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Research Article

Geotechnical Characteristic Assessments of Floodplain Soils Using SCPTU Data in Nanjing, China

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In order to improve the understanding of such floodplain sediments and determining the validity of the tests, an extensive series of multifunctional seismic piezocone tests with pore pressure dissipation phase have been performed and supplemented with conventional borings, standard penetration, laboratory testing, and so forth. Sounding results from SCPTU were used to determine the stratigraphic profiles and the soil characteristics of two anchorage sites. A comparison of the boring and laboratory results with the CPTU profiles showed that the CPTU provided excellent information on soil stratigraphy and good guidance for determination of behavior and engineering implications of recent Yangtze River floodplain. At an area where local correlations based on modern SCPTU do not exist, methods for estimating coefficient of earth pressure at rest (K_0), hydraulic permeability (k_h), and equivalent stiffness (G_0) associated with bridge foundation design are presented, compared, and verified. Results also illustrate the complexity and variability of the floodplain stratigraphy and soil properties, which means that the suggestions in this study should be updated when more local experience is obtained. This case study suggests that such enhanced seismic piezocone test should be considered as a potential tool and the instrument of first choice in site characterization programs for design of bridges founded on complicated soils in China.

1. Introduction

As representative of the most dynamic region economically in China, Nanjing city is located on the alluvial and diluvial floodplain of the Yangtze River Delta. In the past and at the expected time of the following five to ten years, 16 cross-river passageways including bridges and tunnels were (or are being) constructed in the top shallow recent floodplain sediments, as illustrated in Figure 1. Undoubtedly, site characterization of recent floodplain soils is the first and the most significant. However, the variation of the palaeoclimate influenced the evolution of the lower riches of the Yangtze River Delta and resulted in a very complicated sedimentary environment. The Quaternary deposit is composed of an alternated multi-sandy-clayey soil; in particular, intermediate soil (silt mixtures and sand mixtures) is widespread. In such situations, it is often difficult to accurately define a complete soil profile and determine soil characteristics including strength, deformation, stress history, flow, and consolidation by using conventional boring methods supplemented by a few laboratory tests.

Construction experience in this area in the last two decades, such as the deep excavation of previously built bridges foundations, subway lines and tunnels under rivers, and retaining walls, shows that the following geohazards occurred or possibly occur during the construction and maintenance of infrastructures: (i) accidents associated with deep excavation, including slope sliding and collapse, quicksand and piping hazards, as well as water seepage through bracing of foundation pit; (ii) pumping induced hazards to surrounding environment, including land subsidence, settlement and cracking of pipeline, and differential

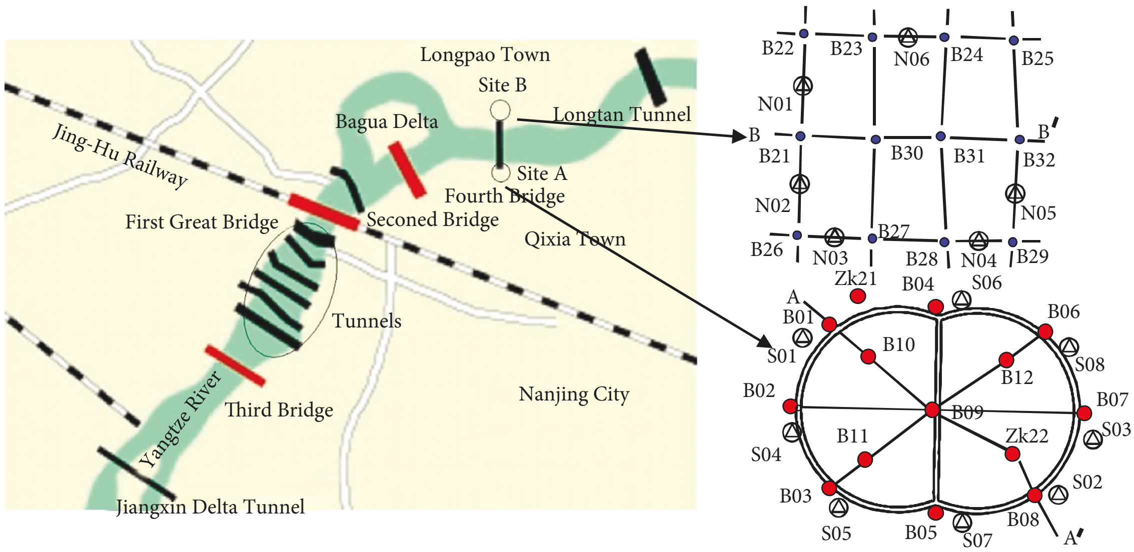


Figure 1: Layout of existing bridges and new alignments crossing the Yangtze River and location of the study area.

consolidation settlement of soft soil; (iii) long-term settlement due to the seasonal fluctuation of groundwater level; and (iv) liquefaction potential of sandy clay and silts during earthquake. The planned Nanjing Fourth Bridge will also face the abovementioned threats during construction, and a limited subsurface exploration was performed in 2007 and 2008 to study bridge alignment for the preliminary design. The exploration programs have included a mix of soil test boring, "undisturbed" sampling, standard penetration test (SPT), predrilled pressuremeter tests (PMT), downhole shear wave testing, and laboratory testing. In reality, it is quite unrealistic and inappropriate to rely solely on soil test boring, a single *N*-value, and a few laboratory tests due to a number of difficulties recognized with routine drilling practices in obtaining field test values, drive samples, and undisturbed samples [1].

As a complement or alternative measure to soil boring with SPT *N*-values and laboratory tests, seismic piezocone penetration tests with dissipation phases are particularly useful for geotechnical site investigation as they can provide approximately continuous simultaneous measurements of tip resistance (q_t), sleeve friction (f_s), and pore pressure, as well as shear wave velocity [2]. The piezocone method is gaining popularity in the Euro-American countries. It has not been, however, much used in China during the past few decades due to the lack of equipment and corresponding application study [3–6].

First, this paper reviews the state of the art of piezocone tests in China and their further development briefly. Then, we study the behavior and engineering implications of the floodplain soil in the Nanjing Fourth Bridge site, concerned by the geoengineers and designers in the case of deep excavation, by using seismic piezocone tests. The primary objective of the research was to compare the results of soil parameters from laboratory tests and in situ tests, with emphasis on SCPTU. Specific comparisons presented here are for results from permeability tests, CPTU profiles, and other in situ or laboratory tests. Geotechnical design parameters obtained from CPTU and dissipation tests are also evaluated in this area through the existing methods. At last, the paper presents an assessment of the applicability of SCPTU tests to interpret the engineering properties of the Yangtze River floodplain sediments and concludes with recommendations for use of the SCPTU for foundation design of bridge in this floodplain.

2. The State of the Art of Piezocone Tests in China and Their Common Problems

As described by literature [3, 4], the cone penetration test without pore pressure measurement is widely used for site investigations in China, which can provide two measurements, q_c and f_s, or just one measurement, p_s. Comparatively, the piezocone tests with dissipation phases can provide four independent readings with depth from a single sounding, as well as time-rate information [7]. Of particular note is the seismic piezocone penetrometer test (SCPTU), which is a hybrid field method, combining the virtues of the CPT with downhole geophysics [8]. With the measurements of q_t, f_s, u_2, shear wave (V_s), and dissipation processes taken together, an entire stress-strain-strength-flow representation can be derived for all depths in the soil profile [9]. The main advantages of the piezocone tests over the conventional Chinese CPT include the following: (i) calibrating measured data to describe soil characteristics accurately; (ii) evaluating soil flow and consolidation characteristics; (iii) distinguishing between drained, undrained, and partially drained strength; and (iv) improving the reliability of soil profiling and classification.

Due to these advantages, the Holland CPTU was introduced into China in the early 1980s. Several research projects on CPTU were performed at Shanghai-Nanjing highway and Zhujiang Delta area [10, 11]. Similar probe cones were produced by Nanjing Hydraulic Research Institute and other research institutes. However, compared with western countries, the reliability and repeatability of the

Chinese piezocone have always been questionable. In particular, the methodologies, digital and multifunctional sensor technology, and application range in China are considerably lagging. For instance, SeisCPT, ResisCPT, and other derivatives were quickly becoming popular in western countries, while their application is very limited in China. During the recent five years, several research institutes in China, including Southeast University, Hohai University, and Tongji University, have introduced the latest equipment from Europe and the United States, for example, the Vertek-Hogentogler CPTU system, which is also used in this study, and the Geotech AB cordless CPT system. The relevant research is now conducted step by step in China, which will be very useful for optimizing the engineering design and enhancing international communication. Furthermore, the CPTU data require a good estimate of correlation coefficients to determine soil parameters, which depend on the geologic formation and can be site-specific. The database of piezocone tests in China is very important for the validation of existing CPTU-based methods.

3. Database

3.1. Project Details and Description of Site. The Nanjing Fourth Bridge will be constructed in the following three to five years, which will be a three-span suspension bridge. The project site is approximately 20.5 km north to the Nanjing Great Bridge between the towns of Longpao and Qixia, where the north and south cable anchorages will be constructed, respectively, as shown in Figure 1. Several underwater tunnels are also considered for connecting the two sides of the Yangtze River. The south anchorage (referred to as Site A) lies in the south bank of the Yangtze River, 150 m to the Yangtze River embankment. The diaphragm wall has been designed as the bracing structure in the case of deep excavation, with the shape of ∞ (82×59 m), the height range from 40 to 50 meters, and the thickness of 1.5 m. Oppositely, the north anchorage (referred to as Site B) is located on the north bank of the river, 90 m to the north embankment, with the shape of rectangle (60×59 m). The open caisson foundation is selected, with the height of approximately 55 m.

3.2. Regional Geology Outline and General Surface Conditions. The project area belongs to the floodplain of lower riches of the Yangtze River. The ground surface is flat, with a mean elevation of 3 to 5 m and a general inclination from west to east. The ground water level is found at 0.85 to 1.35 m, fluctuated with tidal motion and seasonal variation. The geological sketch is marked by alluvial, diluvial, silted, and lacustrine deposits of the Yangtze River Delta. The Quaternary deposits, which range from Late Pleistocene to Holocene, primarily consist of alternating clay to silty clay, slits and sands, and gravel. Due to the varying depositional mechanisms and environments, the stratigraphy is always complex with silt mixtures and sand mixtures widespread. The thickness of Quaternary deposits varies greatly from less than 10 meters to more than tens of meters. The underlying bedrock is primarily formed by Cretaceous sandstone and conglomerate. Occasionally, the soft mudstone and muddy siltstone are interbedded. The deepest depth to bedrock surface underlies the Quaternary deposits at a depth of 34 to 65 m below ground surface.

In a typical vertical profile of the Quaternary sediments in this project region, a dual structure can be identified, the top of which is hydrostatic deposition and the lower part is alluvial, deluvial, and lacustrine deposition, except the top soil formed by arable land or backfilling of only 0.5~1.5 m in thickness. From the top layer down, the grain size in sedimentation becomes coarser, and the deposit changes in sequence of silty clay, mucky silty clay, silt mixtures, sand mixtures, and gravel. Due to the fact that the recent floodplain deposits are characterized by high water content, high void ratio, high compressibility, and low shear strength, as well as low hydraulic permeability clayey soil or complicated intermediate soil (silty mixtures and sandy mixtures), construction experiences from the existing projects show that the shallow recent floodplain soft soil will influence the construction of subgrade and piles, especially in the deep excavation.

3.3. In Situ Testing and Subsurface Investigation Program. As part of the major projects in Nanjing, a detailed site characterization study was carried out at the two anchorage sites in the Yangtze River Delta. One of the sites is near Longpao town and the other is near Qixia town. The total geotechnical investigation program completed in 2007 and 2008 consisted of a combination of laboratory and in situ tests, including 21 borings, various conventional laboratory testings, 8 downhole shear wave testings, 4 predrilled pressuremeter tests, and 14 seismic CPTU. Of particular interests are the piezocone tests designed to deliver more detailed information about the stratigraphy and properties of the soils found on site, taking advantage of the investigations conducted previously and adjacently. It is hoped that a by-product of this tentative research will be development of greater confidence in the CPTU as a site investigation tool in China. The CPTU can be used economically in partnership with other in situ testing methods or laboratory tests, as well as in the establishment of a database of in situ soil parameters for the optimization of bridge foundation design. Furthermore, a comparison is made between the interpreted soil parameters from CPTU and those obtained from laboratory tests and other in situ tests, primarily focusing on the *K0* and the coefficient of the permeability. The data obtained will be also used further for calibration and comparison.

Laboratory testing mainly included moisture content, particle size distribution, Atterberg limits, unit weight, one-dimensional consolidation, direct simple shear, consolidation quick direct shear, undrained triaxial tests, and falling head permeability test. All the laboratory tests were performed in general accordance with the Chinese Code for Investigation of Geotechnical Engineering [12] and the Chinese Standard for Soil Test Method [13], which are compatible with ASTM standards.

As mentioned above, a classical type-2 CPTU device (15 t cone), with a penetration speed of 2 cm/s and readings every 5 cm, was employed in this study, which is produced by Vertek-Hogentogler Co. of USA. The equipment is a versatile piezocone system equipped with advanced digital cone penetrometers fitted with 60° tapered and 10 cm^2 tip area cones, which can provide measurements of five independent readings: tip resistance (q_t), sleeve friction (f_s), penetration pore-water pressures (shoulder u_2), vertical inclination with depth, and downhole shear wave velocity (V_s), which is recorded at 1 m deep intervals during the pause of connecting the rod. Particularly important in piezocone tests, pore pressure dissipation tests can be performed in steady-state in situ conditions at specific depths during a pause following one sounding, yield information about the coefficient of consolidation and permeability of a soil deposit. Note that, to have a good pore pressure response during piezocone penetration, a rigid procedure to assemble and saturate the piezocone system presented by Lunne et al. [14] is employed.

A series of six seismic CPTU were carried out around Site B adjacent to the borings. The investigated depth was generally ranging from 35 to 40 m. Eight other seismic CPTU tests were performed at Site A with depths up to 40 m below ground surface. The test locations were also planned around the designed diaphragm walls.

4. Interpretation and Evaluation of Piezocone Results

4.1. Soil Delineating and Profiling. Based on the boring logs and indoor experiment, summary plots of index parameters are shown in Figure 2, including natural water content, plastic limit, liquid limit, percent fines, as well as the N_{60} value. The profiles of Overconsolidation Ratio (OCR) results obtained from laboratory odometer tests indicate that such floodplain sediments are under normally consolidated to slightly overconsolidated condition throughout the profiles. The overconsolidated state at shallow depths in Site A, where the OCR ranges from 3 to 7, was believed to be caused by man-made construction activities. At great depths, it is likely that the slightly overconsolidated state is caused by a combination effect of aging and removal of overburden.

At Site A, four obvious geotechnical-stratigraphic units are identified based on changes in N_{60} and other laboratory indexes, named ①, ②, ④, and ⑦ from top to bottom, and subdivided using Arabic numbers. The uppermost Holocene sediments of unit ① mainly consist of 5.3 to 11.7 m of silty clay and mucky silty clay, followed by interbedded silty sand and silty clay with depth of 19.9~56.6 m; note that lenses of fine sand can be detected somewhere. The sediments of following unit ②, which is also of Holocene age, are silty sand and silty clay with varying thickness (9.9~30.4 m), which occur interbedded or with a diffuse horizontal lamination of silt or coarse sand and gravel. The next unit ④ of Late Pleistocene age mainly consists of silty sand, gravel, and silty clay with thickness from 6 to 14.8 m; lenses of fine sand may also occur somewhere. Of particular concern is the fact that unit ④ is discontinuous; in particular, the gravel and silty clay are always missing. The underlying stratum is bedrock unit ⑦ of Cretaceous age formed by sandstone, glutenite, and conglomerate which is partially penetrating.

At Site B, the general architecture of the floodplain consists of four lithological units, which from top to bottom are as follows: (i) an uppermost unit of silty clay and muddy silty clay underlain by silty sand with thickness of 14.8~29.5 m, with a silt interlayer also detected in the mucky silty clay layer; (ii) fine sand with lenses of medium sand forming a lower aquifer, usually massive with thickness of 23.9~34 m, with little obvious stratification; (iii) late Pleistocene alluvial deposits made of fine sand angle gravel; and (iv) a lower unit made of Cretaceous siltite.

From the boring logs, the most prominent feature of the profiles is the significant stratigraphic variations due to depositional environment fluctuation, whereas the deposit of Site B is seemingly more simple and consistent than that of Site A. The general architecture frame has been characterized as inhomogeneous bodies with highly interbedded layers of clays, sands, silt mixtures, and sand mixtures. Proper characterization of layered soils, such as layer clay, is important for assessment of anisotropic soil behavior as well as for studying paleoclimatic history.

However, the above-mentioned deposit sequences were mainly identified by visual description of boring logs simply supplemented by limited laboratory tests. For sands/silts or transitional soils (sand mixtures and silt mixtures), limited boring numbers, low sampling rate, unclear soil interface, and thin layering may result in erroneous judgment or loss of important information about the stratigraphy, for example, location of critical layers or soft zones and subtle changes within a deposit. Of particular note is the fact that highly stratified deposits may include small seams, laminations, lenses, and intrusions, each having significant implications on engineering behavior and design, which also result in the complexity. At the same time, subject to the limited budgets of the exploration program, the increase of boring number and continuous core sampling are always unrealistic. So, as a quick, expedient, and economical way, the versatile seismic piezocone tests with dissipation phases offer an optimal complement or alternate for improved site stratigraphy and layer characterization, which are continuous or at least near-continuous soil profiling techniques to delineate subsurface stratigraphy and soil properties.

Representative sets of readings from seismic piezocone tests taken at the two sites are presented in Figure 3. The superpositions of q_t, f_s, and u_2 diagrams for Site A show very poor repeatability in the record data. The results for q_t, f_s, and u_2 display significant variation at most depths. Definitely, highly variations of q_t and f_s are attributed to the highly interbedded deposits, which agree well with the testing boring logs. The frequent spikes in the tip resistance and drops in the pore water pressure show that silty sand or silt seams occur within the silty clay layers. Meanwhile, the highly excess pore water pressure response, strongly influenced by the thin layers, indicates that thin clay layers or silty layers are encountered within silty sand or silt clays. These permeable or impermeable intralayers or small interface

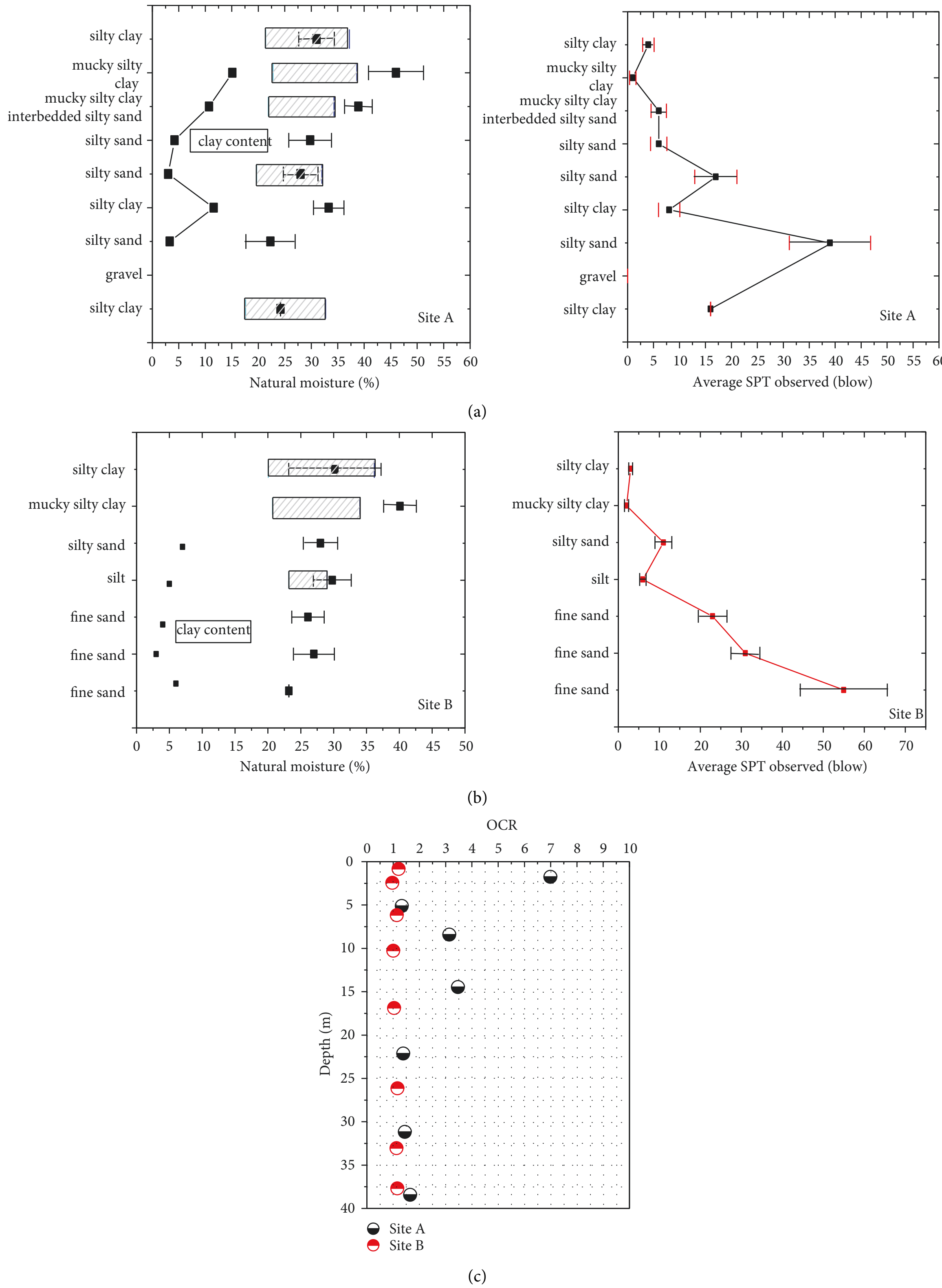
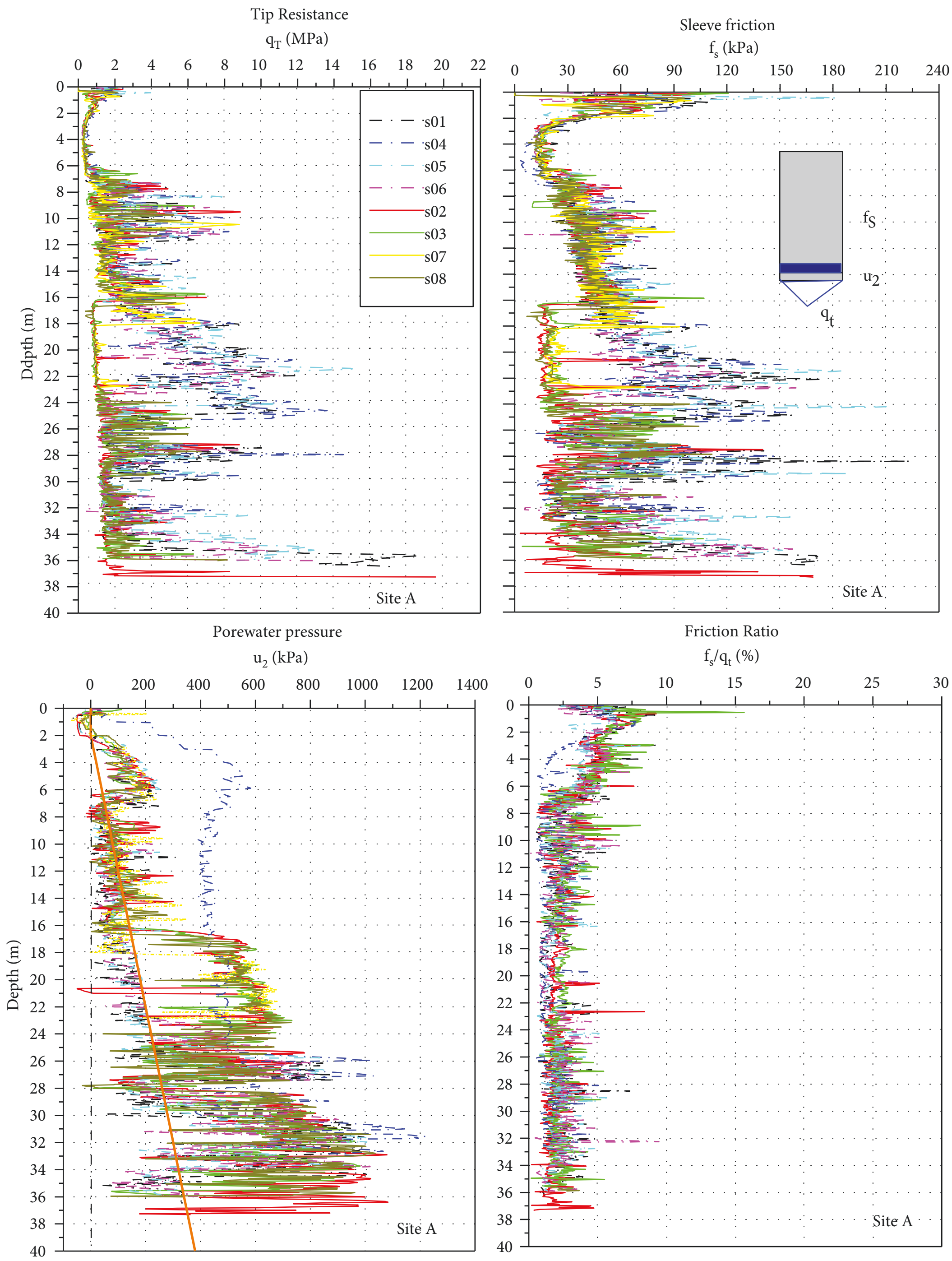


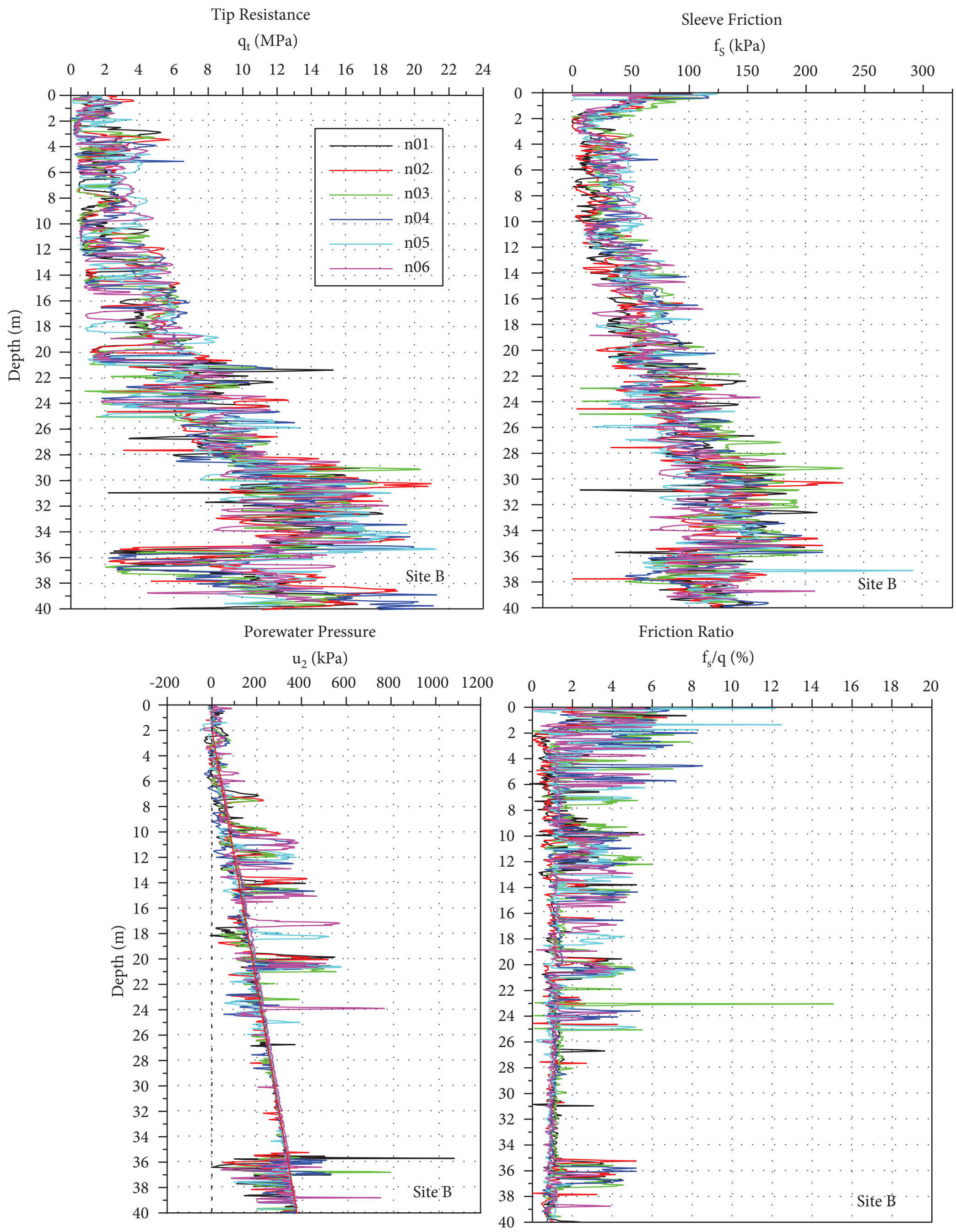
Figure 2: Summary of soil indexes. (a) Average index properties: natural water content, void ratio, Atterberg limits, and percent fines content; (b) average SPT observed; (c) overconsolidation ratio.

changes are always not represented by the discrete SPT test N_{60} values as illustrated in Figure 2 or overlooked on the portion of boring logs. However, detection of stratigraphic interfaces and thin layers can be critical to the construction for these interfaces or weak layers, if sufficiently numerous, continuous, and permeable (or impermeable), may promote



(a)

FIGURE 3: Continued.



(b)

FIGURE 3: Series of piezocone penetration tests at the two sites.

anisotropic soil properties or alter groundwater flow regimes, then resulting in accidents, for example, increasing the likelihood of slope failure in the case of deep excavation, hinder or accelerate rates of consolidation, and so forth.

We also notice that the responses of q_t, f_s, and u_2 appear to be consistent except at the depth from 16 to 22 m. Close examination of the distribution of CPTU holes highlights that the recordings of four CPT holes, s02, s03, s07, and s08 (referred to as Series-1), are similar, which is different from another series of CPT holes, s01, s04, s05, and s06 (referred to as Series-2). The explanation is evident when compared with the cross section obtained from boring logs. The encounter of silty clay at the corresponding depth of Series-1 results in the decrease of tip resistance and large pore water pressure response, while Series-2 reflects the existence of silty sand at the same depth. Of particular note is the abnormal pore water pressure response of s04 caused by malfunction of pore water element when conducting a special resistivity piezocone testing, not discussed in this paper. As representative of Series-1, s03 is selected to conduct soil classification illustratively using the soil classification chat proposed by Robertson et al. [15]. The soil types in Figure 4 predict that, below the depth of about 6 m, many thinly intralayers are identified, supporting the sediment logical variations.

It can be seen that the deposit at Site B is a relatively simple deposit. The repeatability and consistency of recording data are acceptably good, which is also in accordance with boring logs. The prominent feature of Site B is silty sand and fine sand constituted the main soil layer below the depth of about 12 m, overlain (mucky) silty clay interbedded with silt. However, within the silty sand layer or fine sand layer, the low permeability silty clay layers are often encountered as shown in Figure 5, especially at elevations between 12 and 25 m approximately. Even at the top of the site (0~12 m), the thickness of clayey soil (mainly silty clay) at Site B is far less than that at Site A. By contrast, the silty clay layers become weak, thin layers which may be more compressible than nearby materials, often resulting in more differential settlement or uncertain slope failure, of particular interest to geoengineering agencies and companies. The overall pore water pressure response of Site B is very different from that of Site A, the overall trend of which is nearly close to hydrostatic pore water pressure except encountering thin clayey layers. Comparatively, the potential problems associated with the sediments at Site B mainly consist of quick sand, piping, collapse, and water inrushing.

It is shown that using the piezocone with multiple soundings as a supplement to borings significantly improved the precision and accuracy of the anchorage sites delineation for the Nanjing Fourth Bridge project, especially detection of small interface changes and thin permeable or impermeable layers. Moreover, for the Yangtze River floodplain, with seasonally depositional environment, the pore water pressure measurements show particular advantages compared to the conventional boring and the Chinese CPT (generally without pore pressure measurement) in detecting local variations at the small scale of a few centimeters, for example, thin sandy seams within silty clays or thin clayey layers within sandy layers, which is also demonstrated by other literatures [16–18]. The obvious benefit of more reliable knowledge on soil stratigraphy and layer characterization, including the existence, thickness, and composition of floodplain deposits, obtained from boring tests supplemented by piezocone tests, is that such detailedly improved investigations will enable more cost-efficient management of construction processes.

4.2. Coefficient of Earth Pressure at Rest (K_0). In situ horizontal stress, σ'_{h0}, and the coefficient of lateral stress at rest, K_0, are important parameters, both for use in design and as an intermediate parameter in interpretation of CPT results [14]. However, there are presently no reliable methods of determining K_0 from either lab or field tests in fine-grained soils or sandy soils [14].

Many methods have been proposed for estimation of K_0 from CPTU data [14, 19, 20]. Most methods are based either on the OCR or directly on the piezocone measurements. According to different soil types, silty clay, or silty sands, the following methods were selected in this study to predict K_0 from CPTU data. These methods are summarized as below.

Schmertmann [21] suggested estimating K_0 based on the OCR as follows: firstly, from CPTU data either estimate S_u and then S_u/σ'_{v0} or estimate OCR; then use the plasticity index and S_u/σ'_{v0} or OCR estimate K_0 from a correlation chart [22] in fine-grained soils or using the following equation:

$$K_0 = \left(1 - \sin\phi'\right) OCR^{\sin\phi'}. \tag{1}$$

Kulhawy and Mayne [23] related K_0 with the normalized net cone resistance $(q_t - \sigma_{v0})/\sigma'_{v0}$ and suggested the following equation:

$$K_0 = k\left(\frac{q_T - \sigma_{v0}}{\sigma'_{v0}}\right). \tag{2}$$

With $k = 0.1$, which is used for uncemented, unaged, and mechanically overconsolidated fine grained soils, the value of k may be soil type and site dependent.

If it is possible to assess OCR from geological evidence or from neighboring clay layers, then K_0 may be derived from the following empirical correlation [24]:

$$\frac{K_{0(oc)}}{K_{0(nc)}} = OCR^m, \tag{3}$$

where K_0 (nc) corresponds to the NC horizontal stress coefficient. For clays, $K_{0(nc)} = 1 - \sin\phi'$; for sands, $K_{0(nc)} = 0.95 - \sin\varphi'$; φ' is the effective stress friction angle. Lunne and Christophersen [24] recommended $m = 0.45$. However, Mayne and Kulhawy [25] recommended $m = 0.65$. Moreover, Mayne [26] tentatively suggested the following formula for practical use in coarse-grained soils:

$$K_0 = 0.35\,\mathrm{OCR}^{0.65}. \tag{4}$$

Using a large database ($n = 590$) compiled from 26 separate series of calibration chamber tests, Mayne [9] suggested the following simplified regression equation for NC and OC sands ($r^2 = 0.871$):

$$K_0 = 1.33\left(q_T\right)^{0.22}\left(\sigma'_{v0}\right)^{-0.31}\mathrm{OCR}^{0.27}, \tag{5}$$

where q_t is in MPa and σ_{v0} is in kPa. It should be noted that the formulation applies only to unaged and uncemented quartzitic sands and has been verified by a limited number of field test sites [27].

For mixed soils (sands, silts, and clays), when $0.1 < B_q < 1.0$ and with a range of $20^\circ < \varphi' < 45^\circ$, an approximate form for effective stress friction angle from NTNU method is given [28] by

$$\varphi'\,(\mathrm{deg}) = 29.5^0 B_q^{0.121}\left[0.256 + 0.336 B_q + \log Q\right], \tag{6}$$

where B_q is pore pressure parameter $(=(u_2 - u_0)/(q_t - \sigma_{v0}))$ and Q is normalized cone resistance $= (q_t - \sigma_{v0})/\sigma'_{v0}$.

Additionally, for $B_q < 0.1$ corresponding to granular soils, the following expression for clean sands would apply [23]:

$$\varphi'\,(\mathrm{deg}) = 17.6 + 11.0 \cdot \log\left[\frac{q_T}{\sqrt{\sigma_{v0}/\sigma_{\mathrm{atm}}}}\right]. \tag{7}$$

Then equation (1) is used to estimate K_0.

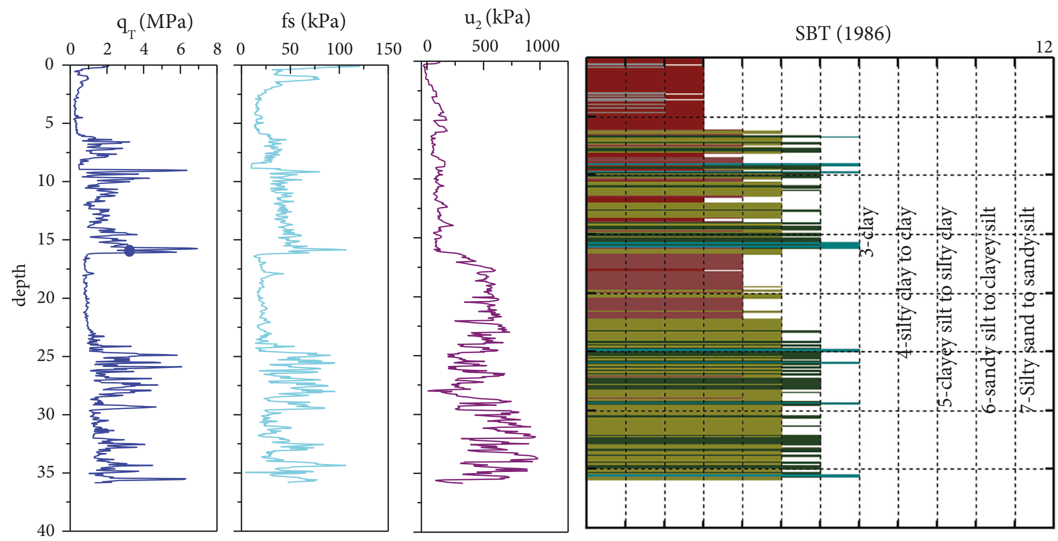


Figure 4: Illustrative profile of soil classification of s05 using Robertson et al.'s method [15].

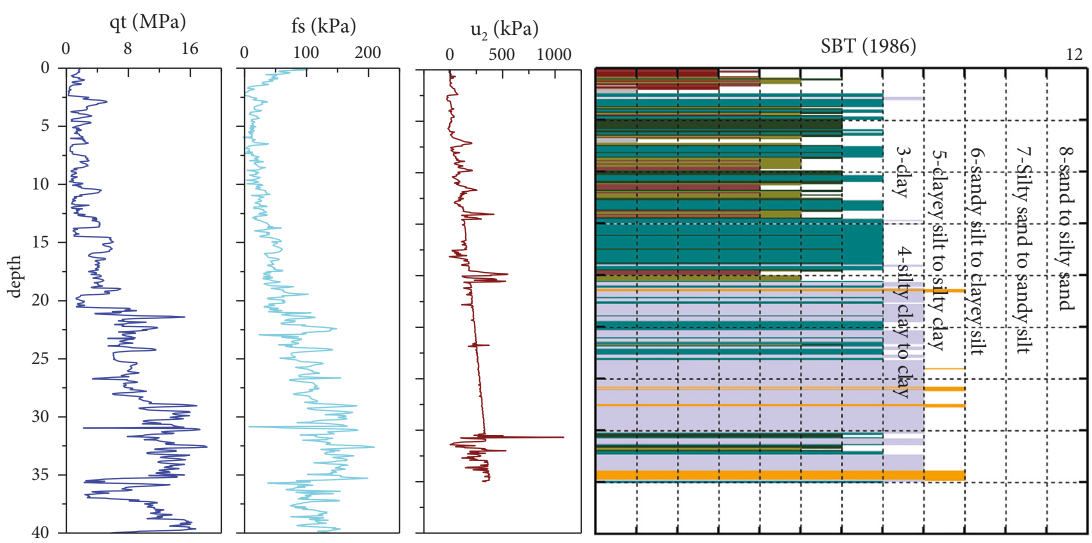
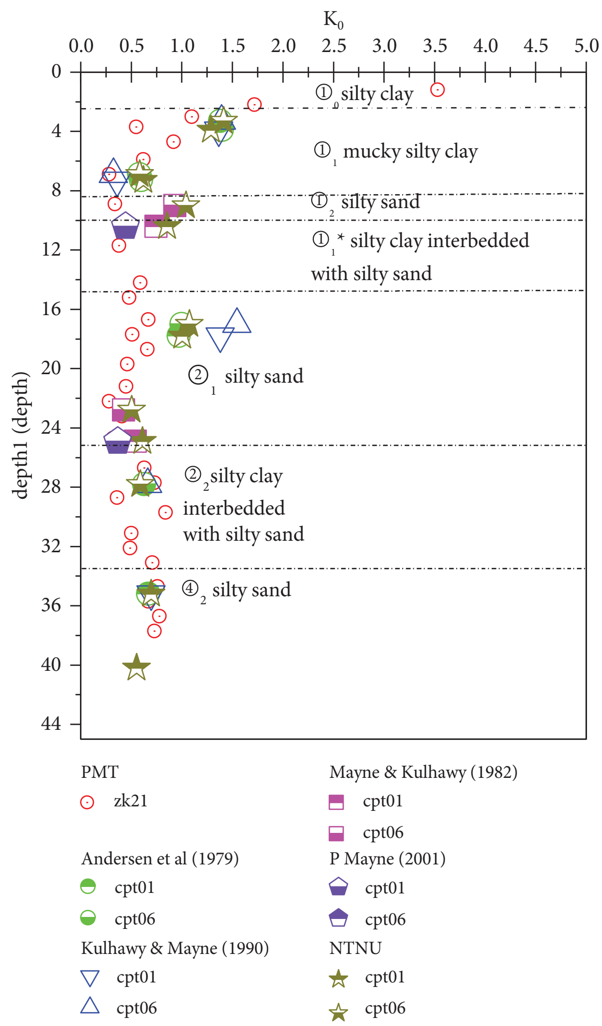


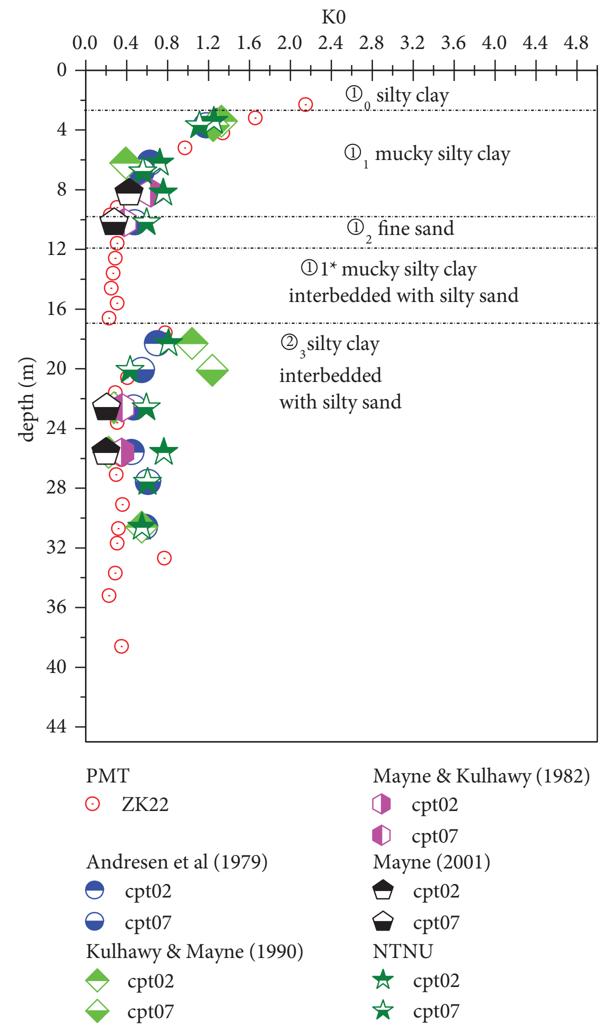
Figure 5: Typical results showing soil types of N01 at Site B.

In this study, the aforementioned methods were evaluated for their capability to reasonably predict K_0 utilizing the piezocone data. These methods are referred to as the Andersen and Kolstad [22] method, the Kulhawy and Mayne [23] method, the Mayne and Kulhawy [25] method, the Mayne [9] method, and the NTNU method [28] method. It should be noted that these methods are applied to different soil types, respectively. The first two are both used for fine-grained soils, the next two are used for coarse-grained soils, and the last one is used for mixed soil types. The predicted K_0s were then compared with the measured K_{0s} obtained from the predrilled pressuremeter tests at the same sites. Figures 5(a)–(c) present this comparison for different methods. The soil profiling was accepted as given in the case records.

Based on the results of this analysis, the overall performance of these CPTU-based methods was then examined. Taking the PMT measured K_0 as the actual reference values, it is indicated clearly that the overall reasonable trends are observed although considerable scatter exists between all the selected prediction methods. Again, Figure 6 shows that at Site A all the prediction methods tend to overestimate the measured K_0, while at site B all the prediction methods tend to slightly underestimate the measured K_0. In general, at Site A, the selected methods exhibit a higher dispersion than that at Site B due to the more highly stratified and layered nature of the deposit. Moreover, for intermediate soils (silty clay, silty sands, and silts), the variable estimated K_0 values show more uncertainty than those for relatively homogeneous sands, except for some of data points in Figure 6(c). For fine-grained soils (silty clay or mucky silty clay), the estimated K_0 values from the Andersen and Kolstad [22] method and the NTNU method [28] method are more close to the reference values from PMT tests compared to those from the Kulhawy and Mayne [23]

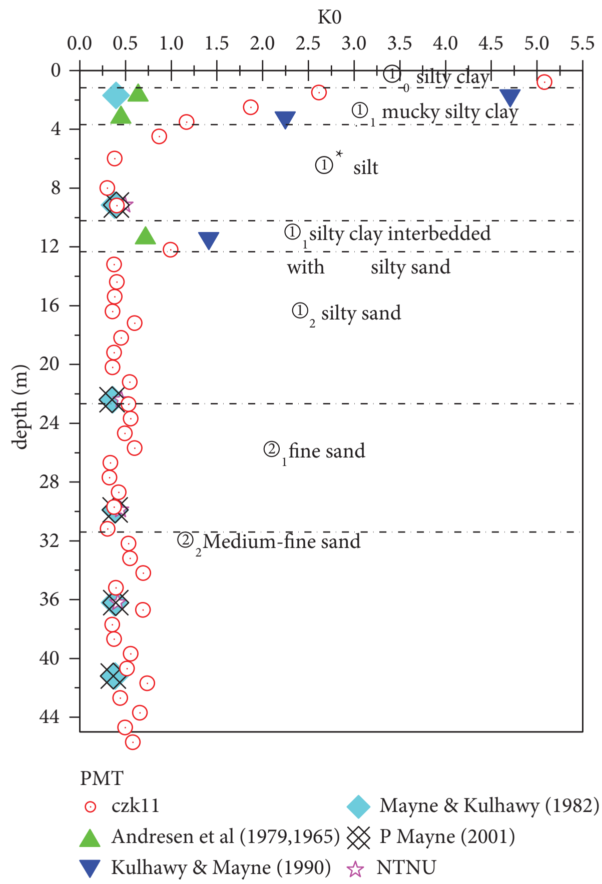


(a)



(b)

FIGURE 6: Continued.



(c)

FIGURE 6: Estimated versus measured K_0 using different methods.

method. For coarse-grained soils (silty sand or sands), the estimated K_0 values from the Mayne and Kulhawy [25] method, the Mayne [9] method, and the NTNU method [28] are similar, especially the Mayne [9] method, which shows relatively good agreement to the reference values. Meanwhile, due to the disturbance when conducting predrilled pressuremeter tests, the reference K_0 values may be uncertain more or less, and the floodplain sediments have highly stratified characteristics; it is difficult to conclude that one method is definitely superior to another method.

Based on the above discussion, it is recommended to evaluate K_0 in such floodplain sediments from CPTU as follows: For projects where little experience is available, the Andersen and Kolstad [22] method and the NTNU method [28] method are recommended to estimate K_0 for fine-grained soils, while the Mayne [9] method and the NTNU method [28] are applied to coarse-grained soils. It is specially noted that the above values muse be considered as a guide. If previous experience is available in the same deposit, the estimated values should be adjusted to reflect this experience.

4.3. Permeability Evaluations. Knowledge of hydraulic properties of soil deposits is one of the most critical aspects of geotechnical engineering, because it determines the rate of flow of groundwater through the subsurface, which controls seepage in rock and soil. Additionally, it is of great importance for geoenvironmental risk assessment involving groundwater inflow into excavations and basements, as well as for water resources management, consolidation, and dewatering [29–33], which are also the concerned geotechnical aspects for the deep excavation engineering in the Fourth Yangtze River Bridge sites.

The coefficient of permeability in the horizontal or radial direction (k_h) can be obtained with field tests, such as the pumping tests or the Matsuo Akai permeability test, and laboratory tests, such as constant head permeability test or falling-head permeability test. These conventional methods, however, may be time-consuming and expensive and even of low reliability. So, some researchers give another way to augment hydraulic conductivity data using seismic piezocone penetration tests with dissipation phases. Several methods for CPTU dissipation tests interpretation have been proposed [34–38].

Typical pressure dissipation tests performed at Site A indicate that u_2 pore pressure decreases almost monotonically. Due to the approximately monotonic pore pressure response exhibited in the dissipation tests, the following empirical methods are used to infer the horizontal soil permeability (k_h). Common methods mainly include (i) the

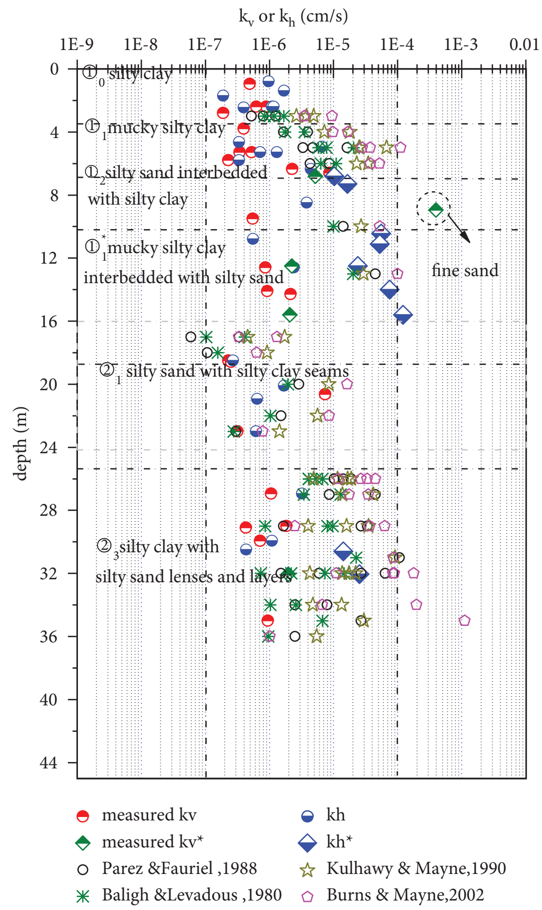


FIGURE 7: Comparison of vertical or horizontal k values obtained from laboratory tests to k_h values obtained using SCPTU-based methods (note: *samples with silty sand seam).

Baligh and Levadoux method [39], (ii) the Parez and Faureil method [40], (iii) the Burns and Mayne method [36], and the Kulhawy and Mayne method [23].

In order to assess the aforementioned methods, comparisons were made with laboratory-determined permeability values from falling-head permeability tests. The results of the comparison for the soft clays at Site A are presented in Figure 7, demonstrating significant scatter between different methods. With the exception of some measurements in the homogeneous clays, most of the lab k_h values are significantly lower than the CPTU-determined k_h values. This is especially the case for the great depth at the ranges of 10–14 and 28–36 m. The reasons are mainly related to the layer clay containing embedded and more or less continuous permeable layers or discontinuous lenses and the unavoidable disturbance of laboratory tests. The grain content curve also reflects the nonhomogeneity of the floodplain sediments (see Figure 8), which raises the uncertainty of the prediction.

Overall, it is observed that the lab measured k_h values are generally lower than those estimated from SCPTU within 1-2 orders of magnitude or even 3 orders. The CPTU-determined k_h and the laboratory-determined k_h values from samples with thin silty sand layers, however, are approximately within the same range (see Table 1). An important aspect is the relationship that exists between the lab-measured horizontal permeability and vertical permeability at Site A with an average of $k_h/k_v = 2$, indicating anisotropic characteristic of the floodplain sediments. At some depths, this value reaches 10 or more, even 60, which is also in line with the description by Robertson [37].

If taking the predicted k_h values from the Parez and Faureil method as a check, the horizontal coefficient of permeability k_h obtained from the Baligh and Levadoux method shows similar trend but generally smaller. Meanwhile, k_h values determined using the other two methods were significantly larger than the reference values. Of additional note is the fact that the reported field pump tests gave more large values of k_v (averaged 4.97E-3 cm/s), mainly because of the existence of highly permeable silty sand interbedded with silty clay. According to the approximate estimate of soil hydraulic conductivity using the nonnormalized (or normalized) CPT/SBT chart by Robertson et al. [8, 41], the results from Kulhawy and Mayne method and Burns and Mayne method seem more reasonable. Additionally, while the two latter methods are based on very different approaches to evaluating the permeability, strikingly similar results are produced. For the most part, the velocity-based method tends to significantly overpredict the laboratory-measured values of hydraulic permeability. The reason may be involved in speed effect. It must be pointed out that the estimation of soil

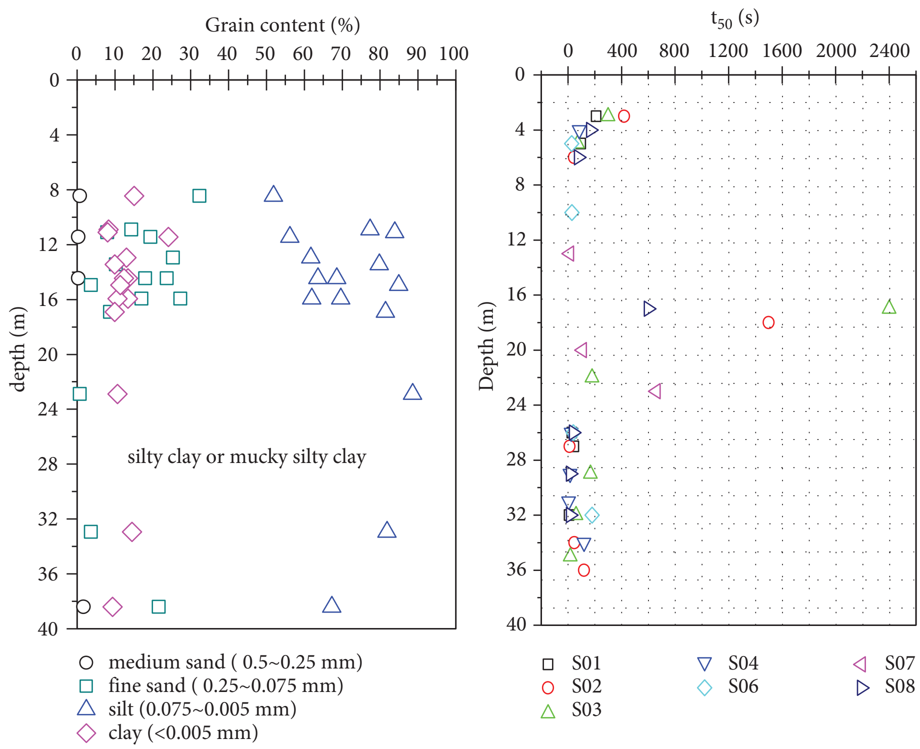


FIGURE 8: Grain content curve and t50 from pressure dissipation tests.

TABLE 1: Summary of results from laboratory and CPTU evaluations of hydraulic conductivity (k_h and k_v).

Soil type	SCPTU predicted k_h (10^{-7} cm/s)				lab k (10^{-7} cm/s)	
	Parez and Faureil	Baligh & Levadoux	Kulhawy and Mayne	Burns and Mayne	k_v	k_h
①$_0$ silty clay	5.3~39.4 (16.3)	8.4~35.3 (18)	26.4~180.3 (73)	35.1~168 (86.3)	1.91~1.93 (1.92)	3.98~17 (10.5)
①$_1$ mucky silty clay	33.3~162.6 (74.5)	58.2~207 (103.3)	230.7~671.1 (351.5)	279~1110 (532.4)	2.29~5.28 (3.67)	3.38~165* (53)
①$_1$* mucky silty clay interbedded silty sand	142.6~448.2 (295.4)	99.4~202.5 (151)	270.1~295 (282.5)	526~994 (760)	5.53~24.1 (16.3)	239~1209* (656)
②$_3$ silty clay interbedded with silty sand	15.2 ~1065.9 (192.2)	7.3~226.2 (53.8)	9.2~908.8 (147.2)	6.3~889 (288.5)	2.3~19.7 (8.64)	2.7~253* (58.6)

Notes: min~max (average); *values from samples with thin silty sand layers.

permeability from CPTU dissipation data is relatively uncertain and should be used as reference only.

The results of this study show the remarkable variability in k_h value when using different test methods and different predicted methods. Some of this variability is due to factors such as variable soil properties, stratified and layered nature of the deposit, specimen size and orientation, sample homogeneity, different boundary constraints, and the particle size produced by the method of placement. Another important source of variability is the different measuring locations of q_t, u_2, and shear wave velocity along the shaft which may result in serious influence on the predicted values at such highly stratified deposits. It may also be argued that no one method was found to be superior to the others based on this limited set of tests performed on such silt mixtures and sand mixtures. Of future interest, the grain composition and depositional environment may be subtly investigated and considered when using the aforementioned predicted methods. Although these estimates are approximate at best, they can provide a guide to variations of possible permeability [39].

4.4. Prediction of Equivalent Stiffness. The deformation characteristics of soils include the consolidation indices (Cc, Cs, and Cr) and elastic moduli (E and G), as well as rate and creep parameters. The stiffness of soils is needed in evaluating deflections of shallow and deep foundations, retaining walls, excavations, and embankments, in addition to site-specific seismicity and amplification analyses [9, 42]. In fact, most of the activity of interest in earthwork deformations takes place close to the in situ K_0 state and corresponding small-strain region characterized by G_{max} or expressed by the initial soil stiffness G_0, given as the following formula:

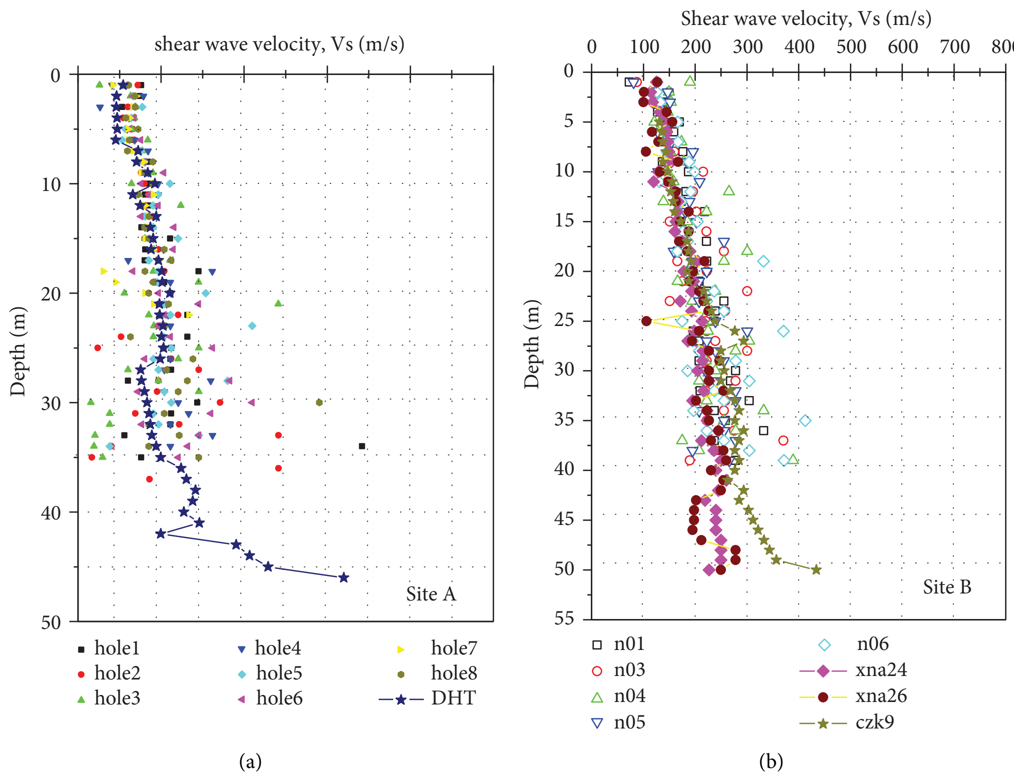


Figure 9: Comparison of SCPTU and DHT data from Site A (a) and Site B (b).

$$G_0 \text{ or } G_{\max} = \rho_T V_s^2, \tag{8}$$

where ρ_T is total mass density $= \gamma_T/g$, γ_T is soil unit weight, and gravitational constant $g = 9.8\,\text{m/s}^2$. V_s is shear wave velocity determined by various field and laboratory methods [43]. Seismic piezocone tests (SCPTU) provide an economical and expedient means of assessing small-strain properties (G_0) and large-strain behavior ($\tau_{\max}$) of soil deposits from a single sounding and measurements are taken at complete opposite ends of the stress-strain response for soils. In this paper, a series of SCPTU soundings are performed for site-specific mapping of V_s in conjunction with a conventional downhole series (DHT) for the Nanjing Fourth Bridge project. Figure 9 presents the results derived from downhole testing using the SCPTU and DHT. The two methods appear in general agreement and confirm that the SCPTU is advisable. The general trends of V_s profiles are evident and show the increase with depth, followed by a suddenly big change at greater depths of 40 m. The fairly low shear wave velocity also can be seen at the depths where there are mucky silty clay deposits at Site A. As expected, it is obvious from these figures that SCPTU-measured V_s show more scatter than those obtained from DHT tests. Moreover, the variability of Site B is also relatively less than that of Site A. This is corresponding to the complexity of such floodplain sediments.

It is also interesting to observe that the DHT-measured V_s values seem to be equal to the average of SCPTU-measured V_s values. This is especially obvious from Figure 9(a). In other words, considering the high cost of DHT tests and their time consumption, SCPTU can be taken as a main means for V_s measurement, while the DHT test can be used as a check.

In order to derive the profile of initial stiffness which is particularly valuable for both static and dynamic geotechnical analyses, the following equation proposed by Mayne [9] is used:

$$\rho_T = 0.85 \log V_S - 0.16 \log z, \tag{9}$$

where z is depth below the soil surface in meters and V_s is in m/s. The obtained profiles of saturated mass density at Site A and Site B are illustrated in Figure 10. It can be seen that the predicted values using equation (9) are generally lower than the laboratory measured values. In this case, by using data from silty clay, silts, and silty sand, the following correlation between V_s, ρ_T, and depth was developed ($n = 263$, $r^2 = 0.99$, shown in Figure 11):

$$\rho_T = 0.89 \log V_s - 0.13 \log z. \tag{10}$$

At some cases without V_s measurements, $V_s - q_t$ relations also can be possibly established because the cone tip resistance and shear wave velocity depend on the effective geostatic state of stress [42]. Burns and Mayne [44] related V_s with q_t and void ratio (e_0) and suggested the following equation to estimate V_s from the piezocone data:

$$V_S = 9.44 (q_t)^{0.435} (e_0)^{-0.532}. \tag{11}$$

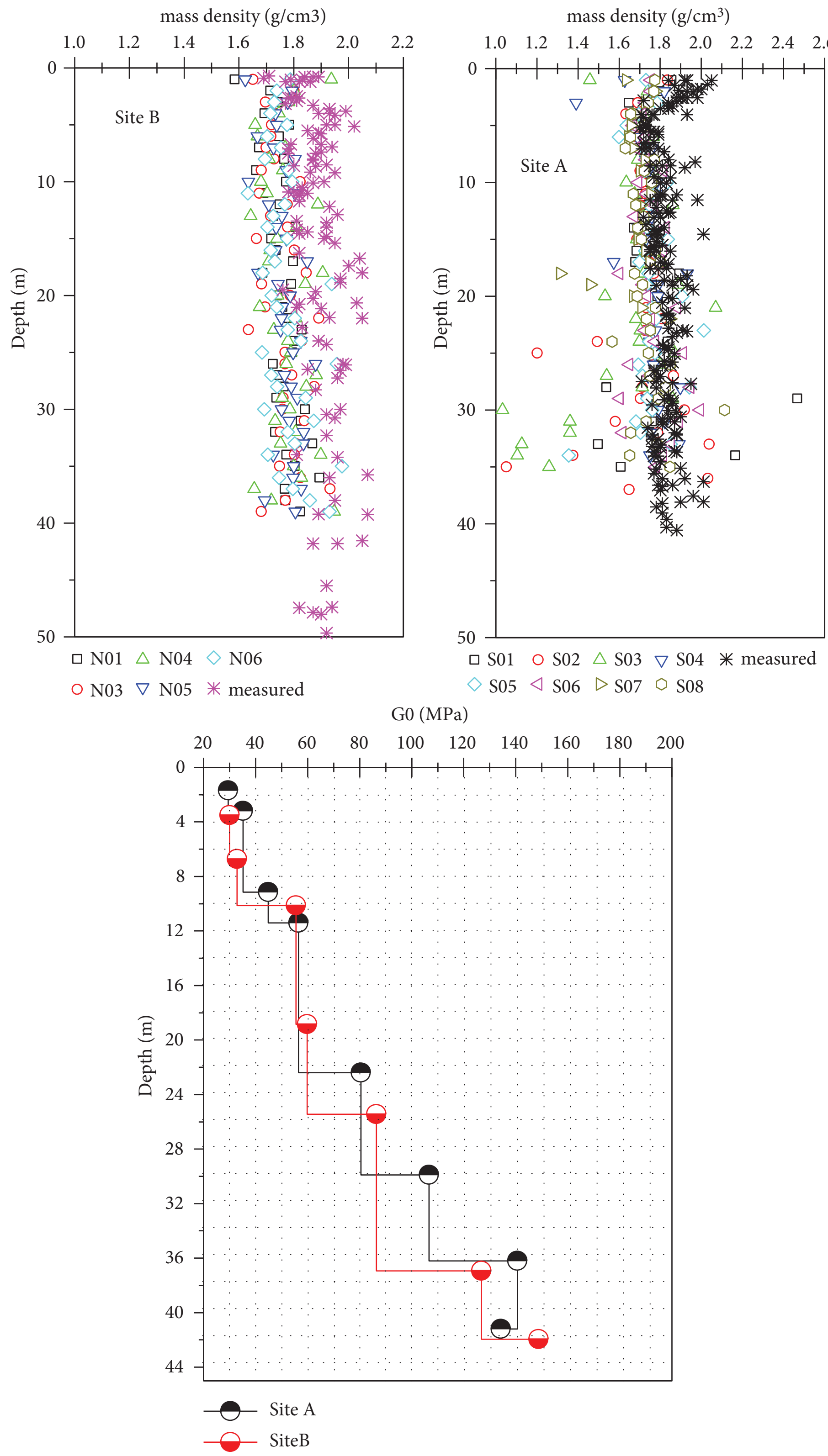


Figure 10: Mass density and initial shear modulus profiles at Sites A and B.

Figure 12 shows the comparison between measured V_s and predicted V_s using equation (11) at Sites A and B. Somewhat scatter can be seen, especially for sandy soils, maybe due to the database in which the aforementioned equation used mainly consists of only clay sites. So, the more reasonable correlations for Sites A and B can be obtained by

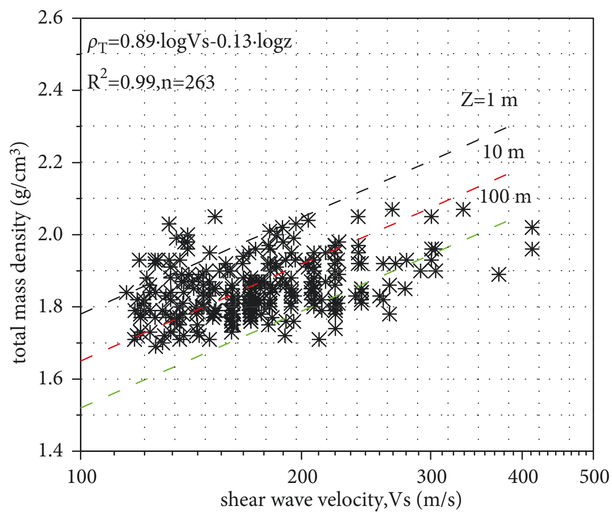


FIGURE 11: Correlation for specific density from depth and V_s.

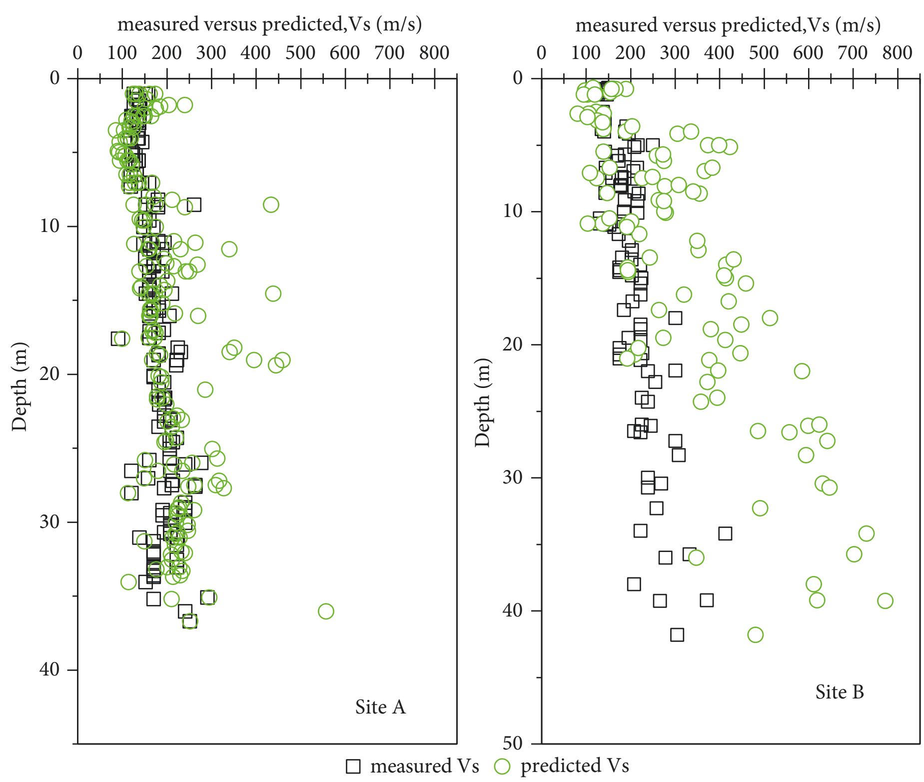


FIGURE 12: Measured versus predicted shear wave velocities at Sites A and B.

multiple regression. By using data from intact clays and sands, the following correlations between V_s, q_t, and σ_v' or e_0 were developed:

$$V_S = 229 (q_t)^{0.11} (\sigma_v/)^{0.15}, n = 263, r^2 = 0.62, \quad \text{where } V_s \text{ (m/s)}, q_t \text{ (MPa) and } \sigma_v' \text{ (MPa)},$$
$$V_S = 38.8 (q_t)^{0.21} (e_0)^{-0.03}, n = 263, r^2 = 0.75, \quad \text{where } V_s \text{ (m/s) and } q_t \text{ (KPa)}. \tag{12}$$

Figure 13 indicates the trend and resulting statistics from multiple regression with data superimposed for comparison. A strong correlation was observed between the cone resistance and the measured shear wave velocity. Rough

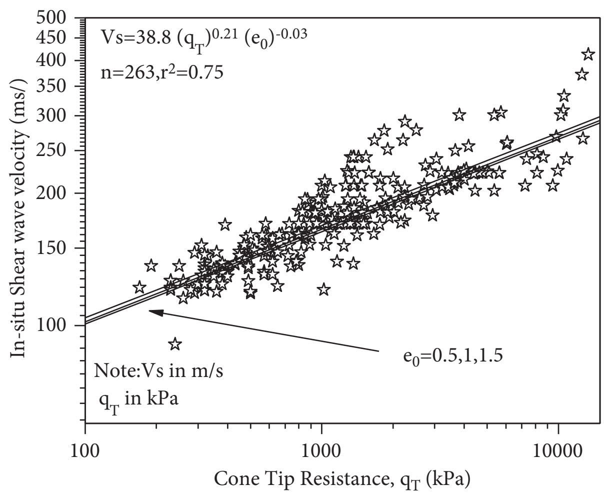


Figure 13: Shear wave velocity-cone tip resistance-e_0 correlations at Sites A and B.

estimates of V_s solely as a function of q_t and σ_v' can be made; however, the correlation will be definitely improved when the void ratio is also included as a correlative parameter. This is also verified by Burns and Mayne [44]. Because e_0 is recognized to change slightly within the range of 0.6~1.45 (averaged value = 0.97), the influence of e_0 is not noticeable in this study. Of particular importance is the fact that, through inversion of the equations, e_0 and ρ_{sat} can be evaluated approximately and immediately.

5. Conclusions

The piezocone (CPTU) is a widely accepted tool in western countries. However, CPTU has limited use in China due to some complicated reasons. In this study, several series of SCPTU data were collected at two bridge anchorage sites in Nanjing, China. Although the number of sites involved in the study was limited, some valuable findings resulted from this study, including the detailed subsurface stratigraphic profiling and evaluation of soil properties in the Yangtze River floodplain. Such information is useful in the planning phase of any civil engineering works. It also allows better design of field surveys, including the selection of the most appropriate techniques to use. Comparisons made between the original soil boring logs, conventional field and laboratory tests, and piezocone tests showed that the evaluation of soil behavior based on SCPTU was reasonably accurate at these sites. The following conclusions can be drawn:

(1) A comprehensive program has been directed at the improved understanding of Yangtze River floodplain soils which are comprised of intermediate soils (silty clay, silts to silty sands, etc.). It is evident that the use of multifunctional SCPTU system in conjunction with drilling, sampling, and lab testing has improved the ability to resolve small changes in soil stratigraphy and associated soil properties, such as stratigraphic interfaces, thin soil layers, lenses and inclusions, and intralayer properties. The feasibility of using piezocones with pore pressure measurements to improve site stratigraphy and layer characterization for the Yangtze River Bridge project was demonstrated. The Yangtze River floodplain sediments are unusual in that they exhibit behavioral features of both clays and sands, thus creating a complicated situation in practice.

(2) Five existing CPTU-based methods to predict K_0 for the Yangtze River floodplain soils are evaluated and compared to the PMT-based method. The outcome of the study showed clearly that the Andersen and Kolstad [22] method and NTNU method [28] can be utilized to estimate K_0 for fine-grained soils, while the Mayne [9] method and the NTNU method [28] can be applied for coarse-grained soils.

(3) The applicabilities of four piezocone-based methods to predict the hydraulic permeability are compared and evaluated for intermediate soils (silty clay to silts). Although no one method seems to be superior to the others for determining k_h in this study, the variability of results reflects both the stratified and layered nature of the floodplain deposit and the variability in particle composition. Compared to the laboratory falling head tests, the CPTU-determined k_h values are generally larger than the lab-measured values within 1-2 orders of magnitude. If taking both the laboratory and field pumping testing and experiences into consideration, the results from Kulhawy and Mayne method and Burns and Mayne method based on CE-CSSM model seem more reasonable.

(4) The seismic piezocone provides information about soil behavior at very small and high strain within the same sounding. Empirical $V_s \sim q_t$ and $V_s \sim \rho_T$ relations in layer soils at the Fourth Bridge site are developed for estimating the low-strain shear modulus ($G_{max} = \rho_T V_s^2$), which is important in the design of statically and cyclically loaded foundations. Moreover, the proposed relationships can be used to obtain preliminary G_{max} profiles of similar floodplain sediments in the absence of direct measurements of shear wave velocity.

Data Availability

The data used to support the findings of this study are included within the Supplementary Materials.

Conflicts of Interest

The authors declare that they have no conflicts of interest to report regarding the present study.

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Supplementary Materials

Detailed information on 14 seismic CPTU tests: (1) 8 seismic CPTU tests at Site A. (2) 6 seismic CPTU tests at Site B. (*Supplementary Materials*)

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