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Assessment of tunnel stability in layered ground

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ABSTRACT: The stability of a circular tunnel in layered ground, with both fine-grained and coarse grained soils below the water table, is investigated experimentally and theoretically. Centrifuge tests have been carried out at City University, London, investigating in detail tunnelling effects in layered ground, in terms of both soil displacements and strains and in terms of the kinematics at failure. Also, analytical upper bound solutions for layered ground which closely reflect the observed failure mechanism of the tunnel have been derived independently. The results from all different approaches have been compared by emphasising the effects of the different hypotheses on the assessment of tunnel stability.

1 INTRODUCTION

The assessment of tunnel stability is an important issue considering the catastrophic effects induced by the tunnel collapse, especially when it concerns urban areas. Surface surcharge loadings may occur in practice where tunnels are excavated below pre-existing structures through fine-grained soil which is overlain by granular material: typical conditions of excavation in urban areas. Although the weights of the overlying coarse-grained materials are easily taken into account when assessing the tunnel stability, the contribution of their strength and stiffness on tunnel stability is often ignored or too difficult to be quantified.

The problem will be approached through the upper bound plasticity theory together with results of centrifuge tests on model tunnels. The case of a tunnel excavated within an over consolidated clay deposit overlain by a sandy layer is considered. Tests included a masonry wall setting in the coarse grained upper layer of soil and thus also gave additional information on the interaction of the tunnel with pre-existing buildings.

This study began from literature references, following two different approaches. In the first case the sandy layer has been taken into consideration as surcharge acting on the top of the clay layer where the tunnel is excavated. In the second case, the sandy layer is properly considered by assigning its appropriate unit weight, thickness, and strength in order to simulate the real initial stress state of soil around the cavity and to emphasise the different behaviour in terms of both undrained and drained conditions.

2 STABILITY OF A SHALLOW TUNNEL

The excavation of a shallow tunnel is clearly a three-dimensional problem. Neglecting the effect due to the volume loss at the front face of excavation, this problem might be analyzed under plane conditions. Figure 1 shows an idealization of shield tunnelling, where a circular tunnel of diameter D is constructed with a depth of cover C. The tunnel lining is regarded

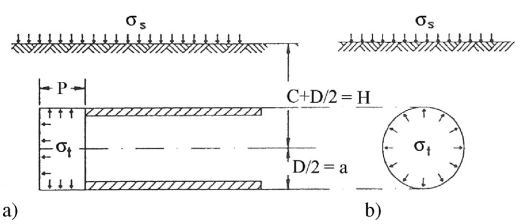

Figure 1. An idealization of shield tunnelling.

as rigid and in front of it the tunnel heading is represented by a cylindrical cavity of length P in which a uniform fluid pressure σ_t acts, and a uniform pressure σ_s acts on the soil surface (Figure 1a). For relatively big value of the distance from the tunnel lining to the tunnel face, the problem can be idealized under plane conditions (Figure 1b).

The collapse of tunnel heading is usually a sudden event caused, for example, by a sudden loss of tunnel support pressure. Then, the stability of tunnels in fine-grained soils can be evaluated by referring to undrained conditions, while the stability of tunnels in coarse-grained soils can be analyzed under drained conditions.

2.1 *Stability of a tunnel in undrained conditions*

The stability of a tunnel excavated in undrained soils may be assessed using the stability ratio, N:

$$N = \frac{\sigma_s + \gamma H - \sigma_t}{S_u} \tag{1}$$

where γ is the soil unit weight; H is the tunnel axis depth from the ground surface; σ_s and σ_t are respectively the surface surcharge pressure and the tunnel support pressure (according to Figure 1); and S_u is the soil undrained shear strength at the tunnel axis level.

Davis et al. (1980) estimated the safety of a shallow tunnel excavated without internal support by considering three different shapes of shallow underground opening, for which upper and lower bound stability solutions were derived. The soil strength S_u was assumed to be constant with depth. Sloan & Assadi (1992) presented a rigorous study taking into account for the variation of the undrained shear strength with depth. The theoretical approach of upper and lower bound solutions were compared to experimental results from a comprehensive study in clays conducted at Cambridge University over the last decades – e.g. Kimura & Mair (1981) who gave a range of design lines to estimate the stability ratio for different tunnel geometries, mainly focusing on undrained conditions.

Generally speaking, the bounding solutions gave good estimates of the collapse load and supported the use of classical limit analyses for undrained conditions. It should be noted that the stability of a plane strain unlined circular tunnel ($L_R/D \rightarrow \infty$) is more critical than that for the lined circular tunnel heading with $L_R/D = 0$ and the analysis is therefore conservative.

Previous literature proposed different charts where the tunnel pressure at collapse is related to soil mechanical properties and tunnel geometry: the term $(\sigma_s - \sigma_t)/S_u$ or the stability ratio N is given as a function of C/D for different ratios of $\gamma D/S_u < 4$. If S_u is constant with depth, then for values of C/D greater than 3 the upper and lower bounds of N do not change significantly with $\gamma D/S_u$. Below C/D equal to 3, there is a larger spread but the lower bound for $\gamma D/S_u = 0$ can be adopted as a safe criterion to calculate the required tunnel support pressure: see Figure 4, by referring to the upper and lower bound curves after Davis et al. (1980). If the ratio $\gamma D/S_u$ is sufficiently large then the collapse will take place for any value of uniform tunnel pressure. Sloan & Assadi (1992) also concluded that for tunnels with C/D>3, these solutions are not fully reliable since the stability of deep tunnels is usually related to a complicated local collapse, involving both elastic and plastic deformation, with only small settlements taking place at the ground surface.

2.2 *Stability of a tunnel in drained conditions*

Atkinson and Potts (1977) derived kinematic upper bound, and statically admissible lower bound plasticity solutions for the two-dimensional idealization in Figure 1. Only dry sand was considered, but the theoretical solutions, based on effective stress, may be applied to saturated sands provided that the pore water is stationary and the pore water pressures around the tunnel are known. The collapse of a tunnel in saturated sand is then simply the sum of the pressures predicted by either the upper or lower bound solutions and the pore water pressure. The situation is rather more complicated if there is a steady state, or transient, seepage. The authors gave equations to obtain the dimensionless ratio $\sigma_t/\gamma D$ as function of C/D: the upper bound solution (being inherently unsafe) gives lower value than the lower bound solution which is inherently safe. The authors concluded that the support pressure is independent of the ratio C/D. They also obtained good comparisons between these theoretical solutions and experimental data from centrifuge tests.

2.3 *Stability of a tunnel in layered ground*

As previously discussed, as far as the authors are aware there is currently no standard procedure to account for the contribution to the tunnel stability of overlying layers of different materials.

Grant & Taylor (2000) studied the stability of tunnels excavated in clay with overlying layers of coarse grained sands and gravels, referring to the design line for clay only given by Kimura & Mair (1981) for $L_R/D \rightarrow \infty$. Experimental data from centrifuge tests with an upper layer of loose sand fitted the design line for clay only, indicating that it may act as surcharge loading and not contribute to the stability of tunnel except in terms of weight. In contrast, the presence of a significant thick layer (at least 1D) of relatively dense coarse grained material, combined with little cover above the tunnel crown (C<2D), will aid the

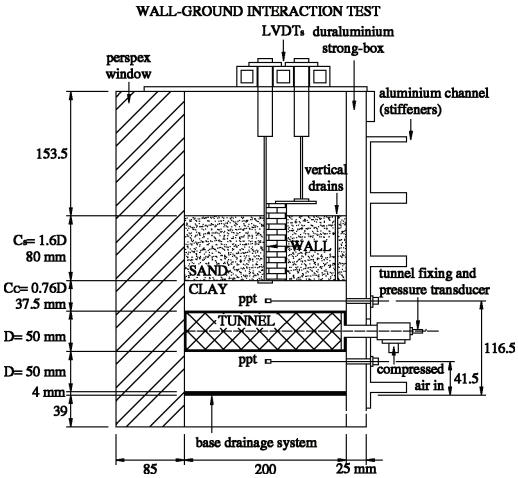

Figure 2. Picture of the experimental model: wall-ground interaction configuration.

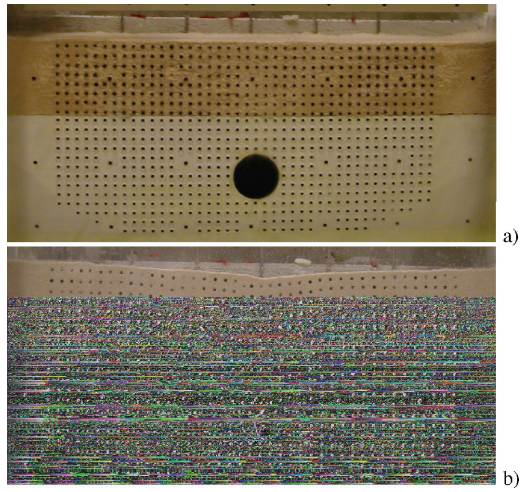

Figure 3. Front view of the model during the test from image processing: a) before reduction of σ_t; $\sigma_{t0} \cong 386$ kPa; b) at tunnel collapse with $\sigma_t \cong 25$ kPa.

stability of a tunnel, and the reasonable value of N equal to 4 might be assumed.

3 CENTRIFUGE MODEL TESTS

Centrifuge model testing has proved a very useful tool to examine the behaviour of shallow tunnels in layered ground, in terms of soil displacements and strains, such as in terms of kinematic failure. A number of tests have been performed at City University, by modelling both the greenfield conditions and the interaction problem between tunnels and pre-existing masonry structures. Details of the apparatus and procedures of tests have been given elsewhere (Caporaletti 2005; Caporaletti et al. 2006) and only the essentials features will be described in this paper.

All centrifuge tests were performed under plane strain conditions: Figure 2 shows a schematic of the centrifuge model. A layer of pre-consolidated kaolin is overlain by a layer of medium dense sand. The kaolin slurry was pre-consolidated by applying a one-dimensional load in a press to a vertical effective stress of 500 kPa, before allowing the clay to swell back to 250 kPa. The overconsolidation ratio ranged between 1.4 to 2.8 with depth. The stratum of medium dense sand was made by manual pluviation and then frozen. A 50 mm cavity was cut through the clay layer using a thin walled-cutter and lined with a thin rubber membrane of negligible stiffness and strength. All tests were carried out at the same scale factor $N_g = 160$ in order to model a real tunnel of diameter equal to 8 m, excavated in a clay layer of 22 m depth overlain by a sand layer of 12.8 m depth, with the tunnel axis at about 23 m from the ground surface. The effects of tunnelling on pre-existing structures were studied by modelling a completely buried thin masonry wall perpendicular to the tunnel axis, with foundations just at the sand-clay interface. It represents a stiffer and heavier inclusion in the sand layer.

During the centrifuge spin-up the tunnel air pressure σ_t applied a uniform radial total stress into the cavity to balance the total stress at tunnel axis level:

$$\sigma_t = \gamma_s C_s + \gamma_c (C_c + D/2) \quad (2)$$

where γ_S is the unit weight of sand; γ_C is the unit weight of clay; C_s is the thickness of the sand layer; C_c is the cover above the tunnel in the clay layer; and R is the radius of tunnel as shown in Figure 2. This tunnel pressure represents the compressed air, bentonite slurry or a shield used in practice to achieve the tunnel stability during the excavation process. Water was supplied to the model through a stand-pipe to maintain a predetermined water-table throughout the model: the water level was set up at different depths from the ground surface in order to evaluate the influence of the different effective stress distribution on the tunnel stability. Equilibrium pore pressure was measured by miniature pore pressure transducers around the cavity. The tunnel excavation has been performed by reducing the air tunnel pressure until the cavity collapse, in a period of 3–4 minutes at a rate of approximately 85 kPa/min.

Digital images were taken every second by using a video camera fixed on the swinging platform to view the front face of the strongbox during tests and to follow the movements of black targets pushed both in clay and in sand. Figure 3 shows images of the model in flight.

In spite of the relatively high value of the kaolin permeability, the stratum of clay performs general

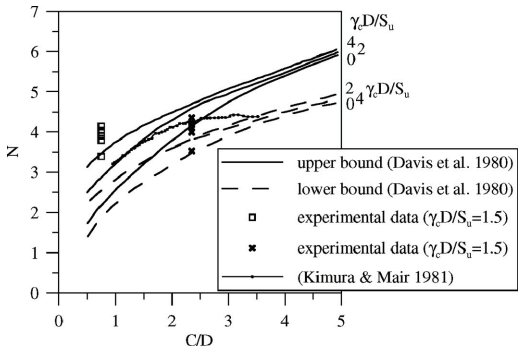

Figure 4. Stability ratio, N: experimental data against theoretical solutions for clays under plane conditions.

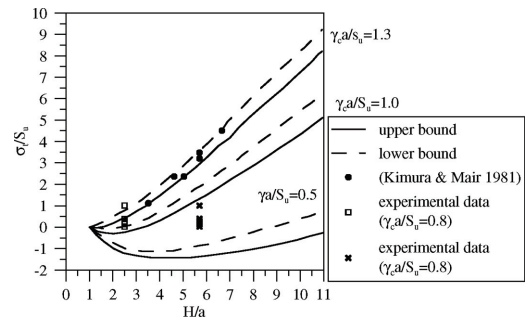

Figure 5. Tunnel pressure, σ_t: experimental data against theoretical solutions for clays under plane conditions.

undrained conditions during the reduction of tunnel pressure. However, considering that the upper layer of sand represents a significant boundary of drainage for the lower kaolin layer, and considering the small thickness of the clay cover above the tunnel cavity, local drained conditions in this upper part of the kaolin layer were likely in those tests.

3.1 *Centrifuge test results*

The value of the tunnel pressure, σ_t, has been evaluated for each test, in order to compare experimental results to the guidelines from literature references.

The cavity was excavated in clay, and so design lines for homogenous undrained soils have been considered first. Figure 4 and Figure 5 present the comparison in terms of the stability ratio, N, and of σ_t/S_u depending on the soil mechanical property and tunnel geometry. The profile of the undrained shear strength with depth has been calculated following the equation (Koutsoftas & Ladd 1985):

$$\left(\frac{S_u}{\sigma_v'}\right)_{OC} = 0.22 \cdot OCR^{0.8} \tag{3}$$

were σ_v' is the vertical effective stress, and OCR is the overconsolidation ratio. Considering the relatively small thickness of the clay cover, a constant value of S_u has been chosen in order to compare test results with theoretical solutions by Davis et al. (1980) for the corresponding value of the term $\gamma_c D/S_u = 2$. In the interpretations analyzed afterwards, the constant value of S_u refers to a characteristic depth respectively equal to 5 m and 1.8 m from the sand/layer interface (Ribacchi et al. 1993).

Experimental data represented by open squares have been analyzed by simply assuming the sand layer as surcharge acting on the top of the clay layer, σ_s, and it is equal to its own weight. Then, the tunnel cover, C, is only equal to C_c. On the contrary, full symbols represent the experimental results analyzed by properly including the sand layer in the evaluation of the tunnel cover. The sand layer is allowed to make a contribution to the tunnel stability as a soil with similar strength to the clay. The surface surcharge pressure, σ_s, is therefore equal to zero, and the tunnel cover $C = C_s + C_c$. The comparison between the two different data interpretations emphasizes that the ratio of the cover to the diameter of the tunnel is clearly a significant aspect on the assessment of the pressure at collapse. A good fitting of experimental data to previous plasticity solutions from literature references was obtained only when the tunnel cover is properly considered (full symbols). While, the open squares are always out of the range given by theoretical lower and upper bounds. However, this agreement definitely worsens if the real value of S_u at tunnel axis is assumed instead of the value at the characteristic depth as previously explained.

Afterwards, experimental data have been compared to theoretical solutions for cavity excavated in sands. Design lines after Atkinson and Potts (1977) are shown in Figure 6, by distinguish the original lower and upper bound approaches for dry sands, and the lines obtained for saturated soils. Once again, the physical results are clearly out of the two ranges given by theoretical solutions.

The load factor, which is the ratio between the stability ratio at working conditions and at collapse (the subscript $_0$ refers to the beginning of the test before the reduction of the tunnel support pressure):

$$LF = \frac{\sigma_{t0} - \sigma_t}{\sigma_{t0} - \sigma_{tc}} \tag{4}$$

has been calculated at different values of the volume loss, V_L, defined as the volume of settlement trough at the tunnel axis, and plotted in Figure 7.

In order to have a clear representation of experimental data, only results from few tests have been shown. Test PC2 and test PC8 had greenfield conditions but with two different hydraulic boundaries: the

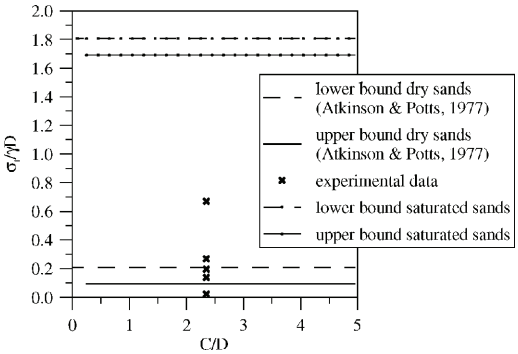

Figure 6. Tunnel pressure, σ_t: experimental data against theoretical solutions for sands under plane conditions.

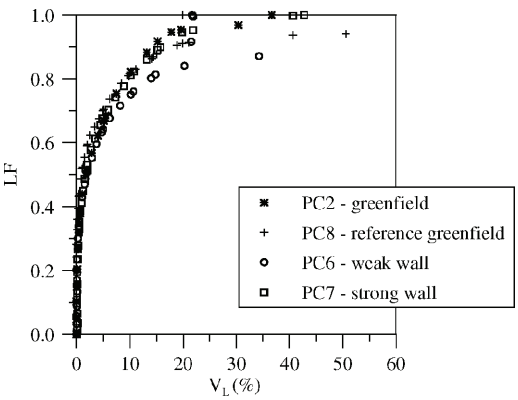

Figure 7. Curves of load factors against volume loss.

water level is very close to the ground surface (PC2), and very close to the sand/clay interface (PC8). Soil-structure interaction conditions existed in Tests PC6 and PC7, in which were modelled respectively the weakest and the strongest masonry structure, with the same hydraulic conditions fixed during test PC8. As expected, focusing at first on the greenfield tests, higher values of the tunnel support pressure were measured during test PC2, which means a more unstable configuration due to the different effective stress distribution experienced as the consequence of differences in the hydraulic boundary conditions. At the same way, analyzing the results of soil-structure interaction tests, the masonry wall built in the sand layer represented a stiff inclusion within the soil and made the tunnel less stable. The tunnel collapse occurred for higher values of the cavity pressure, and the worst condition was reached when the strong wall was modelled. Looking at the point of intersection between the load factor curves related to the soil-structure interaction tests, and the reference greenfield curve, it is evident that the weakest wall always stayed in contact with the upper face of kaolin, which did not occur with the strong wall that lost this contact at value of $V_L \approx 15\%$. In spite of this, the failure mechanism observed in the soil around the tunnel seemed to be very similar. It is particularly interesting to emphasise that for all tests the experimental curves approximately show a horizontal asymptote after volume losses of around 20%. Therefore, the condition corresponding to the tunnel collapse has always been taken as that corresponding to the stage when $V_L > 20\%$.

3.2 *Tunnel mechanism of failure*

Experimental data have been analyzed in terms of the effect due to the layered configuration on the field of soil stress and strain (Caporaletti, 2005; Caporaletti et al., 2006). The two fine-grained and coarse-grained materials, sand and clay, have clearly shown different mechanical behaviours. In particular, the sand tendency to dilate was constrained by the lateral walls of the strongbox, and resulted in vertical settlement decreasing with depth. At the base of the sand layer, the pattern of soil strains indicates expansion near the tunnel axis. In contrast, vertical settlements within the clay layer always increased with depth, and non-zero volumetric compressive strains were measured above the cavity close to the tunnel axis, maybe due to there not being perfectly undrained conditions locally maintained during the tests. According to literature references, the mechanism of failure for tunnel in clays propagates upwards and outwards from the cavity invert becoming significantly wider than the tunnel diameter. In sands, failure tends to involve a narrow "chimney", propagating almost vertically from the cavity up to the ground surface (Mair & Taylor (1997)). However, in these centrifuge tests the mechanism at failure for a layered configuration involves a wide area of soil both in sand and in clay, with pseudo-vertical settlements at the sand-clay interface. The kinematic mechanism is clearly characterized by soil displacements pointing towards the cavity on the top of the sand layer and everywhere within the clay layer, and by a rigid vertical block translation at the base of the sand layer, for a thickness of about D/2 (see Figure 8). The field of ground displacements in sand is strictly related to soil movements induced at the top of the clay layer more than to the reduction of the tunnel support pressure itself.

4 THEORETICAL ANALYSIS

The comparison of experimental data to published literature guidelines has clearly shown discrepancies in terms of evaluation of the soil behaviour at collapse, when both fine and coarse grained material strata are involved in the problem. The effect due to the layered

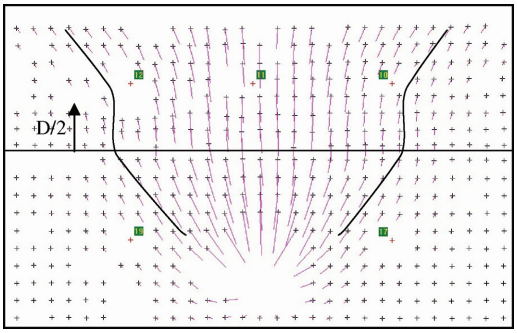

Figure 8. Mechanism of failure from centrifuge tests (VL ≅ 20%).

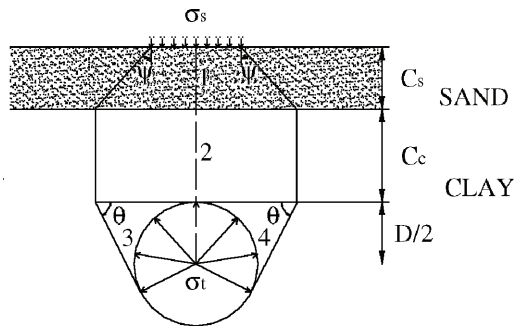

Figure 9. Mechanism for layered ground.

configuration need to be better understood: kinematic analyses could explain the observed behaviour.

A mechanism related to the problem of stability of a plane strain unlined circular tunnel excavated in layered ground has been evaluated. The calculation relies upon the assumption that failure happens under drained conditions in sand and undrained conditions in clay. The soil is idealized as a perfectly plastic material with an associated flow rule: unit weight, γ_s for sand, and γ_c for clay, undrained shear strength of clay, S_u, and angle of dilancy of sand equal to the maximum angle of shear resistance, $\Psi = \varphi'$. The mechanism is shown in Figure 9. It starts from that given by Davis et al. (1980) for clays, but it assigns the appropriate unit weight, thickness, and strength to the coarse-grained stratum. Due to the assumption of associated flow, the mechanism shows vertical movements within the sand layer, according to the experimental data (see Figure 8). Vertical movements are also assumed to be generated in clay above the cavity, by simplifying the real kinematic observed from centrifuge tests (see Figure 8). The work calculation is performed by changing the angle θ in order to minimize the value of the tunnel pressure and to achieve an upper bound solution, with an unsafe estimation of the pressure at collapse.

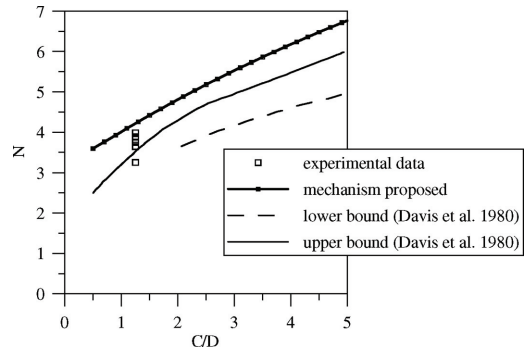

Figure 10. Theoretical solutions and experimental data.

The theoretical solution for this mechanism is plotted in Figure 10 together with centrifuge results and literature solutions for homogenous undrained soils: the stability ratio, N, refers to the actual undrained strength of clay at the tunnel axis. The cover in sand has been assumed equal to D/2, following the kinematic mechanism observed during centrifuge tests, while part of the thickness of the sand layer is evaluated as an external surcharge. The stabilizing contribution due to the friction acting in sand is clearly evident, and its influence on the evaluation of the stability ratio should not be neglected. Experimental data are clearly out of the range given by lower and upper bounds for clays, whereas a satisfactory upper bound is assessed by adopting the theoretical solution by following the mechanism herewith proposed.

5 CONCLUSIONS

Physical modelling and theoretical solutions have been studied in this research and compared to one another, to highlight the effect on the tunnel stability due to a layered configuration that involves both fine-grained and coarse-grained materials, since previous works only refer to homogenous soils.

In the case of tunnels excavated in clay overlain by sand, the contribution to the stability due to the friction acting within the upper layer represents a significant contribution, which should not be neglected. Coarse-grained material cannot satisfactory be considered as an external surcharge equal to its own weight only. A significant over-estimate of the tunnel support pressure to prevent collapse might result if simple reference to the literature theoretical solutions obtained for homogenous clays is made and the sand layer treated only as a surcharge. The proposed new mechanism provides a good upper bound to the experimental data though some simplifications of the proposed kinematic mechanism could be removed, and though it

is recognised that a new lower bound is needed to complete the analysis.

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