simulation results of HSS constitutive model subroutine are very consistent with the experimental stress-strain curves, and the deviator stress and strain change are hyperbolic in the initial loading, which is consistent with the results of Duncan Chang model. The results show that the stiffness of unloading and reloading is obviously larger than that of Mohr Coulomb constitutive model, and the fitting effect is excellent when shear yield occurs again after reloading.

When the MCC model, the Modified Cambridge model, used to fit the soil, the fitting effect is better with the soil stress-strain curve at the initial stage. With the increase of strain, the hardening effect is not significant, and the strain value calculated by MCC model will be smaller than the test value.

V. CONCLUSIONS

The silty clay sample preparation was completed in the deep foundation pit project of Lufeng Road Station of Changsha Metro Line 6 under construction and the foundation geotechnical test, confined compression test and triaxial compression test were carried out. Based on the analysis of the test results, the calculated parameter tables of HSS constitutive model in Ganzhou were obtained. The constitutive models of HSS, MC and MCC were compared with the test results by ABAQUS. The triaxial stress-strain curves obtained by HSS constitutive model are very consistent with the triaxial stress-strain curves obtained by test. It can be determined that the test data are scientific and reliable. These parameters can be used in other similar projects simulation and numerical analysis of the project.

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