Centrifuge experiments on stability of tunnel face in sandy ground

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ABSTRACT: This paper presents the results of centrifuge experiments on the stability of tunnel face in sandy ground. The model consisted of two main parts, an aluminium tube, representing the tunnel lining, and a thin pressurised rubber bag placed inside the aluminium tube to support the soil at the tunnel face. During the experiment, the pressure inside the rubber bag was reduced until the collapse of the face occurred. The main purpose of the experiment was to determine the minimum supporting pressure to the tunnel face and visual observation of the failure mode and the extent of the failed zone.

1 INTRODUCTION

A centrifuge apparatus was employed to model the behaviour of soil around the tunnel face supported by means of pressure. Miniature model of the tunnel was manufactured for this purpose allowing controlling the supporting pressure at the face. Due to the symmetry of the modelled problem, only half of the tunnel was manufactured. The model was placed against the window of the centrifuge container so that the soil movement and deformation around the tunnel face could be visually observed and recorded by CCD camera. The tunnel face was supported by means of pressures, compressed air was used in PT test series and water was used in DT series. The level of the supporting pressure was gradually decreased during the experiment, until the face collapsed. It was found that the collapse occurred at relatively low level of the supporting pressure and that the cover above the tunnel does not have a significant influence on the measured pressures at the face collapse for the C/D ratio varying from 2 to 6. In the case when the face was supported by the air pressure, the collapse occurred when the pressure dropped below 5.3 kPa, which was a surprising result. However, this is in acceptable agreement with similar experiments reported by Chambon and Corté in 1994. The centrifuge experiments conducted by Mashimo and Suzuki also indicate very stable behaviour of the tunnel face in granular soil.

2 MODEL GROUND

The model ground was prepared from dry Toyura silica sand being carefully rained from a hopper to the centrifuge container. The particle density of Toyura sand is 2.647 g/cm^3, typical gradation for $D_{50} = 0.19$ mm, $D_{30} = 0.16$ mm and $D_{10} = 0.14$ mm, and the angle of internal friction of 42 degrees.

The desired relative density of the model ground was 80%, however, the density actually achieved varied from 79% to 83%. The average density of the rained sand was 1.56 g/cm^3.

3 CENTRIFUGE MODEL AND TESTING PROGRAM

The size of the centrifuge container was 70 × 60 × 30 cm. The tunnel model was placed along the centrifuge window, allowing the visual observation of the ground movement. Digital images were taken during the experiment in order to construct the displacement vector. Supporting pressure was measured by a miniature pore pressure transducer (PPT) at the invert level of tunnel. The surface settlement was measured with a lased displacement transducer. The set up of the centrifuge container is schematically shown in Figure 1.

The designated acceleration field for the experiments was 80 G, in which the tunnel diameter was 4 m and the cover above the crown was 8, 16 and 24 m with the C/D = 2, 4 and 6, respectively.

The testing program is summarized in Table 1.

4 AIR PRESSURE CONTROLLED TESTS

The tunnel face was supported by a compressed air in this series of tests. Experiment setup was quite simple;

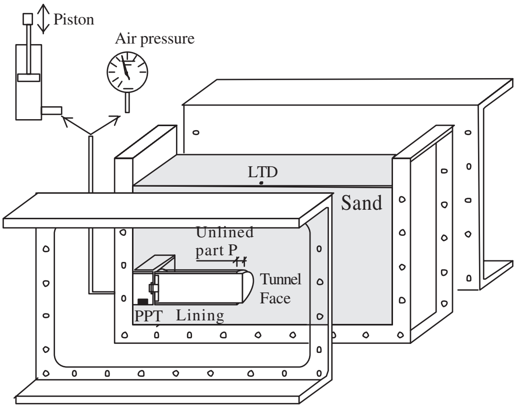

Figure 1. Experimental setup.

Table 1. Test program.

Test	C/D	*P (unlined length)*	Stabilizing medium
PT2	2	0.0	air
PT4	4	0.0	air
PT6	6	0.0	air
DT2	2	0.0	water
DT4	4	0.0	water
DT6	6	0.0	water
DT-P2	2	0.1D	water
DT4-P	4	0.1D	water
DT6-P	6	0.1D	water

Table 2. Supporting pressure, σ_s, at the failure for PT series.

C/D :	2	4	6
σ_S[kPa] :	3.6	2.6	5.3

the tunnel model was connected to the air compressor through a controlling valve. The air pressure was being reduced from its initial value (equal to "at rest" pressure at the tunnel centre line) in constant rate of 12 kPa per minute.

The measured magnitudes of the supporting pressure at which the collapse of the face occurred are summarized in Table 2. As it can be noticed, the face failed at very low values of pressure, ranging from 2.6 to 5.3 kPa. The difference in the obtained pressures for different cover is small. This suggests that the cover has a little influence on the collapse pressure for a moderately deep tunnel. It appears that the required supporting pressure to the face depends on the tunnel diameter.

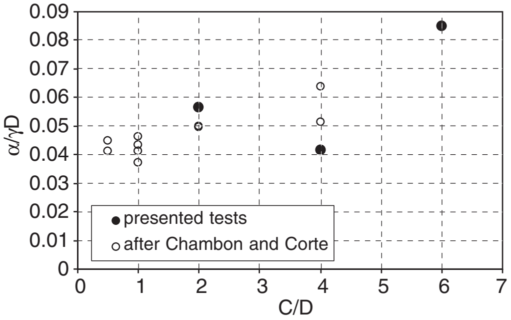

Figure 2. Normalized supporting pressure at failure against C/D ratio.

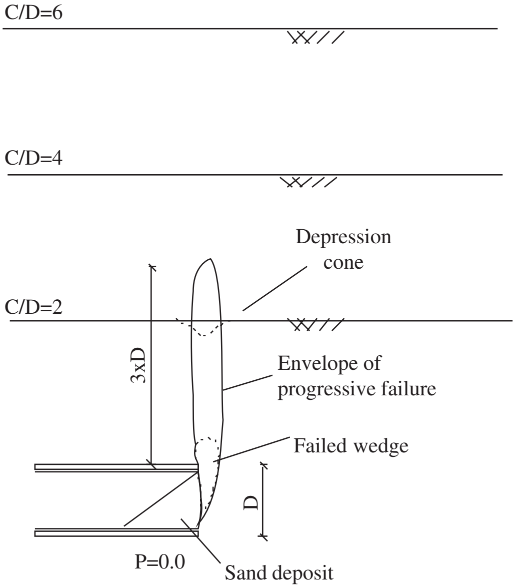

Figure 3. Sketch of the failure pattern in PT series.

The comparison between here presented tests and the experiments reported by Corte is made in Figure 2 in dimensionless form.

It was observed that the face failure occurred suddenly, without any noticeable movement of targets around the face prior to the collapse. The failure mechanism was initialized by loosening of the soil in front of the tunnel extending above the tunnel crown, as schematically indicated in Photo 2. Then the failure zone propagated progressively upward. It reached the surface in the C/D = 2 test, and in cases C/D = 4 and 6, the movement stabilized at the height approximately three times of the tunnel diameter above the tunnel crown. The development of the failure mode is shown in the sketch of Figure 3, and in Photo 1, showing

Test PT; Case C/D = 2

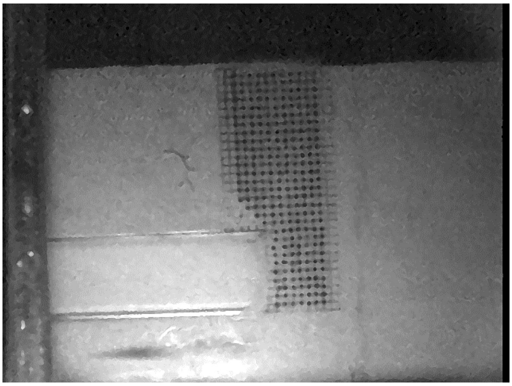

Photo 1. Condition before collapse Time 0.0 sec.

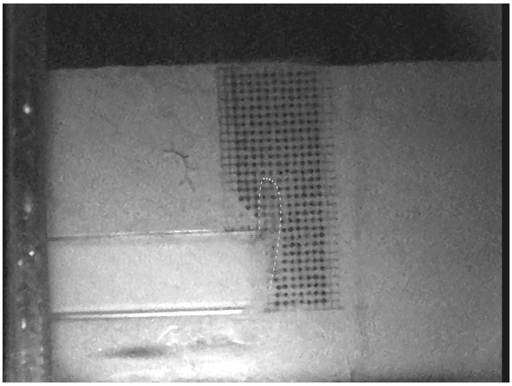

Photo 2. Loosening of sand ahead of the face – collapsing wedge Time + 1/30 sec.

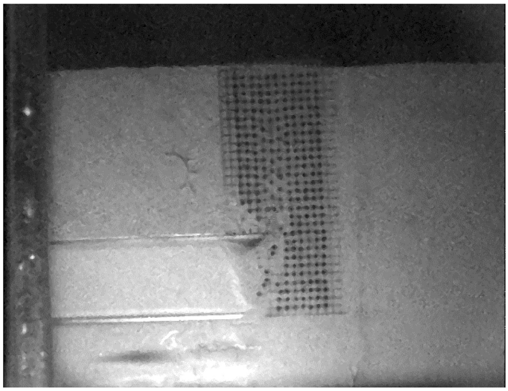

Photo 3. Progressive propagation of failure Time + 0.5 sec.

the face just before the collapse, Photo 2 showing the soil movement at the moment of face collapse and Photo 3 showing the propagation of the failure toward the surface.

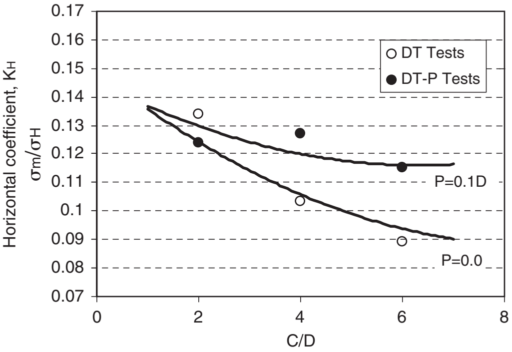

Figure 4. Horizontal earth pressure coefficient against the C/D ratio.

5 VOLUME DISPLACEMENT CONTROLLED TESTS

In the second series of tests, the water was used as the supporting medium. The rubber bag was connected to a cylinder with a piston located outside of the centrifuge container by a plastic pipe and the system was filled up with de-aerated water. During the experiment a constant movement of 3 mm per second was applied to the piston directed outward, so the water inside the rubber bag was being displaced into the cylinder and thus the supporting pressure at the face was reduced. The series of test is referred as the volume displacement controlled tests and denoted as DT. There were two configuration of this setup, a tunnel without unlined part, DT, and a tunnel with short unlined part of 0.1D denoted as DT-P.

The cylinder, plastic pipe-line and the rubber bag (inside the tunnel) created a closed hydraulic circuit. The pressure generated inside this system therefore consisted of pressure components due to the hydrostatic pressure in the high gravity field and the earth pressures acting at the tunnel face. This allowed the measurement of the pressure at which the forces at the tunnel face are in equilibrium and since the water was assumed to be perfectly incompressible, the situation will correspond to the at rest pressure situation in the retaining wall approach. Figure 4 shows obtained horizontal coefficient, K_H,

$$K_H = \frac{\sigma_m}{\sigma_V} \tag{1}$$

in which σ_m is the measured pressure inside the tunnel and σ_V is the theoretical vertical pressure at the PPT level. As Figure 4 indicates, the measured supporting pressure inside the tunnel was only about 10 per cent of the theoretical vertical pressure.

After this stage, the piston was droved outward and the supporting pressure to the tunnel face was reduced.

Table 3. Supporting pressure, σ_s, at the failure for DT series.

C/D :		2	4	6
DT	σ_S[kPa]	19.6	16.6	20.3
DT-P	σ_S[kPa]	17.8	18.4	17.7

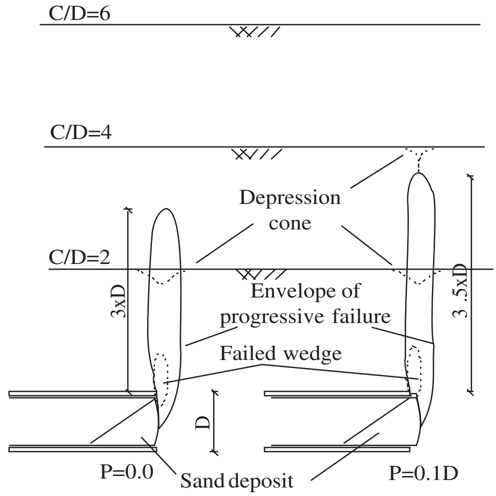

Figure 5. Sketch of the failure development in DT and DT-P series.

There was a sudden drop in the measured pressure the instant as the piston motion was activated. This drop in the measured pressure is attributed to high stiffness of the soil around the tunnel face at the early stage of unloading at very small strains. The supporting pressure was then gradually decreased as the piston moved until the face collapsed. Table 3 summaries the obtained values of supporting pressure at the moment of the face collapse. It varies from 16.6 to 20.3 kPa. Such pressure is equivalent to hydrostatic pressure of two meters high water column. The tunnel diameter was 4 m and fully flooded by the water. It is believed that some negative pressure had been generated during the piston movement and the pressure distribution acting on the face is, therefore, not known exactly.

The failure mode of the face supported with water followed different pattern. The collapse of the face did not occurred suddenly, as it was observed in previous series of tests, the soil movement started around and above the tunnel crown, followed by an abrupt slight of a wedge extending above the crown to a distance of half of diameter. With the farther decrease in the supporting pressure, the soil was sliding into the tunnel in irregular instants, progressively moving upward. The soil movement was stabilized at the distance of about 3.5D above the crown. The development and propagation of the failure is schematically shown in sketch of Figure 5.

6 CONCLUDING REMARKS

It was observed that the face collapse if the supporting pressure of the compressed air drops below 5 kPa. In the case of the face being supported with water, the collapse occurred at pressure about 20 kPa measured at the tunnel invert. The actual distribution of the pressure, however, is not known exactly since unknown negative pressure was generated as the piston movement initialized. The results indicate that the stability of tunnel face of a moderately deep tunnel in granular soil can be maintained by relatively low supporting pressure. This can be taken into account when designing the minimum pressure in the betonite TBM or temporary support to tunnel face in conventional tunnelling methods.

It was assumed that the water used as the support to the tunnel face was perfectly incompressible and that there was a perfect contact between the rubber bag and the sand. Therefore, when the equilibrium was established in the high gravity field, the measured pressure inside the tunnel was the reaction to the earth pressure. It was found out that the reaction was 9 to 12% of the theoretical vertical pressure at the sensor location.

Once the collapse of the face happened, the failure zone was progressively extended upward. The movement stabilized at distance about 3 to 3.5 time of the tunnel diameter above the crown.

ACKNOWLEDGMENT

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