The Investigation of Stability of Tunnels and Settlements with Centrifuge Modeling

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ABSTRACT

In most of the larger cities underground transportation systems have been getting desired. Such systems are constructed in urban areas and involve a tunnel, especially in soft ground and in shallow zones.

One of major concerns for tunneling operations in urban area is the effect on neighboring buildings, because the tunneling operation and near structures highly interact each other. Whatever the used construction method is, the excavation of a tunnel causes displacement around the opening and may expand towards the ground surface. The dislocations of the buildings interact with the ground movement, and the rigidity of existing structures will promote reduction of the magnitude of displacements induced by tunneling.

In this investigation, to determine displacements induced by tunneling, the centrifuge modeling was used. The small scale centrifuge model provided dependable information about the face collapse of a shallow tunnel. A required support pressure for shield driven tunnels in soft materials, and the ground deformations along the longitudinal section of the tunnel model, can be identified by simulating a loss of tunnel face stability.

1. INTRODUCTION

PECK (1969) presented a first state-of-the-art report based on many studies, stating three important requirements to construct a sufficient tunnel. The first one is about stability, because in order to build the tunnel safely the construction method used must be selected with paying attention especially to stability of the tunnel face, before placing the tunnel lining. Secondly, excavation and construction of the tunnel should not cause any ground displacements which may lead unwanted damages to neighboring structures, utilities, and roadways. Thirdly, during the design lifetime of the tunnel the lining should be serviceable in the case of exposing any subsequent influence.

Many researchers have been conducted regarding ground displacements related to tunneling in clay. Some of the initial centrifuge tests on this subject were performed by MAIR (1979), who worked on centrifuge modeling research to examine collapse of tunnels in soft clay.

Up to date, research on the centrifuge modeling of tunnels in sandy soils has been limited. The initial centrifuge studies about the relationship between face pressure and face stability was conducted by CHAMBON and CORTÉ (1989, 1991, and 1994). They performed centrifuge tests on tunnel models in dry sand. Examination of the pressures at which face stability was lost and observation of the post-instability ground deformations related to tunnel failures at various depths was also investigated by them. In order to examine the face stability of tunnels in sand and offer charts for evaluating the required face support pressure, LÉCA and DORMIEUX (1990) applied limit analysis techniques. Analysis of safety against both collapse and blow-out were performed. According to these upper and lower bound solutions a range of pressures for which tunnel face instability might occur, were predicted.

Face stability conditions in cohesionless soil under drained conditions, on slurry shield and earth-pressure-balanced(EPB) shield driven tunnels were examined by ANAGNOSTOU and KOVÁRI (1994, 1996). Recently, this two machine tunneling methods have been successfully used throughout the world.

The major purpose of this research is investigating the ground movements which take place due to tunnel face stability, and which depend on the different soil grain size, different line thickness and weather there is a structure on surface or not. Tunneling effects on deformations and surface settlements is another discussion subject. As is known to all, provided the deformations extend beyond the highest values, extend of the ground movements reaches such a size that neighboring buildings may be highly damaged. The experiment subjected to this thesis was conducted at Geotechnical laboratory of Bodunkultur University, Vienna.

2. MATERIAL

For the experiments, two different types of sand are used in dry form. The first ground (S1) is coarse grained silica sand which is produced according to DIN 1164/58 norm sand II rules which is named as the Norman Sand. The ground is used in a dry and loose form in the experiments. Because the ground has a uniform grain diameter distribution, it is rarely observed in natural ground conditions.

Table 1 Parameters of Soil S1

Specific weight ρ s [g/ cm^3]	2,65	Coefficient of Uniformity C_u	1,4
Density range ρ_{min}, ρ_{max} [g/cm^3]	1,44 - 1,65	Coefficient of Curvature C_c	1,03
Void Ratio e_{min}, e_{max}	0,607– 0,844	Friction angle φ [°]	34
Relative Density (%)	32	Cohesion c [kN/ m^2]	0

2nd ground (S2) is the mixture of fine sands with different grain diameters. This ground is also used in a dry and loose form in order to make a comparison.

Table 2 Parameters of Soil S2

Specific weight ρ s [g/ cm^3]	2,65	Coefficient of Uniformity C_u	3,25
Density range ρ_{min}, ρ_{max} [g/cm^3]	1,47 - 1,62	Coefficient of Curvature C_c	1,94

Void Ratio e_{min}, e_{max}	0,640 – 0,804	Friction angle φ [°]	35
Relative Density (%)	15	Cohesion c [kN/ m^2]	0

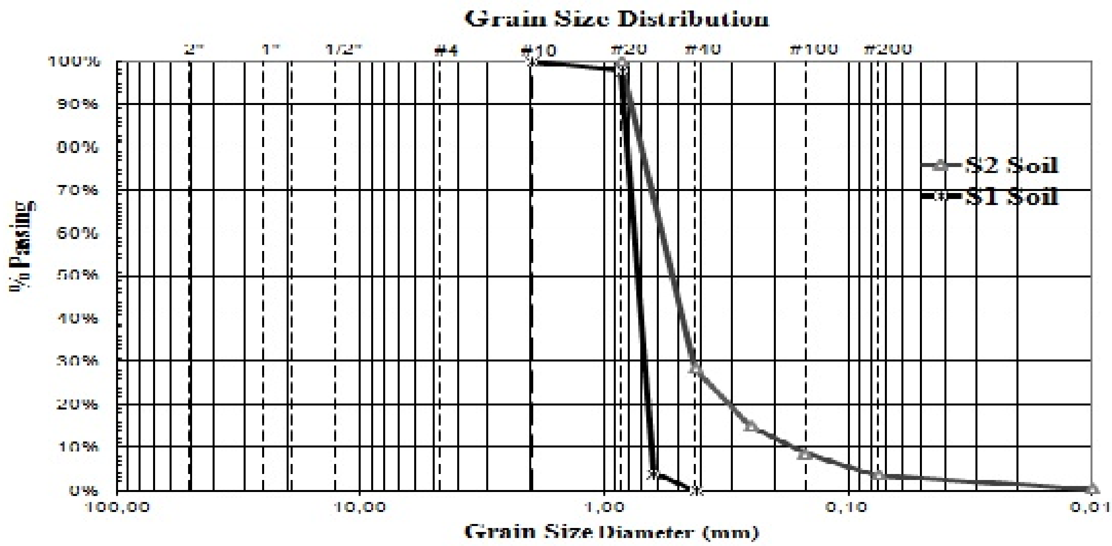


Figure 1 Grain size distribution curves of S1 and S2

3. GEOTECHNICAL CENTRIFUGES

Beam Centrifuge was manufactured by Trio-Tech, CA in 1989. It has been used in numerous research projects about e.g. earth pressure and foundation problems since its installation. The beam centrifuge, Model 1231 Standard Heavy Duty, has a diameter of 3.0m, a load capacity of 10.000G-kg, 56 slip rings and the driving force is supplied by a 15HP DC motor.

In the motor a symmetrical high strength aluminium beam is rotated. Swing platforms are placed at both ends of this rotating arm. It is mounted the model box on one of the platforms, and to provide symmetry an equal counterweight is located on the opposite side.

All the tests are performed distant from the control room. The centrifuge is managed using a control console. The angular velocity is supplied either manual or, for an exact acceleration value, by a computer-programmed remote signal in revolutions per minute (RPM) to reach the desired acceleration.

A compact video camera and a light source (low-voltage halogen spot light) are mounted near the centrifuge axis to observe the behavior of the tested specimen, directed to the swinging basket with the model box placed in centrifuge. The video signal is sent through the slip rings to a monochrome display residing in the control room. Accordingly as well as the upswing angle of the swinging basket, unexpected effects can also be controlled.

4. DESIGN OF THE MODEL

The model box was designed for Dipl. Eng.Gregor Idinger's thesis. The whole mechanism with every detail was designed by him, with the aim to build a test set-up adequate for the -PIV technology.

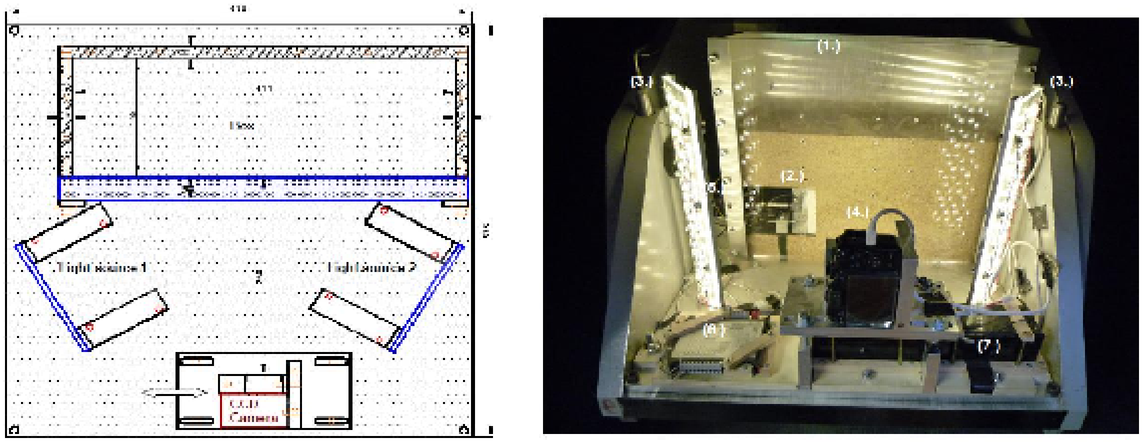


Figure 2 a) Sketch of PIV-model assembly, top view (Idinger, 2010) b) Model box mounted on swing basket before the start of test (C/D=1.0), 1. Model box, 2. tunnel, 3. LED lights, 4. camera, 5. engine, 6. engine driver, 7. batteries

The mechanism cutting vertically through the tunnel axis which can be seen on pervious pictures was modeling the problem in half to gain the soil deformations in the longitudinal axis. Because the greatest deformations occur in this section, analyzing the stability at this level is a particular issue of concern. It should be noticed that surface of the less structured perspex reduces the friction and accordingly effects the path of the grains.

To reduce support pressure a rigid piston is used. The piston was formed of an aluminium tunnel face, a linear actuator which is for carrying out the displacement and a load cell for measuring the acting pressure. In order to connect the actuator and the load cell a steel rod with windings on both sides was installed completing the piston axis. The diameter of the face plate was preferred to be smaller than the inner diameter of the shell to attain a friction reduction in the displacement.

The effective earth pressure acting on the tunnel face is measured with the load cell operating behind the semi-circular tunnel face. In order to control the piston displacement a displacement transducer is mounted.

5. EXPERIMENT SET UP

If an acceleration of N times Earth Gravity (g) is put on a material with density ρ, in the model, the vertical stress σ_v at depth h_m (subscript m indicates the model) is obtained by the following equation (1) :

$$\sigma_{vm}= \rho Ng \tag{1}$$

In the prototype, (subscript p indicates the prototype) then;

$$\sigma_{vp}= \rho g \tag{2}$$

Hence for $\sigma_{vm} = \sigma_{vp}$, then

$$h_m = h_p N^{-1} \quad (3)$$

and for linear dimensions the scale factor (model: prototype) is 1:N. Because the model represents a linear scale of the prototype, scale factor for displacements will also be 1:N. Therefore the scale factor of that strains is1:1 and so the part of the soil stress-strain curve mobilized in the model will be the same as that of the prototype.

Table 4 Scaling factors for centrifuge modeling (FERSTL, 1998)

Physical Value or Event	Dimensioning Prototype	Dimension in Centrifuge Model at (N*G)
Gravity	1	N
Length	1	1/N
Displacement	1	1/N
Area	1	$1/N^2$
Volume	1	$1/N^3$
Strain	1	1
Mass	1	$1/N^3$
Density	1	1

In this research, The Scaled factor was determined as 75 so all experiment exposed 75 times bigger earth gravity.

Diameter of the tunnel (D) is modeled as 10 cm with helping of the small scale modeling. Except the 6th and 7th experiments, the strat thickness (C) is modeled as 5 cm. Thus the overburden ratio (C/D) determined as 0.5. At the 6th and 7th experiments, the strat thickness (C) is modeled as 10 cm. Because, for ground surface settlement, the distance between the tunnel face and surface of the soil, is critical. When the distance is designated with C and the tunnel diameter is designated with D, whether the surface will be affected from the settlement is obtained by the rate of these parameters. If the C/D ratio is equal to or smaller than 0.5, the surface settlements take place, on the contrary if C/D is higher than 0.5, There is a possibility about surface settlements can-not reach to the ground surface.

At the 5th and 7th experiments, the geotextille used to show affects of soil improvement techniques to face stability of tunnel.

In the experiments, the soil samples are used under loose and dry conditions. During at the 2th and 4thexperiments the surcharge load is used as 2 kg.

6. RESULTS

-Surface settlements

Assessment of the experiment results,

In the case when Norman Sand (S1 soil) is used without any surcharge load, the maximum surface settlements 5 mm. The width of the surface settlement is approximately 55 mm. However, in a representative manner, in the second experiment where a 5-storey building is located on the surface ground, the surface settlement has reached to 5.5 m and the width of the surface ground where the settlement occurred has reached to approximately 60 mm. Depending on the surcharge load, it is obviously seen that the settlements have increased.

In the 3rd and 4th experiments, S2 soil, fine grained sand, is used. In case when no surcharge load is present, the maximum surface settlement is 7.3 mm the width of the surface settlement is approximately 65 mm. When the surcharge load is added, surface settlement has reached to 10.2 m and the width of the surface ground, has reached to approximately 75 mm where the settlement occurred.

As obviously seen from these results, the effect of the surcharge load on the settlements is considerably high and in case the surcharge loads increase, it is clear that these values will also increase more. However different surfaces have a well effect on the surface settlements. Although the surfaces used are sand, despite the maximum settlement in Normal Sand (S1) is limited with 5 mm without any surcharge load, maximum settlement in the other surface (S2) has increased to 7.3 mm, an increase in the surface where the settlements are developed is observed (Figure 3 a). With the increment of the surcharge load, the maximum settlement in Norman Sand (S1) is measured as 5. 5 mm and maximum settlement in the other surface (S2) has reached 10.2 mm, also there is an increase on the surface where the settlements are formed (Figure 3 b).

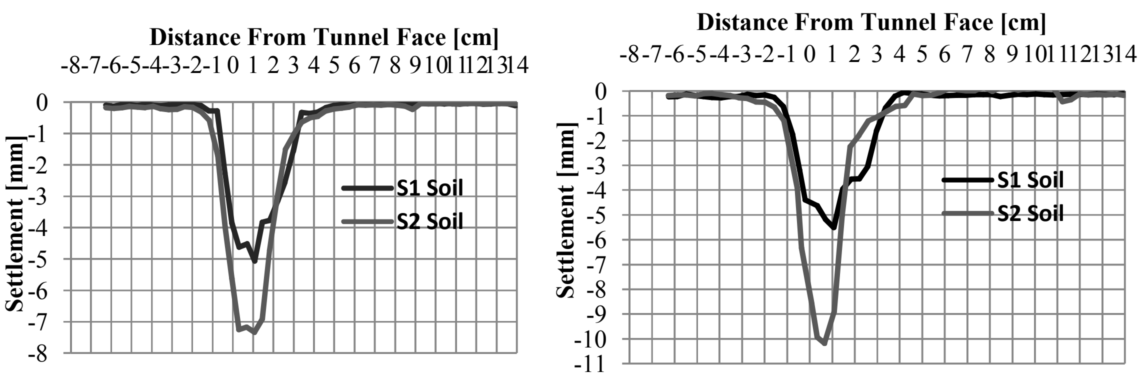


Figure 3 C/D=0.5 a) surface settlement after total face displacement ds=5mm; with S1 Soil max. settlement: 5.0 mm; with S2 Soil max. settlement: 7.3 mm b) (WITH SURCHARGE AFFECT) surface settlement after total face displacement ds=5mm; with S1 Soil max. settlement: 5.5 mm; with S2 Soil max. settlement: 10.2 mm

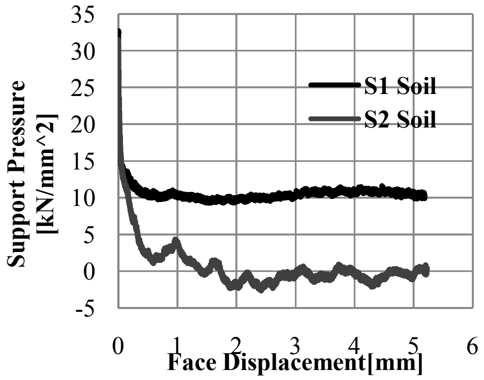
-Surface Pressure

In the experiments the face pressure was measured behind the piston with a load cell as compressive force. This was divided by the semi-circled area of the aluminum face to gain a mean face pressure.

The under 75G arising friction of the moving piston was evaluated. Therefore a constant correction value was added to the originally measured compression data.

In the 1st experiment (no surcharge load, Norman sand, C/D=0.5), maximum support pressure formed during the experiment is measured as 32.5 kN/m^2 and minimum support pressure is measured as 10 kN/m^2. In the 3rd experiment (no surcharge load, fine sand, C/D=0.5); the maximum support pressure formed during the experiment is measured as 31.5 kN/m^2 and the minimum support pressure is measured as 0 kN/m^2 (Figure 4 a).

In the 2nd experiment (surcharge load present, Norman sand, C/D=0.5); the developed maximum support pressure is measured as 45 kN/m^2 and minimum support pressure is measured as 5 kN/m^2. In the 4th experiment (surcharge load is present, fine sand, C/D=0.5), the developed maximum support pressure is measured as 62 kN/m^2 and the minimum support pressure is measured as 0 kN/m^2 (Figure 4 b). With the addition of the surcharge load; an increase in the maximum support pressure and a decrease in the minimum support pressure are observed.



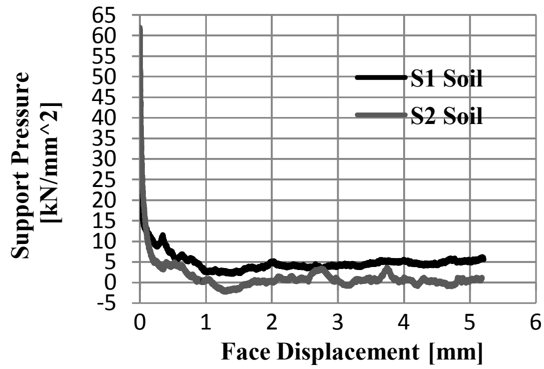


Figure 4 C/D=0.5 a) support pressure over face displacement; five millimeters; mean pressure after failure: with S1 soil pf=10.0kN/m²;with S2 soil pf=0.0kN/m² b) (WITH SURCHARGE AFFECT) support pressure over face displacement; five millimeters; mean pressure after failure: with S1 soil pf=5.0kN/m²;with S2 soil pf=0.0kN/m²

7. CONCLUSIONS

The small scale centrifuge model, which is newly designed, provided dependable information about the face collapse of a shallow tunnel. A required support pressure for shield driven tunnels in soft materials, and the ground deformations along the longitudinal section of the tunnel model, can be identified by simulating a loss of tunnel face stability. When the results are interpreted depending on the parameters,

For the surface settlements; the below mentioned matters are concluded:

1) In case there is an extra structure on the ground surface (extra load), the settlements increase depending on the load,
2) In case there are different soils, the change in the surface settlements can get higher values than existing of the surcharge loads,

For the tunnel support pressure; the below mentioned matters are concluded:

1) In case there is an extra structure on the ground surface (extra load), an increase is formed in the maximum support pressure,
2) In case two different sand samples are used; major differences are seen between the surface pressures.

Engineering practice in real world, however, tunneling through dry, cohesionless sand is quite uncommon. Mostly, at sites with coarse-grained soils, parts of the tunnel length can be excavated and constructed within the vadose zone above the groundwater table, where the coarse-grained soil involves sufficient moisture to generate some amount of visible cohesion. This generalization applies especially for urban areas under which shallow tunnels are possibly to be built. But, in spite of this fact, no physical modeling data come into existence to explain the developing of ground deformations with loss of tunnel face pressure in unsaturated sands.

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