# Stress Distribution and Deformations around Tunnels – A Study Based on Discrete and Finite Element Methods

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**Abstract**

*In recent times due to rapid urbanization along with the significance of sustainable building practices there is an exponential increase in the use of underground space. This emphasizes the need for the proper development for underground facilities. Tunneling is one of the most widely used methods for the development of underground structures. When tunnels are constructed near existing structures or on shallow soft ground there is possibility of very large deformations of the ground. These deformations may in turn cause failure of the tunnel as well as adjoining structures. In addition, the determination of stress levels associated with the yield of wall due to excavation is important. Since the ground behaviour is not properly understood, it is necessary to estimate the appropriate support and reinforcement and their response through numerical modeling. For numerical modeling finite or discrete element methods can be adopted. Finite element method assumes that the rock mass is a continuous medium. Generally rock masses contain discontinuities which can be taken into account by discrete element method. Discrete element method represents rock mass as an assembly of particles joined together by breakable bonds. In order to understand the ground response, the variation of the radial stress at the tunnel perimeter and the corresponding displacement is defined. Along with the stress and displacement variation, the failure pattern around the tunnel is also studied using both FEM and DEM and the results are presented.*

## Introduction

The stability of tunnel is one of the most important subjects in tunnel constructions and it depends on the surrounding material. The evaluation of the mechanical behaviour of the ground is very significant in the case of urban tunneling. In urban tunneling, the major concern is the control of ground stability and deformations since this results in surface settlement. Numerical modeling has become one of the most powerful tools in understanding the response of underground tunnels. The excavation of the underground structure in spite of its depth disturbs the soil and rock masses. This results in the settlement of the ground surface which affects the safety of the existing buildings. Due to several practical reasons and site conditions, tunnels are excavated on soft to hard soils/ weak rocks at shallow depths. In order to understand the stability of tunnels constructed in a soft ground, simulations are done on an unlined tunnel and tunnel with lining. The selection of continuum or discrete methods for analysis depends on many factors like the scale of the problem and the amount of discontinuities/fracture in the considered assembly. In this study both discrete element and finite element methods are adopted to understand the effect of tunneling on the considered ground.

## 3-D Discrete element modeling

The inherent discrete nature of the soil/rock particles results in transferring of forces through the contacts between the particles. Hence the failure and deformation modes cannot be properly analyzed by continuum approaches. Hence to analyze the rock mechanics problems Cundall (1971)

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pioneered the discrete element method (DEM). This is an explicit finite difference method wherein the interaction of the particles is monitored contact by contact and the motion of the particles modeled particle by particle. DEM is following the alternate application of a force–displacement law at the contacts and Newton's second law to the particles. Using Newton's second law, the translational and rotational motion of each particle is calculated whereas the force–displacement law is used to update the contact forces arising from the relative motion at each contact. In this paper, the discrete nature of the granular materials is modeled using the three dimensional particle flow code (2008) which is based on DEM.

## Sample generation

In discrete element method the assembly is modeled as a conglomeration of particles which are discrete in nature and interact only through the contacts. One of the major advantages of this modeling is that the discontinuous nature of the medium or the presence of joints/ fissures can be easily accounted for. In order to account for the strength of the cementing material which binds together the particles in the case of weak rocks, contact bonds are employed. This idea is based on the bonded particle model suggested by Potyondy & Cundall (2004) wherein the rock mass is represented as a dense packing of circular or spherical particles which are bonded together at their contact points. These contact bonds are breakable in nature and the macro behaviour of the rock mass as a whole depends on the strength of these bonds. Since in this study weak weathered rocks are simulated the contact bond strength used is very less.

## Selection of parameters for the study

The proper selection of material properties is one of the most important requirements for numerical simulations. Based on a series of axisymmetric triaxial tests conducted on a cylindrical assembly whose height to diameter ratio is 2:1, material properties were determined (Anitha Kumari and Santharam, 2010). The spherical particles which are bonded together by contact bond to represent the cemented rock mass. The strength of the contact bond is significant as the overall strength of the material depends upon it. Higher the contact bond strength, higher will be the compressive strength. But in all the cases there is a linear variation in uniaxial compressive strength with contact bond strength. In this study contact bond strength of 20kN is taken such that the sample is having a uniaxial compressive strength of nearly 2MN. The variation of Young's modulus at lower values of normal stiffness is almost insignificant. But at higher stiffness there is a steep increase in the modulus value at lower contact bond strengths (Anitha Kumari and Santharam, 2010). At higher contact bond strengths, however the modulus value remains more or less the same. Hence in order to represent a sample of uniaxial compressive strength of 2MN, contact bond strength of 20kN and particle stiffness of 100MN/m is adopted in this study. Also studies (Anitha Kumari and Santharam, 2010) have indicated that the angle of internal friction at various contact bond strengths is almost the same with very little fluctuations. Hence corresponding to a contact bond strength of 20kN, a value of 0.45 is taken as inter particle friction.

## Modeling the Assembly

The assembly is modeled in a section of weathered rock having a dimension of 25m x 10m x 25m and is shown in Fig.1. A total of 50000 particles are used for the simulation of the assembly. The properties of the particles used for the simulation are presented in Table 1. The particles are allowed to settle down under gravity. After applying the gravitational force, the system is subjected to a confining stress of 0.8MPa corresponding to the overburden pressure. This is to develop an isotropic loading condition within the model rock. Contact bonds are installed within the sample to represent the rock mass. Following this, a tunnel of radius 3m is excavated in the soil mass at a depth of 18m from the ground surface. The effect of the excavation of the tunnel is studied with respect to the contact force and circumferential stress variation. The variation is studied for two different cases (a) the tunnel is left unlined/without any support and (b) tunnel is lined. The distributions of contact force before and after the excavation

**Table 1:** Properties of the particles used

| Property | Value |
|---|---|
| Normal stiffness of particles | 100 MN/m |
| Shear stiffness of particles | 100 MN/m |
| Wall stiffness | 1000 MN/m |
| Density of particles | 2600 kg/m$^3$ |
| Normal Contact bond strength | 20 kN |
| Shear Contact bond strength | 20 kN |
| Porosity | 0.4 |
| No of particles | 50000 |
| Inter particle friction | 0.45 |
| Particle size | 0.075-0.1m |

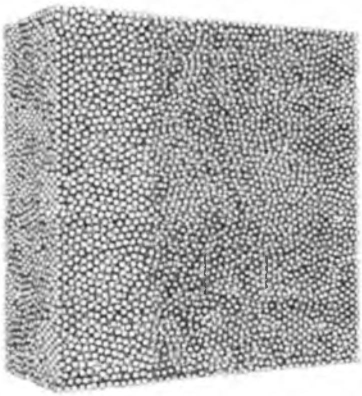
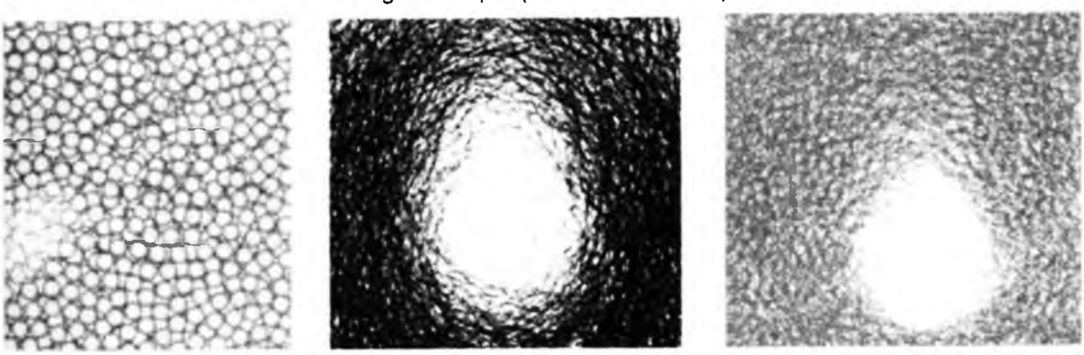
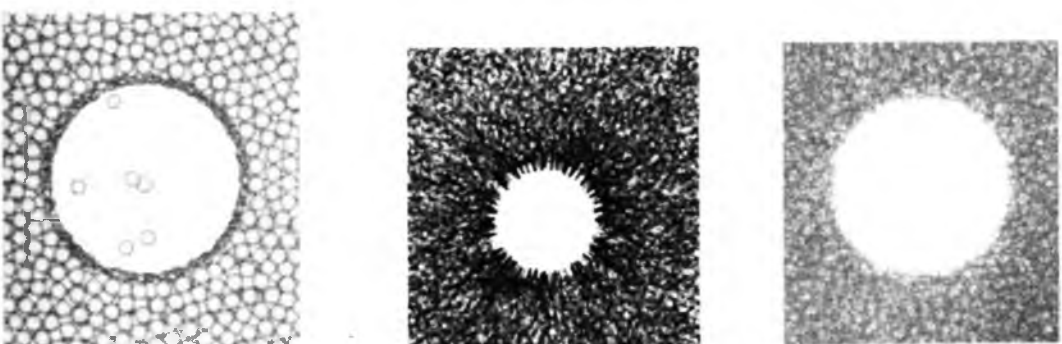


Fig 1: Sample (25m x 10m x 25m)



(a) Unlined tunnel (b) Contact force distribution (c) Contacts distribution

Fig. 2. Stability of Unlined tunnel (Enlarged view near the tunnel)



(a) Lined Tunnel (b) Contact force distribution (c) Contacts distribution

Fig. 3. Stability of Lined tunnel (Enlarged view near the tunnel)

are shown in figure 2. The close up view in Fig. 2(a) indicates that the unlined tunnel has completely collapsed. It can be clearly observed that there is a significant reduction in the contact force once the tunnel is excavated leading to the ultimate failure of the tunnel. But if the tunnel is lined as in Fig. 3, it can be seen that even though there is a reduction in the overall contact force, the system is much stable when compared to that of an unlined tunnel.

## Distribution of vertical stresses and vertical strains around the tunnel

In order to measure the various stresses developed in the assembly, a number of measurement spheres were used. Measurement spheres help to measure various parameters over a specified spherical volume such that it represents a large number of spheres. Circumferential stress around the tunnel was calculated at different inclinations. The circumferential stress distribution shown in figure 4 indicates that the values are higher for the structure with lining. Moreover it can be seen that the stresses are almost uniformly distributed around the tunnel. This can be attributed to the fact that the presence of lining results in a confinement of the area surrounding it. Due to this, the particle movements are restricted. The vertical stress distribution (Fig. 5) of an unlined tunnel indicates that the stresses are very low near the tunnel. However the progress of failure in the case of unlined tunnel is almost same in all the direction. This can be seen from figure 2 where the tunnel collapses from all sides, more being from the top. It is noted that in the case of unlined tunnels, for a small height

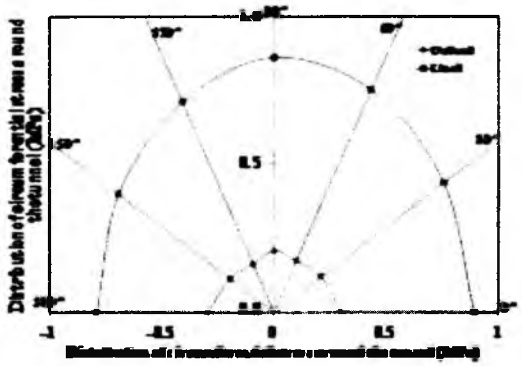



Fig. 4. Circumferential stress distribution

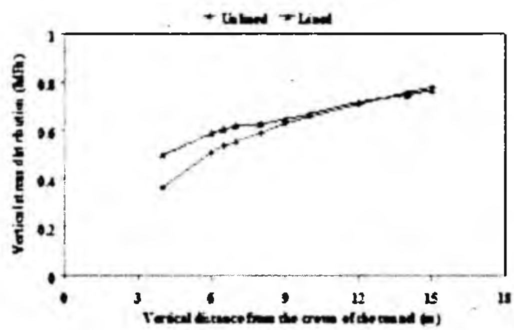



Fig. 5. Vertical stress distribution

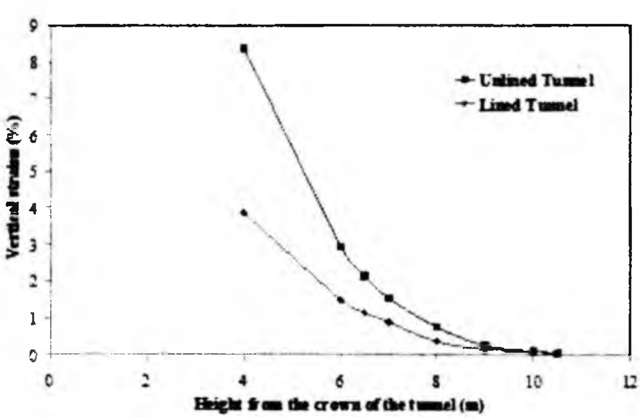



Fig. 6. Vertical strains Distribution

above the crown of the tunnel there is a sharp decrease in vertical stress, but then it is almost constant. Figure 6 indicates the distribution of vertical strains from the crown of the tunnel.

## Finite element modeling

Finite element method can be applied to soil/ rock mechanics problems irrespective of the complexities in geometry, loading or material models and are widely used in rock mechanics applications. In finite element method, the assembly is modeled as a continuum consisting of elements which are connected at discrete points called nodes. In 2-dimensional analysis, the assembly is discretized into triangle or quadrilateral shapes and all the forces are transmitted through nodes. The analysis of the problem is basically done in terms of these nodal forces and nodal displacements.

## Assembly generation

The commercial software PLAXIS is used for the analysis of this problem. It is assumed that the material is isotropic and homogeneous. The geometry of the problem and gravity loading has to be defined. The assembly is modeled with a width of 25m and depth of 32m. A tunnel of radius 3m is excavated at a depth of 18m from the ground surface. Once the geometry is defined, it is divided into a number of 15-noded triangular elements. The discretized geometry of the modeled assembly is shown in Fig. 7. Once the meshes are formed, the material properties as well as the constitutive model has to be specified. The constitutive model is important since it controls the response of the material to a loading condition. In this study, Mohr-Coulomb elasto-plastic model has been used for the weak weathered rock. Finally the boundary/initial conditions for the modeled assembly are given. No explicit boundary condition is specified for the top boundary where a natural condition of zero force and free displacement exists. The bottom boundary is fixed full which means both vertical and horizontal displacements are restricted. Table 2 gives the property of the soil material used in this study. Gravity loading is applied to the assembly as initial conditions following the K0 procedure since a horizontal surface is assumed. After applying the initial stress conditions, loading phase is initiated through automatic load stepping.

**Table 2:** Properties used for the weak rock material

| | |
|---|---|
| Model | Mohr-Coulomb |
| Behaviour | Drained |
| Unit weight | 26.5 kN/m$^3$ |
| Young's modulus | 5e6 kN/m$^2$ |
| Poisson's ratio | 0.2 |
| Cohesion | 20 kN/ m$^2$ |
| Angle of internal friction | 24 |

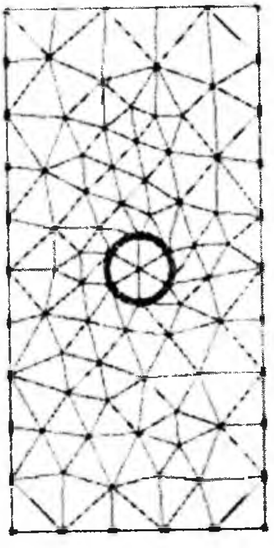


Fig. 7. Discretised assembly

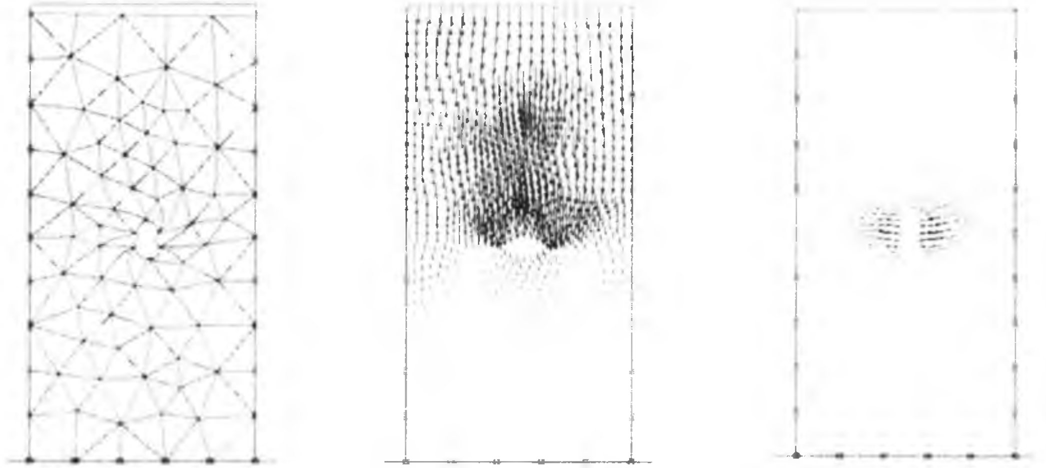
## Distribution of stresses and displacements in unlined and lined tunnels

Figure 8 shows the deformed mesh, vertical displacement and horizontal displacement respectively of the unlined tunnel. Similar to the discrete element simulations, the deformations are very high near the crown of the tunnel indicating a complete collapse of the system. Figure 9 shows the stress contours along the section. The stress contours clearly indicate the variation in stresses due to the excavation. The complete collapse of the assembly is clearly visible from the stress contours as well as the displacement diagrams.

The properties of the lining used are presented in table 3. Figure 10 shows the deformed mesh, vertical displacement and horizontal displacement respectively of the lined tunnel. From the displacement contours and deformed shape it is evident that the presence of lining has considerably increased the stability by decreasing the displacements which is also seen from the discrete element simulations. The stress contours of the lined tunnel are presented in Fig. 11.

Table 3: Properties of the lining used

| | |
|---|---|
| Behaviour | Elastic |
| Stiffness , EA | 1.4e7 kN/m |
| Flexural Rigidity, EI | 1.43e5 kNm |
| Thickness | 0.35 m |
| Weight /m length | 8.4 kN/m/m |



Extreme total displacement $4.44 \times 10^{-3}$ m

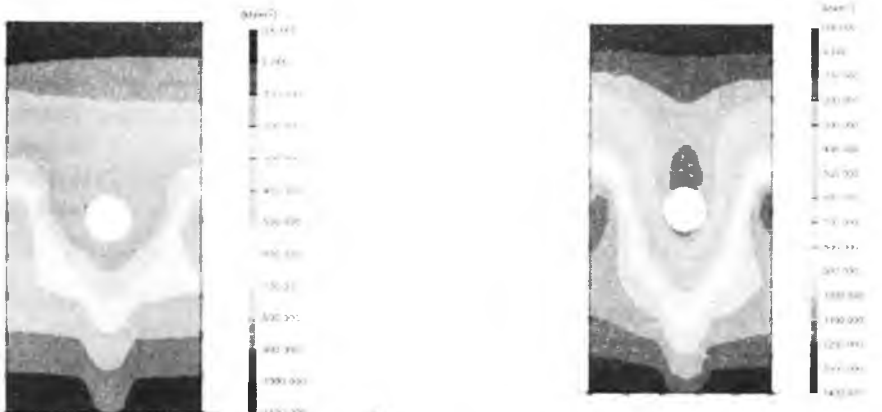
Vertical displacements (Uy) Extreme (Uy) $3.40 \times 10^{-3}$ m

Horizontal displacements (Ux) Extreme (Ux) $3.40 \times 10^{-3}$ m

(Displacements Scaled up $5 \times 10^{3}$ times)

(a) Deformed mesh (b) Vertical deformation (c) Horizontal deformation

Fig. 8. Unlined tunnel deformations



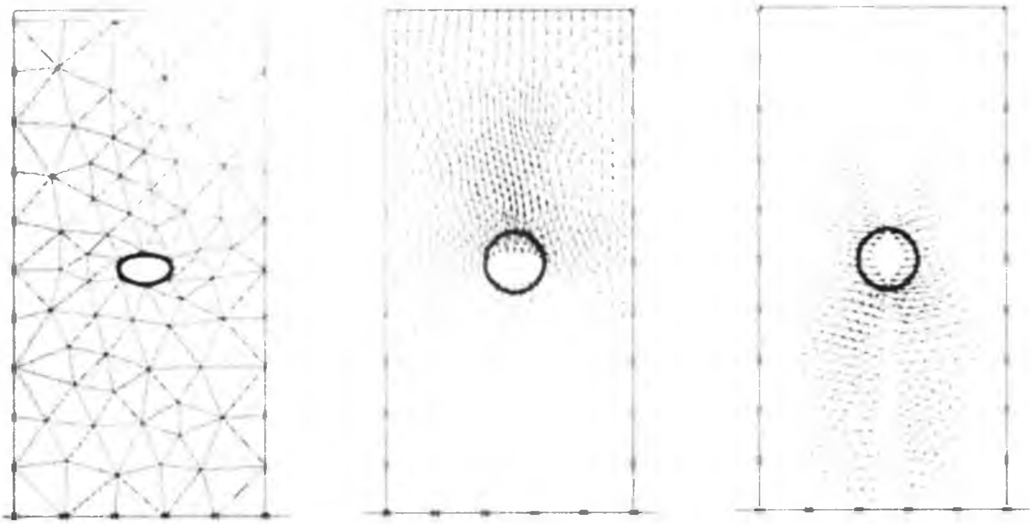
Extreme mean stress - $1.01 \times 10^{3}$ $kN/m^2$

(a) Mean stresses

Extreme Vertical total stresses sig-yy - $1.31 \times 10^{3}$ $kN/m^2$

(b) Vertical stresses

Fig. 9. Stress contours of the unlined tunnel



Extreme total displacement $419.51 \times 10^{-6}$ m

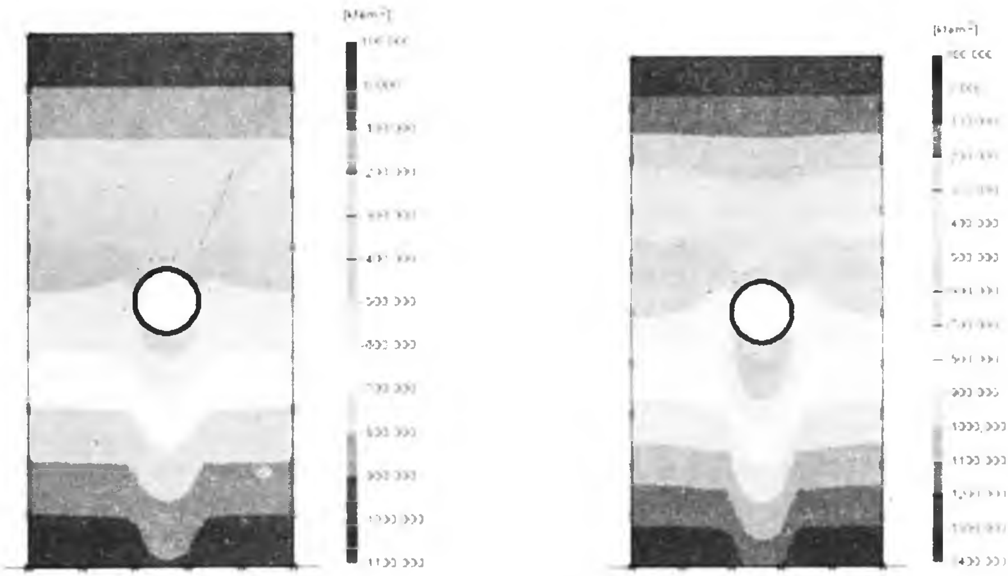
Extreme vertical displacement (Uy) $419.51 \times 10^{-5}$ m

Extreme horizontal displacement (Ux) $0.10 \times 10^{-3}$ m

(Displacements Scaled up $5 \times 10^{3}$ times)

(a) Deformed mesh (b) Vertical displacement (c) Horizontal displacement

Fig. 10. Lined tunnel deformations



Extreme mean stress $- 1.00 \times 10^{3}$ *kN/m²*

(a) Mean stresses

Extreme vertical total stresses *sig-yy* $- 1.30 \times 10^{3}$ *kN/m²*

(b) Vertical stresses

Fig. 11. Stress contours of the lined tunnel

## Discussions and Conclusions

In this study, the stability of a tunnel excavated in weak weathered rock is studied using both discrete and finite element methods. The study indicates that the stability of the tunnel considerably increases with the introduction of lining. This is attributed to the confinement experienced by the ground which increases the arching action. This is evident from the contact force, vertical stress and vertical strain distribution curves obtained from the discrete element method. The same trend is observed in the form of deformations and stress contours from the finite element method. This indicates that both the methods are suitable for the modeling of tunnel assembly, but the choice may depend on the surface conditions assessed. in case the considered stratum has more fractures/discontinuities, a discrete method may provide a more precise result.

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