Considerations on the design of cut-and-cover tunnels

Considérations sur le dimensionnement des tranchées couvertes

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ABSTRACT – The communication shows an alternative for introducing soil-structure interaction in the analysis of cut-and-cover tunnels. An uncoupled method is illustrated through two examples representative of the situations found in practice. The main features of each case are presented and compared. The ultimate limit state behavior and the ductility of these structures are finally discussed.

RÉSUMÉ – La communication présente une alternative pour considérer l'interaction sol-structure dans le domaine des tranchées couvertes. Une méthode découplée est illustrée au travers de deux exemples représentatifs des situations rencontrées en pratique. Les particularités de chaque cas sont discutées et comparées. Finalement, la question de l'état limite ultime et de la ductilité de ces structures est abordée.

1. Introduction

For most structures in civil engineering, the behaviors of a structure and the soil in contact with it can be investigated independently, simplifying the analysis and generally giving satisfactory results. In certain cases, however, such a simplification leads to an unrealistic prediction of the overall system behavior. Cut-and-cover tunnels are an example of this category.

Subjected to the soil pressure, a reinforced concrete structure deforms activating its structural strength. The consecutive displacements in the surrounding soil induce stress redistributions. This generally leads to a more favorable situation for the structure which is usually recognized simply by assuming an active stress state in the soil. Furthermore, in most cases, these stress redistributions have an even greater effect since they enable the development of unloading arches in the soil. The influence of such phenomena on the structural response depends on the geometrical and mechanical properties. It also depends on the capacity of the reinforced concrete structure to accept the displacements required to develop the soil strength.

These effects can be considered by a finite-element model of the overall system. However, it cannot allow the engineer to clearly understand the key features and the role of the many parameters in the soil-structure behavior. This communication presents a possible approach to evaluate the development of unloading arches in the field of cut-and-cover tunnels.

2. Method

The chosen approach to determine the relative contribution of the soil and of the structure is based on a separate evaluation of their behaviors, similarly to the convergence-confinement method developed in the 1970's for the design of supporting structures of driven tunnels (Aftes, 1979 and 1983, Panet, 1995).

Figure 1 shows examples of flexural failure mechanisms for frame and arch-shaped cut-and-cover tunnels with the associated unloading arches in the soil. These mechanisms can be described by a characteristic displacement w and by forces H, called generalized pressures, which summarize in a single value the resistance of the structure under the chosen failure mode.

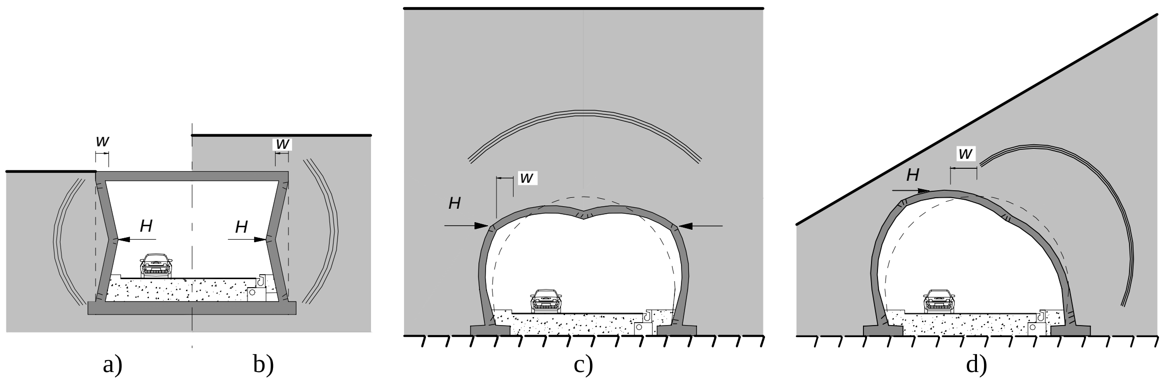


Figure 1. Examples of failure mechanisms for frame and arch-shaped cut-and-cover tunnels with unloading arches, characteristic displacement *w* and generalized pressure *H*

The behavior of the soil is first studied by replacing the structure by an auxiliary unstable structure according to the assumed failure mechanism but stabilized by the generalized pressure *H*. By means of an adequate finite-element software (Z-soil, Zace, 2003 is used in this study), it is then possible to compute the characteristic curve of the soil which describes the relationship between the generalized pressure *H* and the characteristic displacement *w*. The soil is assumed to be elastic perfectly plastic with a Mohr-Coulomb yield criterion. As the soil used for backfill is usually granular, the cohesion is neglected. Contact between the soil and the tunnel is assumed perfectly smooth. Other constitutive laws and contact conditions could also be considered.

In a second phase, the behavior of the structure is evaluated by progressively applying the generalized pressure to the structure. The so-obtained characteristic curve of the structure gives the response of the structure to the action of the soil, represented by the generalized pressure.

The point of equilibrium of the complete system can finally be determined graphically based on a kinematical compatibility criterion. The equilibrium of the system corresponds to the intersection of the two characteristic curves.

3. Examples

3.1. Side wall of a frame-shaped tunnel

A frame-shaped cut-and-cover tunnel is first considered, and more specifically the construction phase when the backfill reaches the top of the side walls. The situation, the failure mechanism and the characteristic values are represented in Figure 1a.

Figure 2a shows the characteristic curve of the soil *(1)* calculated according to the approach described above. The generalized pressure decreases first linearly before it progressively reaches an asymptote corresponding to the complete plastic state of the soil. A failure mechanism is then created in the soil. As a consequence of the different boundary conditions, the kinematics of this mechanism, illustrated in Figure 2c by the corresponding plastic zones, is very different from the Rankine active mechanism (Craig, 1974).

The contact stresses against the wall are also very different from the linear distribution assumed under the Rankine active state. Figure 2d shows a comparison between the two contact stresses distributions. Two particularities can be emphasized. First, the stresses in the lower part of the wall are smaller than the corresponding active stresses. Second, the stresses in the upper part of the wall are much larger than the corresponding active stresses. This is easily explained by the development of unloading arches in the soil. Theses arches take support against the upper part of the wall leading to a stress concentration. The horizontal stresses are then partially transmitted to the wall supports by these arches directly, without loading the wall itself. This results into a decrease of the stresses in the lower part of the wall and a decrease of the generalized pressure. This effect is also partially due to the decrease of the horizontal stresses observed typically when a confined soil mass is relaxed from rest conditions to active conditions. For comparison, the dashed line *(4)* in Figure 2a shows the generalized pressure calculated based on Rankine active conditions. It can be observed that the unloading arches have a non negligible influence on the final generalized pressure. The soil complete plastic state is reached when these arches reach their passive resistance at the top of the wall (region *D* in Figure 2c). This phenomenon was already discussed by Brinch Hansen (1953) and more recently by Mortensen and Steenfelt (2001) in the case of sheet pile walls anchored at the top when a plastic hinge is expected to create in the wall at the ultimate limit state. This case is indeed very similar to the presented example.

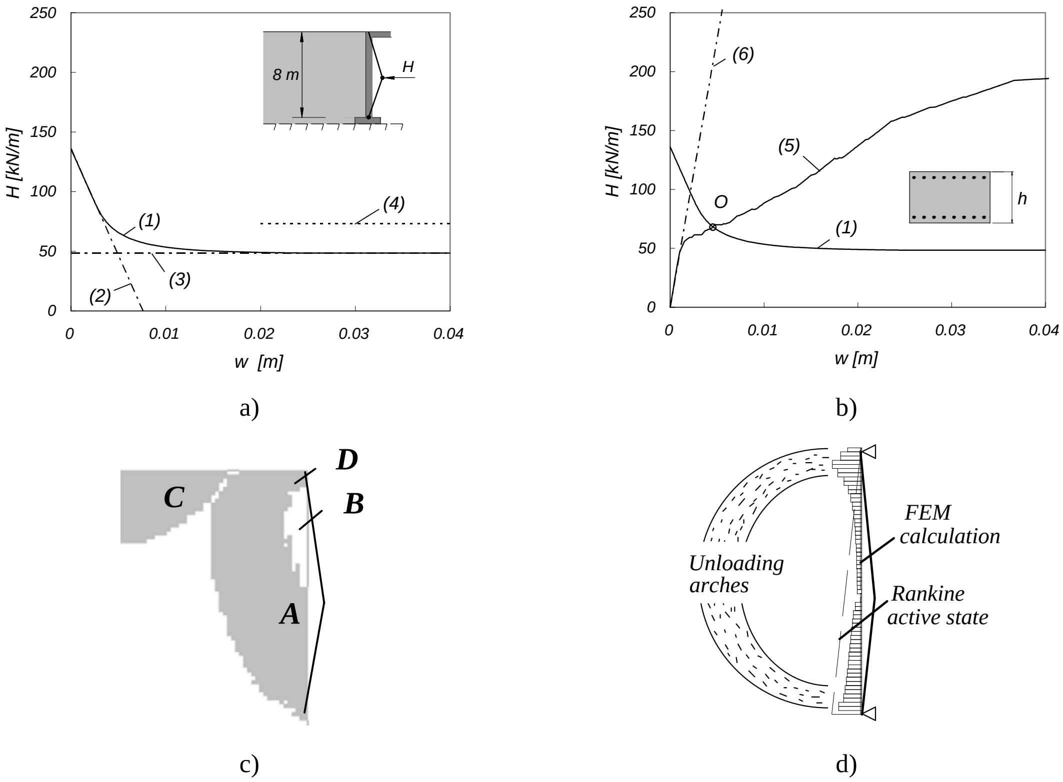


Figure 2. Behavior of a side wall of a frame shaped tunnel at construction stage.
a) soil characteristic curve for E = 40 MPa, ν =0.32, c = 0 and φ = 40°.
b) structure characteristic curve for h = 0.3 m and ρ = 0.54%.
c) plastic zones and d) contact stresses at complete plastic state

Figure 2b shows the characteristic curve of the wall under elastic condition *(6)* and the curve obtained considering cracking of concrete and yielding of steel reinforcement in reinforced concrete *(5)*. The intersection of the two characteristic curves gives the point of equilibrium *(O)* of the system. The soil and structure characteristic curves can easily be recalculated with different sets of parameters, allowing a graphical evaluation of the corresponding effects on the system equilibrium.

In most configurations, the soil plastic state is reached after relatively small displacements that are compatible with the development of the structure strength. It means that in such cases the structure equilibrium can be found without a coupled analysis of the soil and of the structure. The contact stresses at the soil complete plastic state can be determined previously, for example using the theory of plasticity since they only depend on the soil strength properties.

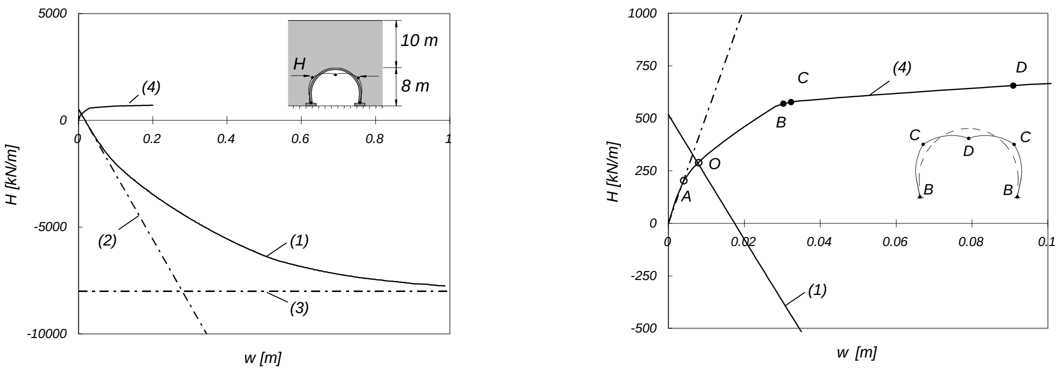
They can then be applied to the structure like an applied load. Unusual situations should however be carefully investigated.

3.2. Arch-shaped tunnel

3.2.1. Symmetrical loading

An arch supporting a large soil cover is now considered. The geometry of the structure is usually chosen to limit the bending moments appearing in the arch. The situation, the failure mechanism and the characteristic values are represented in Figure 1c.

The characteristic curve of the soil given in Figure 3a *(1)* shows that the quasi-elastic phase *(2)* is much more important in this case than in the preceding example. Very large displacements are indeed required in order to activate the total strength of the soil *(3)*. Figure 3b shows the characteristic curve of the structure *(4)* with its various phases (*A*: cracking, *B*, *C* and *D*: development of plastic hinges). In this case, equilibrium is reached while the soil is still in its elastic state *(O)*.



a) b)

Figure 3. Behavior of an arch shaped tunnel under symmetrical loading.
a) soil characteristic curve for E = 40 MPa, ν =0.32, c = 0 and φ = 40°.
b) structure characteristic curve for h = 0.4 m and ρ = 0.60%.

In such cases, the elastic properties of the soil and of the structure play a major role. The soil complete plastic state is reached only for very large displacements incompatible with the structure deformation capacity. Consequently, it does not influence the system behavior. The generalized pressure negative value means that the soil bearing capacity induced by the development of unloading arches is very important.

This behavior is in fact similar to a situation of imposed deformations often encountered in the domain of buildings or bridges. In such cases, the stiffness of the structure should be chosen small enough to limit the resulting forces in the structural members. Consideration of the non linear behavior of reinforced concrete thus becomes important.

As a consequence, the system equilibrium can only be found by a detailed non linear analysis considering the interaction between the soil and the structure.

3.2.2. Asymmetrical loading

The previous arch-shaped tunnel is now submitted to an asymmetrical loading due to an inclined backfill surface with a slope β.

The behavior of the structure can be seen as an intermediate situation between the two previous cases. The structure's shape is no longer adapted to the applied loads and is thus subjected to important bending moments.

This situation can be studied following the general approach discussed in paragraph 2. The assumed failure mechanism becomes asymmetrical and considers the development of four plastic hinges at the points of maximum bending moments (as shown schematically in Figure 1d). The generalized pressure and the characteristic displacement are also shown.

Figure 4a shows the soil *(1)* and structure *(2)* characteristic curves obtained for a set of typical parameters with a slope β = 30°. A comparison with figures 2a and 3a immediately shows that this situation is indeed an intermediate case. In this situation, the displacement required to activate the soil complete plastic state is compatible with the structure deformation capacity but the corresponding generalized pressure is very small. The system equilibrium *(O)* is reached while the soil is still quasi-elastic. The remarks formulated in paragraph 3.2.1 thus remain valid but it should be noted that the structure is highly stressed.

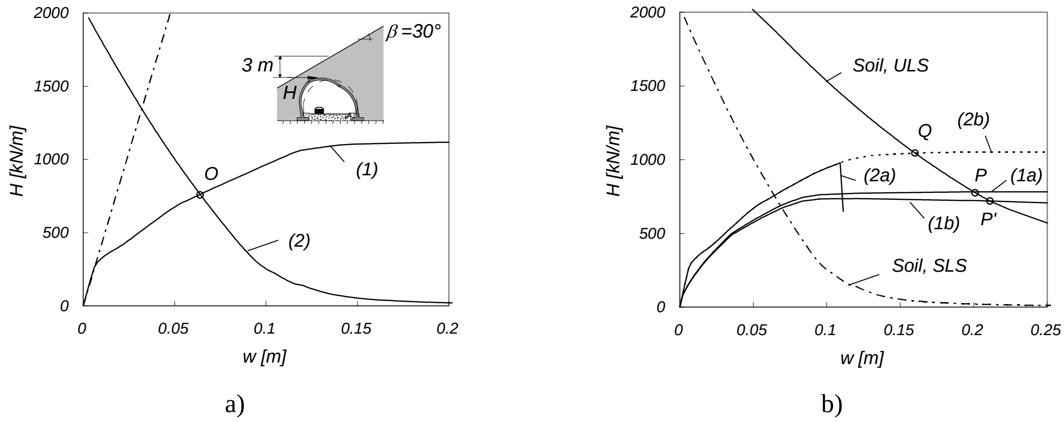


Figure 4. Behavior of an arch-shaped tunnel under asymmetrical loading
a) Soil (E = 40 MPa, ν =0.32, c = 0, φ = 40°) and structure (h = 0.55 m, ρ = 0.83%) characteristic curves b) ultimate limit state

4. Ultimate limit state and ductility

The examples discussed in the previous sections show that different types of behavior can be distinguished depending on the geometry of the situation and on the mechanical properties of the soil and of the structure. These differences also have consequences at the ultimate limit state.

In cases such as the example discussed in paragraph 3.1, the action of the soil on the structure can generally be considered as an applied load. The approach used for traditional reinforced concrete structures with load factors can be applied similarly.

This traditional approach does not seem suitable in the other cases, because it would lead to inadequate stiff solutions. Figures 3 and 4 show that the structural bearing capacity is not a governing design criterion. However, this statement must be limited if second order effects become predominant or if the structure ductility is not sufficient. In such cases, the structure strength is not guarantied for large displacements, meaning that the structure characteristic curve no longer presents a horizontal asymptote.

Ultimate limit state verification should then demonstrate that the structure ductility is sufficient even under unfavorable soil and structure properties. Figure 4b illustrates this approach in the case of the arch-shaped tunnel submitted to an unsymmetrical loading discussed in paragraph 3.2.2.

The soil and structure properties should be decreased or increased depending on the influence of each property on the considered failure mechanism.

The soil behavior must then be determined by adjusting the soil properties in order to increase the contribution of the structure. The factors used in Figure 4b are based on the approach proposed in (Vollenweider and Pralong, 1998) for the design of cut-and-cover tunnels by the finite-element method.

Two different failure mechanisms should be considered for the determination of the structure properties at the ultimate limit state.

The structure resistance properties should be reduced if a flexural failure is assumed to occur by developing four plastic hinges according to Figure 1d. Properties affecting the rigidity of the structure should also be decreased if second order effects are considered. The point of equilibrium is then found without (curve *1a*, *P*) and with second order effects (curve *1b*, *P'*).

The capacity of deformation can however be limited in reinforced concrete structures by brittle phenomena such as crushing of concrete in members submitted to flexure and axial forces (CEB, 1998), shear failure (Muttoni, 2003) or spalling of the concrete cover (Franz and Fein, 1971 and Intichbar et al, 2002) in members without shear reinforcement.

In the example, spalling of the concrete cover in the region of maximum positive bending is considered. Due to the element curvature, positive bending (tension on the inner side of the element) creates radial deviation forces that lead to a brittle failure when the concrete tensile strength is exceeded if no shear reinforcement is provided. A potential failure by spalling of the concrete cover must be verified with an increased reinforcement yield stress in order to maximize the deviation forces in the element. Figure 4b shows that, in this case, the structure fails by spalling of the concrete cover before reaching the point of equilibrium (curve *2a*). The dashed line shows that system equilibrium would be found if the structure ductility was sufficient (curve *2b*, *Q*).

5. Conclusions

Soil-structure interaction in the field of cut-and-cover tunnels can be studied by an uncoupled approach by independently determining the characteristic curves of the soil and of the structure, and then by graphically finding the position of equilibrium of the system based on a kinematical compatibility criterion.

This approach allows to characterize the overall behavior of the system and to quantify the contribution of the soil. Depending on the geometry of a given case and on the mechanical properties of the materials, two different modes are distinguished.

In cases where the soil reaches its complete plastic state with relatively small displacements of the structure, a traditional approach is applicable and a complex analysis considering the soil-structure interaction can be avoided.

In the other cases, system equilibrium is generally reached as the soil is still in a quasi-elastic state. Its determination requires to consider the non linear behaviors of the materials and their interaction. Construction stages and compacting processes also have an influence on the behavior. The system behavior shows many similarities to structures submitted to imposed deformations. The choice of an adequate structure stiffness is of capital importance. At the ultimate limit state, the structure deformation capacity or ductility should be controlled considering cracking of concrete and yielding of reinforcement.

6. References

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