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A Study on shear strength of $aQ_1{}^3vp$ sand in the underground railway line Nhon - Hanoi Railway Station

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ABSTRACT

The objective studied in this article is sand distributed in depth of ten meters to more than 30 meters from the surface. They were located in the underground railway line Nhon - Hanoi Railway Station. In the present study, a series of consolidated drained triaxial tests have been carried out to clarify the shear strength properties of sand. The test results showed that effective cohesion (C') change from 1 to 6kPa, effective internal fiction angle (φ') change from $30^0 26'$ to $38^0 10'$. The shear strength of these soils influenced by some parameters such as clay content, the relative density and types of soil. Effective cohesion increases by the increment of clay content and effective friction angle increases with increasing relative density (N value); the high values of φ' for SW, SW-SM sands. The shear strengths of these soils provide parameters to calculate the stability of underground construction.

1. Introduction

1.1. Ground vibration

In the world, there are some authors studied on the mechanical properties of sand, especially on shear strength. (Bjerrum, et al., 1961) studied in the shear strength characteristics on triaxial equipment in drained and undrained conditions for dense, medium dense and loose sand. The results of normal tests were on the dense and medium dense sand samples. The unexpected results were on loose sand samples.

The other recent research results also showed the change in shear strength of dense and loose sand under drained or undrained conditions, the effect of the initial state on this change. The resistance of the soil is especially critical (Jeffries and Been, 2006; Zlatović, 2006; Kvasnička and Domitrović, 2007; Sokolić, 2010; Jure Ofak, M.C.E., 2015; Jure Ofak, MCE, 2015) studied in the characteristics of sand in triaxial experiments for EROVEC sand, studied on drainage or drainage systems, determination of hardness of sandy soil, critical state of sandy soil (e-p ') as well as q-p'; e- p'; q-ε_q; ε_v-ε_q for the CID test and q q-p', e-p', q-ε_q, Δu- εq for the CIU test.

In Vietnam, the results of the research on the characteristics of shear strength of sand are limited. The shear strength is not studied in laboratory. It only studied in the results of field experiments.

A study on shear strength of sand in the

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laboratory is very important. It can have a simulation of working condition of sand in the field after constructing underground. The test is experimented on sample preparation and laboratory conditions in triaxial test. In this simulation, sand is consolidated and shears in compression with drainage at a constant rate of axial deformation. In the triaxial test, it is necessary to control the saturation of sand sample. So, in the paper, a series test in drained condition on sand samples were carried out to determined shear strength of sands. It provides parameters for calculating underground construction.

2. Methods

2.1. Testing methods

The shear strength of sand in consolidated drained triaxial compression test (CD) is based on American standard ASTM D7181 and Bristish standard BS 1377: 1990: part 8. The test stages include: (1) Sample preparation; (2) Assembly and saturation; (3) Sample consolidation: (4) Shear.

2.1.1. Sample preparation

The American standard ASTM D7181 and Bristish standard BS 1377: 1990: part 8 mentions the method of sample preparation. But, it is not to mention that the sample preparation corresponds to the field density. Therefore, the following procedure for the most accurate sample preparation includes:

a. Samples are prepared corresponds to density in the field. To determine the relative density, it is necessary to determine the following parameters:

- Initial moisture content (W, %), specific gravity (Δ, g/cm^3).

- Maximum void ratio (e_{max}) and minimum void ratio (e_{min}) of sand.

- Relative density of the soil sample. The relative density of soil sample (D) was tested according to the soil conditions in the field (based on the *SPT*).

b. After that, nature void ratio (e_0) of sand, dry unit weight and unit weight were determined from equations: $e_0 = e_{max} - D*(e_{max}-e_{min})$, $\gamma_c = \Delta/(1+e)$ and $\gamma = \gamma_c(1+W)$.

From volume of the mortar (V, cm^3), volume of soil (M) is given to the compaction model corresponding to the density of the sample expected to be tested, $M = V.\ \gamma$ (g). Sand samples are prepared for the triaxial cell pedestal by mixing with water and placing in 10 layers using the compaction technique.

c. After sample preparation, the specimen was weighed and unit weight of the sample was calculated.

d. Sand was added sufficient water to produce the desired water content. The amount of water can be calculated such as $q=0.01m/(1+0.01W_1)*(W-W_1)$, with m - mass of soil samples, W_1- moisture content of sand before adding water, w- the desired of sand]. Soil shall be mixed with sufficient water and store the material in a covered container for at least 16h prior to compaction.

2.1.2. Mounting specimen

Before mounting the specimen in the triaxial chamber, the porous disks and specimen drainage tubes are checked that they are not obstructed by passing air or water through the appropriate lines. The specimen drainage lines, the pore - water pressure measurement device are filled by de-aired water. The specimen is placed with two saturated porous disk and two filter - papers on top and bottom of the specimen and one filter - paper around the specimen. The soil specimen was covered around with the rubber membrane.

The objective of the saturation of the sample is to fill all voids in the specimen with water without undesirable pre-stressing of the specimen, allow the specimen to swell, or causing migration of fines.

The specimen is saturated by increments of cell pressure and back pressure. The cell pressure increment stages are carried out without allowing water flow into or out of the specimen and to determine the pore pressure coefficient B. While applying the back pressure on the specimen at the top, the drainage valve was keeped to open that water can flow into the specimen. Avoiding undesirable pre-stressing of the specimen while applying back pressure, the pressures must be applied incrementally with adequate time between increments to permit equalization of pore-water pressure throughout the specimen.

The size of each increment may range from 35 kPa up to 140 kPa, depending on the magnitude of the desired effective consolidation stress, and the percent saturation of the specimen just prior to the addition of the increment. The difference between the cell pressure and the back pressure during the process should not exceed desire effective test pressure or 35kPa unless it is necessary to control swelling of the specimen during the procedure.

Specimens is considered to be saturated if the value of B is 0.95 or higher, or if B remains unchanged with addition of back pressure increments.

2.1.3. Consolidation

The consolidation stage shall can be as follows:

- Determining effective consolidation pressure due to the vertical effective stress (σ_v'). The range of effective consolidation pressure is over a range of effective stresses related to the vertical stress on the sample of soil in - situ such as $1/2\sigma_v'$, σ_v', $2\sigma_v'$.

- Increasing the chamber pressure until the difference between the chamber pressure and the back pressure equals the desired effective consolidation pressure.

- During the consolidation process, observing and recording the volume change readings at increasing intervals of elapsed time (0.1, 0.2, 0.5, 1, 2, 4, 8, 15, 30 min and at 1, 2, 4 and 8h, and so forth).

- Plotting the volume change Δv versus either the logarithm lgt or square root of elapsed time $\sqrt{t}$. The consolidation was allowed to continue for at least one log cycle of time or one overnight period after 100% primary consolidation. The plot can be used to also determine t_{50} or t_{90}.

2.1.4. Shear

The drained valves are opened before applying axial load to dissipate excess pore pressures throughout the specimen at failure. To determine loading rate, pore pressure is allowed to dissipate. The rate of strain (ε) (loading rate) determined from the following equations:

With side drain (1):

$$\varepsilon = \frac{4\%}{16t_{90}} \tag{1}$$

Without side drain (2):

$$\varepsilon = \frac{4\%}{10t_{90}} \tag{2}$$

4% - assume the axial strain at failure. If it is estimated that the failure will occur at a strain value other than 4%, a suitable strain rate may be determined using these equations by replacing 4% with the estimated failure strain; t_{90} - the time for 90% primary consolidation.

At a minimum, load, deformation and volume change values can be recorded at increments of 0.1% strain up to 1% strain. The loading continues to 15% strain, except the loading may be stopped when principle stress difference (deviator stress) has dropped 20% or when 5% additional axial strain occurs after a peak in principle stress difference (deviator stress).

2.1.5. Calculation

The deviator stress (σ_1-σ_3) versus the axial strain and the axial strain versus volume change is plotted; after that Mohr stress circles at failure is drawed. These circles can be used to determine effective shear strength (C', φ').

2.2. Materials and procedure

All sand samples were taken in the bore holes of project of Metroline Nhon - Ha Noi station (Figure 1). The sand samples belong to the Vinh Phuc formation. Sand soil distributed within the borehole at the depth changing from 10 to more than 30 metres. The main composition of Vinh Phuc formation in this place consists of grayish, yellowish, medium to dense state, sand mixed with some gravel. The N SPT value changes from 12 to 50. Some physical properties of sand disturbed samples were determined. Moisture content were test by ASTM D 2216, specific gravity were test by ASTM D 854. Minimum void ratio, e_{min}, was test by ASTM D 4253 and maximum void ratio, e_{min}, was test by ASTM D 4254. The particle size analyses were carried out by ASTM D422 and the classification of soil is according to the ASTM D 2487. All testing were conducted in the laboratories (LAS - XD 442, LAS - XD 928). The results are shown in Table 1. The classification of soil shown that soils are heterogeneous, composes of Poorly graded sand (SP), Well - graded sand (SW), Poorly graded sand

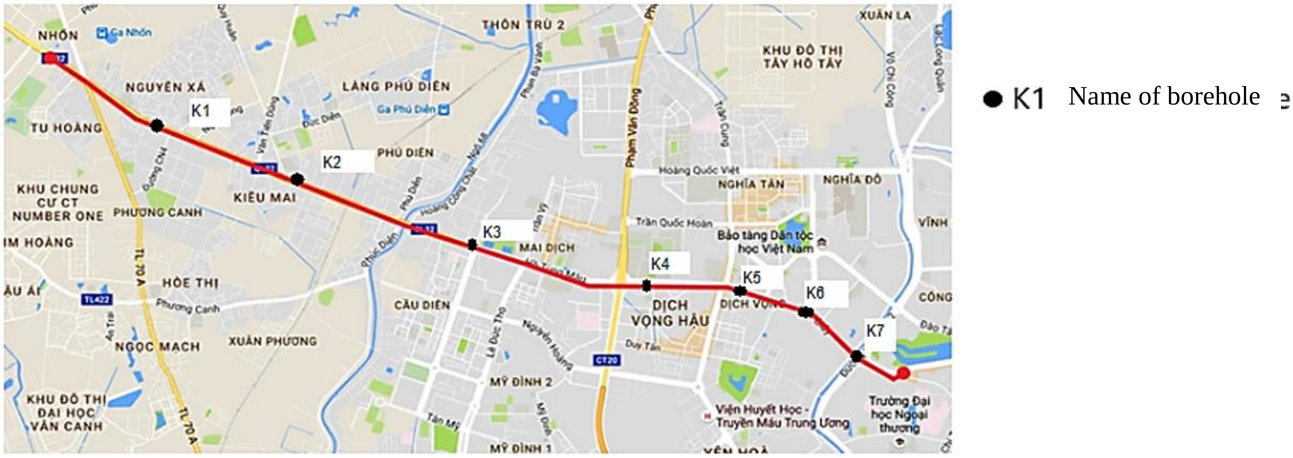

Figure 1. The location of sand samples.

Table 1. Sand sample for determing effective shear strength.

No	No. sample	Borehole	Depth of sample, *m*	*N* values	e_{max}	e_{min}	*W*,%	γ of reconstituted specimens, g/cm^3	Δ, g/cm^3	Soil classification due to ASTM D2487
1	M1	K1	19.7	28	0.851	0.557	14.0	1.81	2.65	SM
2	M2	K1	25.8	42	0.900	0.574	20.0	1.97	2.66	SP
3	M3	K2	15.8	27	1.017	0.521	16.2	1.81	2.66	SC-SM
4	M4	K2	25.4	20	0.866	0.577	20.9	1.86	2.67	SP
5	M5	K3	18.0	25	0.840	0.577	24.4	1.95	2.65	SP
6	M6	K3	25.5	28	0.873	0.593	21.8	1.91	2.66	SP
7	M7	K4	15.7	22	0.884	0.583	19.2	1.84	2.66	SC-SM
8	M8	K4	25.7	19	0.853	0.569	19.4	1.85	2.65	SC-SM
9	M9	K4	27.7	21	0.871	0.578	24.2	1.93	2.67	SP-SM
10	M10	K4	29.7	22	0.868	0.580	25.2	1.93	2.64	SP-SM
11	M11	K5	13.6	12	0.857	0.602	20.4	1.82	2.66	SC-SM
12	M12	K5	15.0	16	0.894	0.571	23.8	1.87	2.66	SC-SM
13	M13	K5	22.7	22	0.890	0.561	17.9	1.84	2.67	SM
14	M14	K5	26.7	27	0.896	0.568	19.7	1.88	2.67	SM
15	M15	K5	28.7	33	0.879	0.566	18.5	1.91	2.67	SP-SM
16	M16	K6	14.6	18	0.925	0.614	19.8	1.83	2.72	SC-SM
17	M17	K6	19.5	20	0.862	0.581	21.7	1.87	2.65	SC-SM
18	M18	K6	23.5	18	0.894	0.583	17.8	1.80	2.67	SC-SM
19	M19	K6	29.5	50	0.903	0.546	9.6	1.89	2.66	SW
20	M20	K7	13.6	15	0.926	0.588	27.5	1.92	2.66	SC-SM
21	M21	K7	19.6	32	0.900	0.602	14.6	1.83	2.67	SW- SM
22	M22	K7	21.6	21	0.893	0.571	16.6	1.80	2.66	SP
23	M23	K7	27.6	50	0.879	0.594	19.1	1.99	2.67	SC-SM
24	M24	K7	29.8	32	0.865	0.565	15.1	1.85	2.66	SP

with silt (SP-SM), Well graded sand with silt (SW-SM), Silty sand (SM) and Silty, clayey sand (SC-SM). The SP, SW, SP-SM, SW-SM soils are not contained clay content with less 12% fine. SM, SC-SM contain clay content with over 12% fine.

3. Results of shear strength of sand and discussion

Soil samples were determined for moisture content and prepared due to above progress. The sample has high of specimen *h*=7.8cm and 3.9cm in diameter. Each soil sample is made up of three tests of effective consolidation pressure (chamber pressure such as σ_1=200 kPa, σ_2=300 kPa, σ_3=500 kPa and effective consolidation pressure σ'_1=100 kPa, σ'_2 = 200 kPa, σ'_3=400 kPa). The specimen is saturated to achieve saturation of 90%. The sample is then consolidated to up to the rate of 90% consolidation. From the consolidation stage, the compression speed determines in the range of 0.002 to 0.006mm/min. The results of effective shear strength are shown in Table 2, Figure 2.

The shear resistance of sand is made up of sliding and rolling friction plus the resistance to volume change by interlocking. In drained condition, the effective stress of sand soil, effective cohesion (C') is small change from 0 to 6kPa, approximately 0, effective internal fiction angle (φ') change from 30^{0}26' to 38^{0}10'. For SP, SW, SP-SM, SW-SM soils, effective cohesion (C') is 0 kPa.

High values of φ' for SW, SW-SM sands (38^{0}10'- 38^{0}09'), result from effecting of uniformity of gradation, the grain shape and grain size.

The relationship between N of SPT value and effective internal fiction angle (φ') shown in Figure 3. It also shows that the relative density (N values) influenced the effective internal fiction angle. As the N of SPT value increases, the effective friction angle increased.

Table 2. The effective shear strength of sand.

No	No samples	Cell pressure, kPa			Effective consolidation stress, kPa			Effective shear strength	
		σ_1	σ_2	σ_3	σ'_1	σ'_2	σ'_3	C', kPa	φ, degree
1	M1	200	300	500	100	200	400	4	32^{0}19'
2	M2	200	300	500	100	200	400	0	36^{0}31'
3	M3	200	300	500	100	200	400	3	30^{0}26'
4	M4	200	300	500	100	200	400	0	31^{0}04'
5	M5	200	300	500	100	200	400	0	32^{0}11'
6	M6	200	300	500	100	200	400	0	34^{0}34'
7	M7	200	300	500	100	200	400	6	31^{0}13'
8	M8	200	300	500	100	200	400	3	30^{0}18'
9	M9	200	300	500	100	200	400	0	34^{0}23'
10	M10	200	300	500	100	200	400	0	35^{0}34'
11	M11	200	300	500	100	200	400	2	30^{0}07'
12	M12	200	300	500	100	200	400	1	30^{0}13'
13	M13	200	300	500	100	200	400	2	31^{0}09'
14	M14	200	300	500	100	200	400	1.4	32^{0}30'
15	M15	200	300	500	100	200	400	0	37^{0}11'
16	M16	200	300	500	100	200	400	6	31^{0}31'
17	M17	200	300	500	100	200	400	4	33^{0}35'
18	M18	200	300	500	100	200	400	1	31^{0}10'
19	M19	200	300	500	100	200	400	0	38^{0}10'
20	M20	200	300	500	100	200	400	4	30^{0}11'
21	M21	200	300	500	100	200	400	0	34^{0}09'
22	M22	200	300	500	100	200	400	0	33^{0}27'
23	M23	200	300	500	100	200	400	5	37^{0}53'
24	M24	200	300	500	100	200	400	0	36^{0}09'

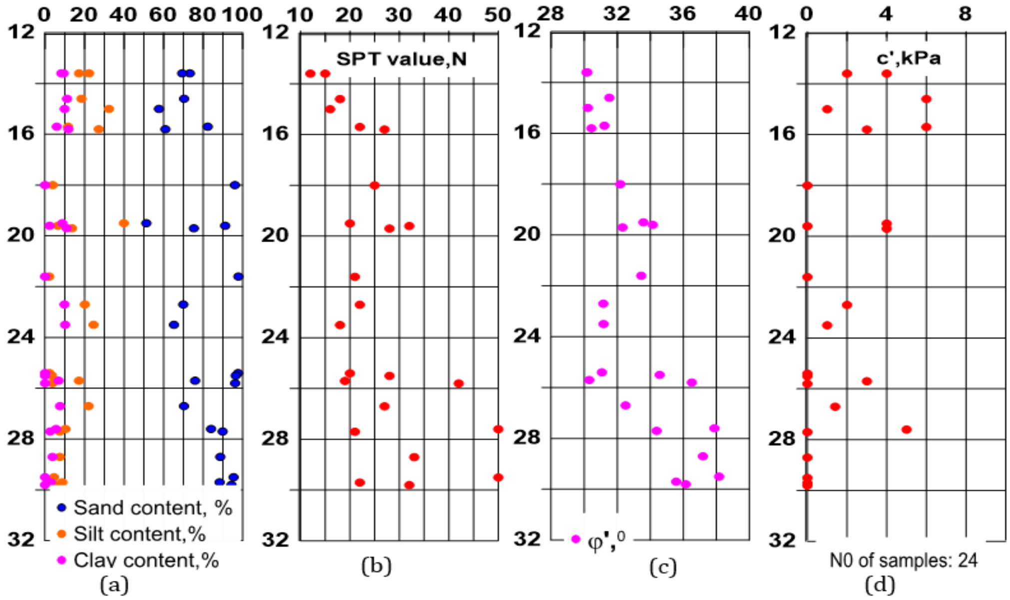

Figure 2. The particle size (a), SPT value (b) and shear strength of sand (c).

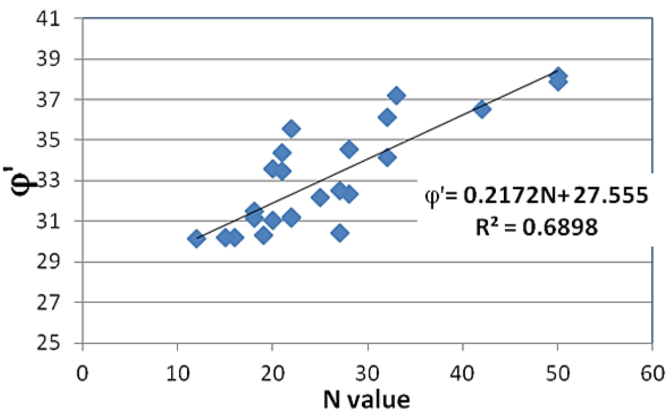

Figure 3. N value versus effective internal fiction angle (φ').

For SM, SC-SM soils, effective cohesion (*C'*) changes from 1 to 6 kPa. The effective cohesion was influent by clay content. The reason for these results is the clay content reflects the percentage of clay mineralogy and the quantity of clay-size particles (Terzaghi, Peck and Mesri, 1996).

4. Conclusion

Based on the research results, the following conclusions can be drawn:

The aQ_1^3vp sand in the underground railway line Nhon - Hanoi Railway Station belongs to Vinh Phuc Formation of Hanoi area with some specific mechanical characteristics.

The effective shear strength of sands, effective cohesion (*C'*) change from 1 to 6kPa, effective internal fiction angle (*φ'* change from 30⁰26' to 38⁰10'). As the clay content increases, so effective cohesion increases. As the increasing of relative density (*N* value), the same effective friction angle increases. The grain shape, grain size, uniformity of gradation is important factors influencing the value of *φ'*, so far the high values of *φ'* for SW, SW-SM sands. This is the important result provided the data for the underground designing.

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