# A Bluffer's Guide to Stability Part 2

**In this article, Dr Benoît Jones, Director of the Tunnelling and Underground Space MSc at the University of Warwick, UK, provides a straightforward guide to stability in soft ground. This Part 2 of 'A Bluffer's Guide to Stability' goes into more detail comparing various methods of drained stability calculation.**

### Introduction to drained stability

As discussed in part 1 (in the last issue), there are two main points to understanding stability:

1 There are two forces that cause instability: gravity and seepage forces.

2 All headings will fail without either cohesion or support pressure.

Without either cohesion to hold the soil grains together or a support pressure applied to the face, a vertical face will fall down due to gravity. In the case of a tunnel below the water table, seepage forces will also contribute to instability. Seepage forces occur when there is a hydraulic gradient between the ground and the tunnel. As groundwater seeps through the ground towards the face, it pushes the soil grains apart with a 'seepage force' proportional to the hydraulic gradient, in the direction of flow.

'Drained' soils are soils that are permeable enough that excess pore pressures are dissipated during the timescale of construction. For tunnel heading stability this means that any negative excess pore pressures that may have helped hold the soil grains together cannot be counted on, as water will have time to flow from the surrounding ground. Therefore, the failure of drained soils depends on the drained cohesion *c'* and the internal angle of friction *Ø*.

### Dry cohesionless soils

The simplest situation to consider is that of dry, cohesionless soils, i.e. where *c'* = 0. In this case failure is governed by the geometry of the heading, the unit weight of the soil and the value of angle of friction *Ø*. In this case it is certain that a support pressure is required. This may be provided by slurry pressure in a slurry machine, compressed air in an open face, or by earth and water pressure in an earth pressure balance (EPB) machine.

Atkinson & Potts (1977) performed 1g (i.e. scale models on the laboratory floor) and centrifuge tests on dry cohesionless Leighton Buzzard sand. They also developed upper bound and lower bound limit state solutions for a plane strain tunnel. If you remember from part 1, the upper bound is any kinematically admissible mechanism (as shown in Figure 1), and it may be said that at this load the heading must fail. The lower bound is a statically admissible stress field (Figure 2) which nowhere violates the failure criterion for the soil, and it may be said that at this load the heading cannot fail. The upper and lower bound bracket the true collapse load, and ideally they are close enough together that a target support pressure may be set.

**Figure 1: Upper bound mechanism in 2D plane strain (from Atkinson & Potts, 1977)**

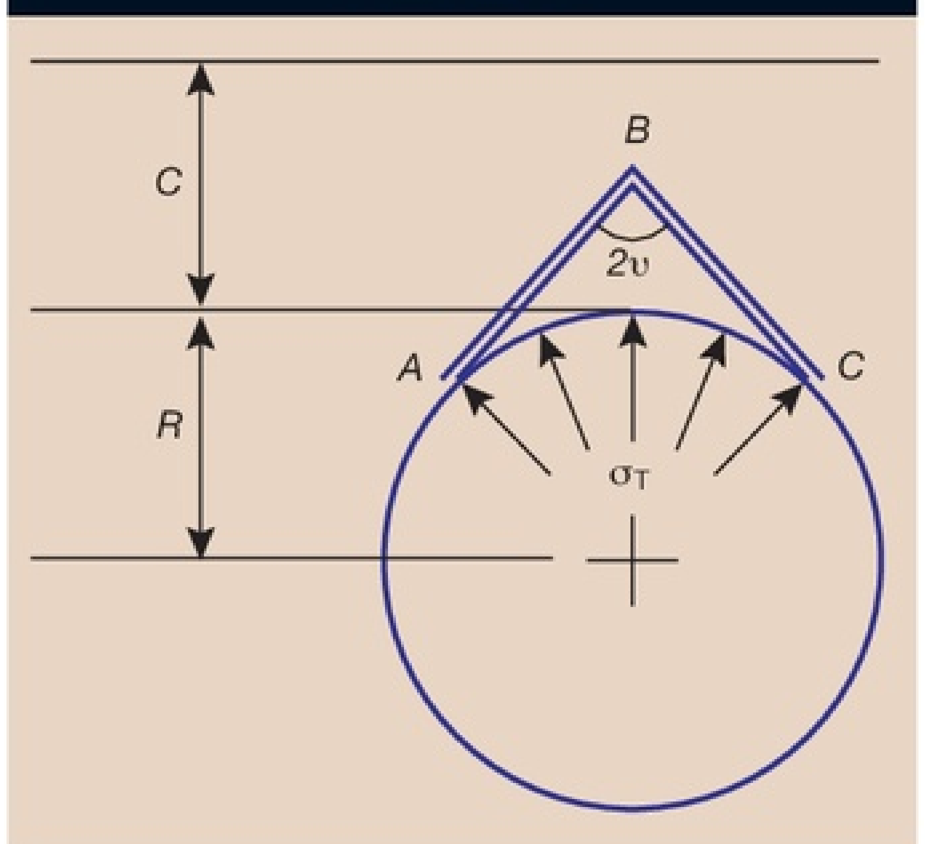



**Figure 2: Lower bound stress field in 2D plane strain (from Atkinson & Potts, 1977)**

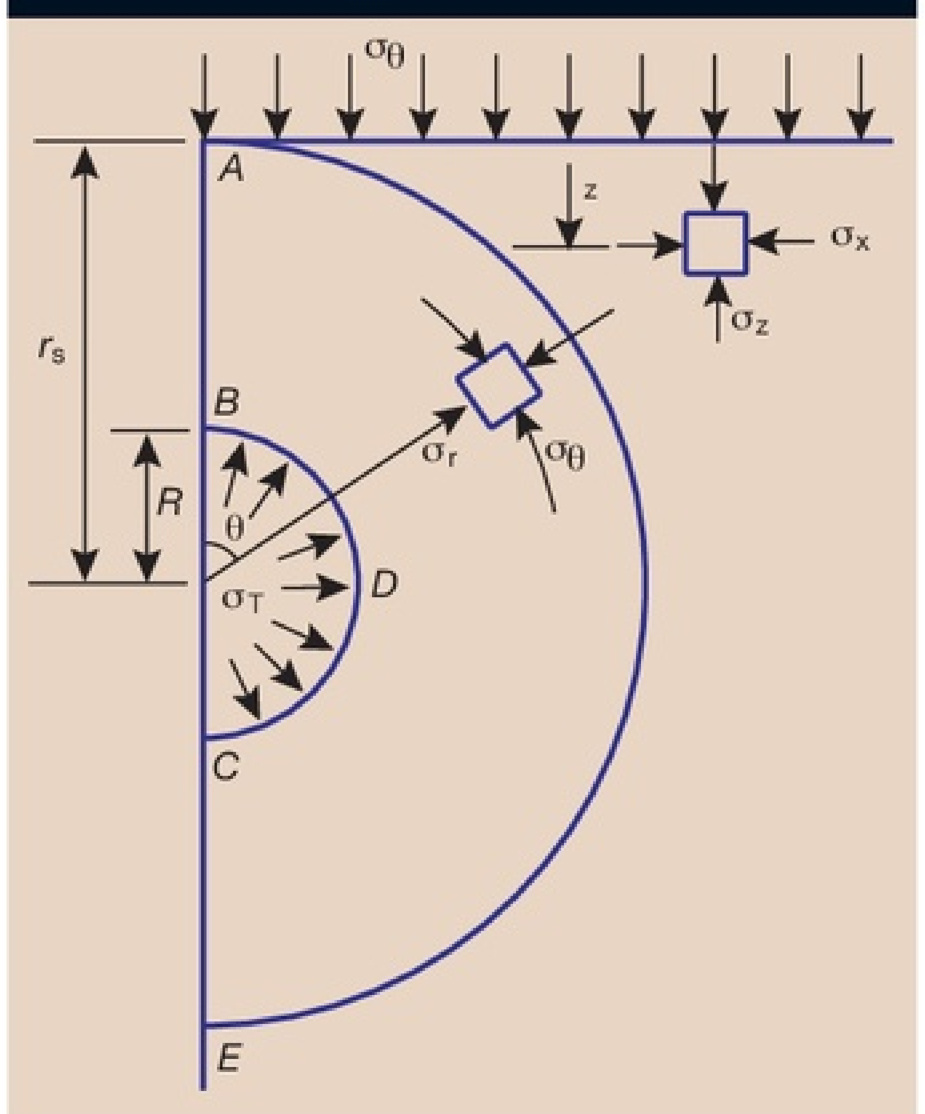
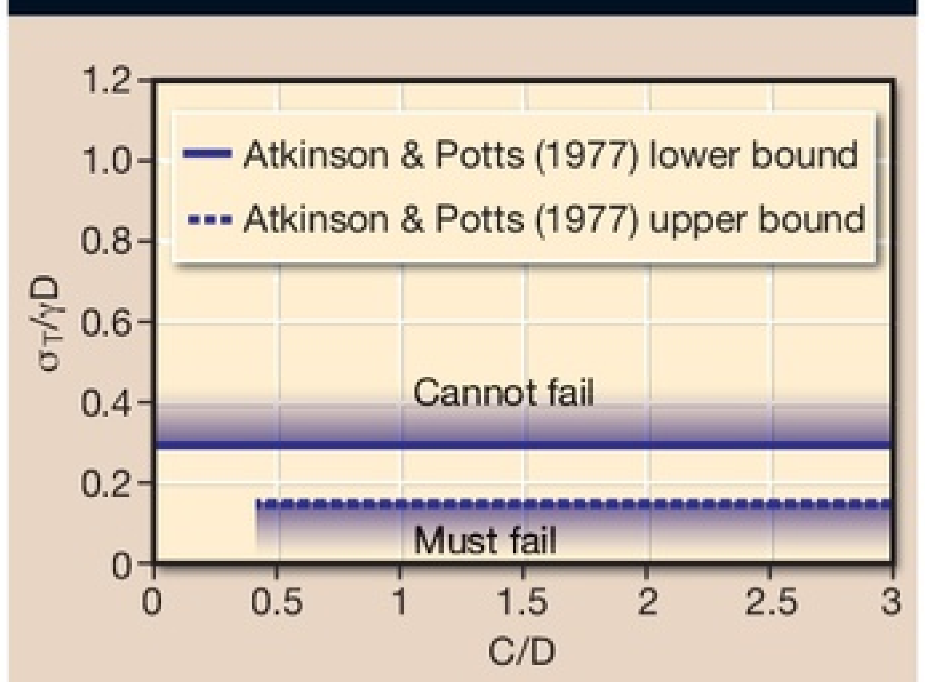



Figure 3 shows the upper and lower bound solutions for a dry cohesionless soil from Atkinson & Potts's paper (1977). The vertical axis is the support pressure $\sigma_T$, normalised by the unit weight of the soil $\gamma$ and the diameter *D*. The horizontal axis is the ratio of cover *C* to diameter *D*, and it can be seen that the depth of the tunnel does not affect the limit state solutions. Remember this is different to an undrained stability problem, where depth is important.

The upper bound is based on a triangular wedge of soil falling from the crown of the tunnel, as shown in Figure 1. The geometry of

**Figure 3: Upper and lower bound solutions for a dry cohesionless soil (Atkinson & Potts, 1977) with *Ø* = 35°.**




this wedge depends on the angle of friction *Ø*. At low values of *C/D*, the wedge intersects the surface, so no values can be determined.

Although intended for dry cohesionless soils, it may be possible to use this solution for cohesionless soils below the water table if it can be assumed that there is no flow of groundwater and hence no seepage forces. In this case one would use the submerged unit weight $\gamma - \gamma_w$.

**Dry c' - Ø soils**

Leca & Dormieux (1990) found upper and lower bound limit state solutions for $c' - \varnothing$ soils, that is, drained soils with cohesion. This was a great step forward because many natural soils have some cohesion. The geometries of the lower and upper bounds considered by Leca & Dormieux are shown in Figure 4 and Figure 5.

Figure 6 shows the effect of increasing the value of cohesion from 0 to 5 to 10kPa for a soil with $\varnothing = 35°$. Both the upper and lower bounds move downwards, meaning that the required support pressure to prevent instability has decreased.

At $c' = 10$kPa, Leca & Dormieux's upper bound is at zero, meaning no support pressure would be required if the upper bound represented the true collapse mechanism, which we don't yet know for sure. This illustrates that tunnel headings in drained soil above the water table may be stable at quite small values of cohesion, which may be provided by moisture in the soil (like a sandcastle), a low clay content or a small amount of cementing of the soil grains. However, driving a tunnel in such a situation must be done carefully, because even a small amount of perched water can cause instability due to seepage forces, and low values of cohesion may not always be relied upon given the inherent variability of geological materials.

Note that Leca & Dormieux's upper and lower bounds are further apart than Atkinson & Potts's for $c' = 0$; this is because they assumed different stress states for the lower bound and different mechanisms for the

**Figure 4: Upper bound mechanisms from Leca & Dormieux (1990).**

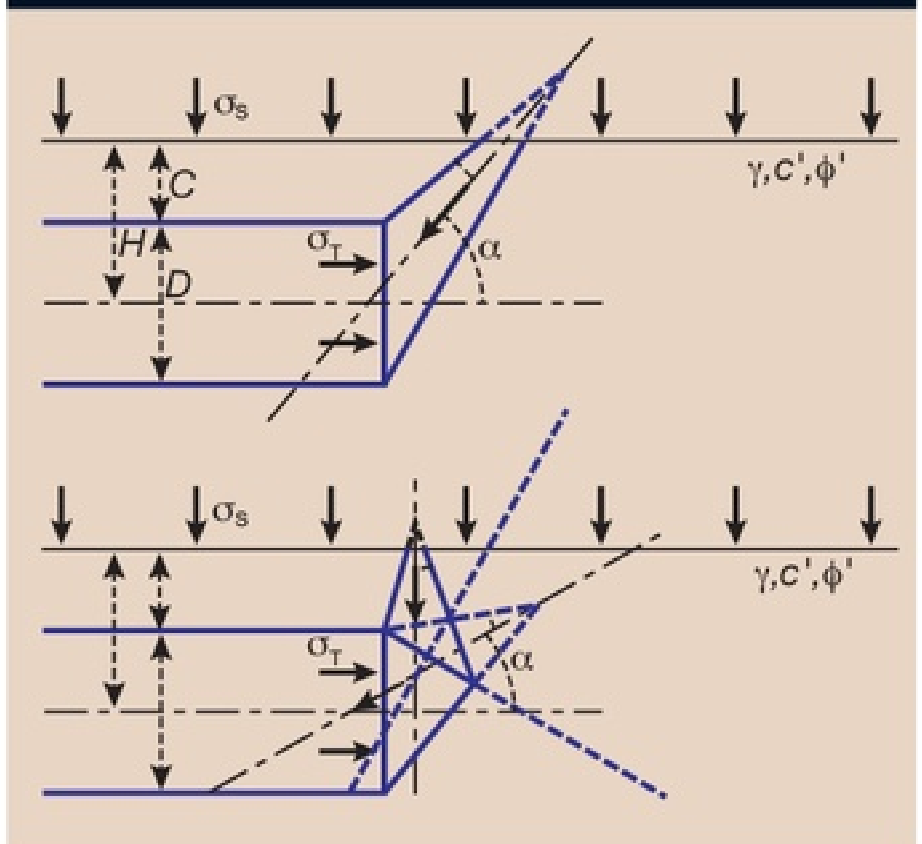


**Figure 5: Lower bound stress fields from Leca & Dormieux (1990). (a) SI, (b) SII, (c) SIII.**

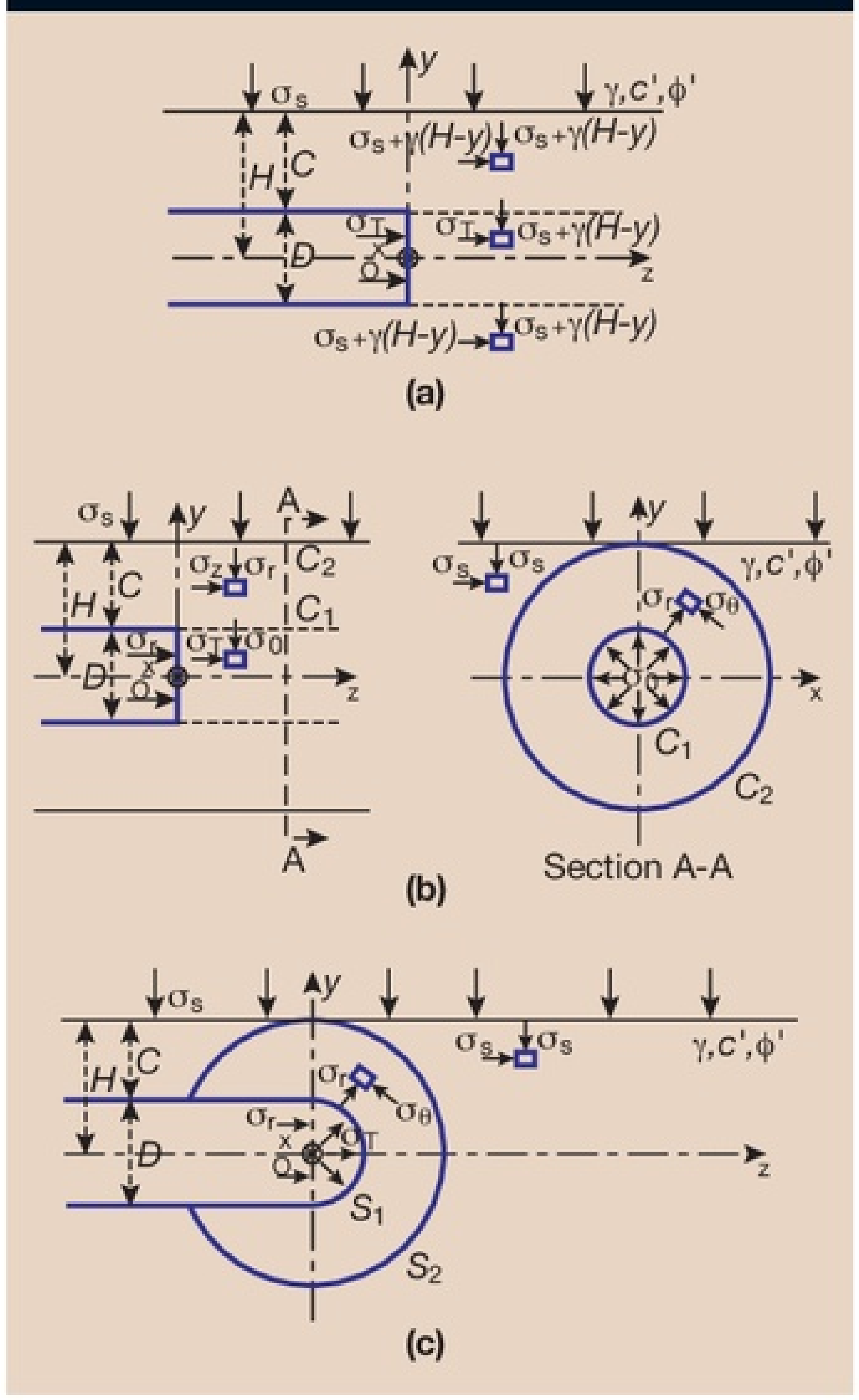


**Figure 6: Effect of drained cohesion c' on required support pressure $\sigma_T$ in a dry c' – Ø soil with Ø = 35°. (A&P = Atkinson & Potts, L&D = Leca & Dormieux, L/B = lower bound, U/B = upper bound).**

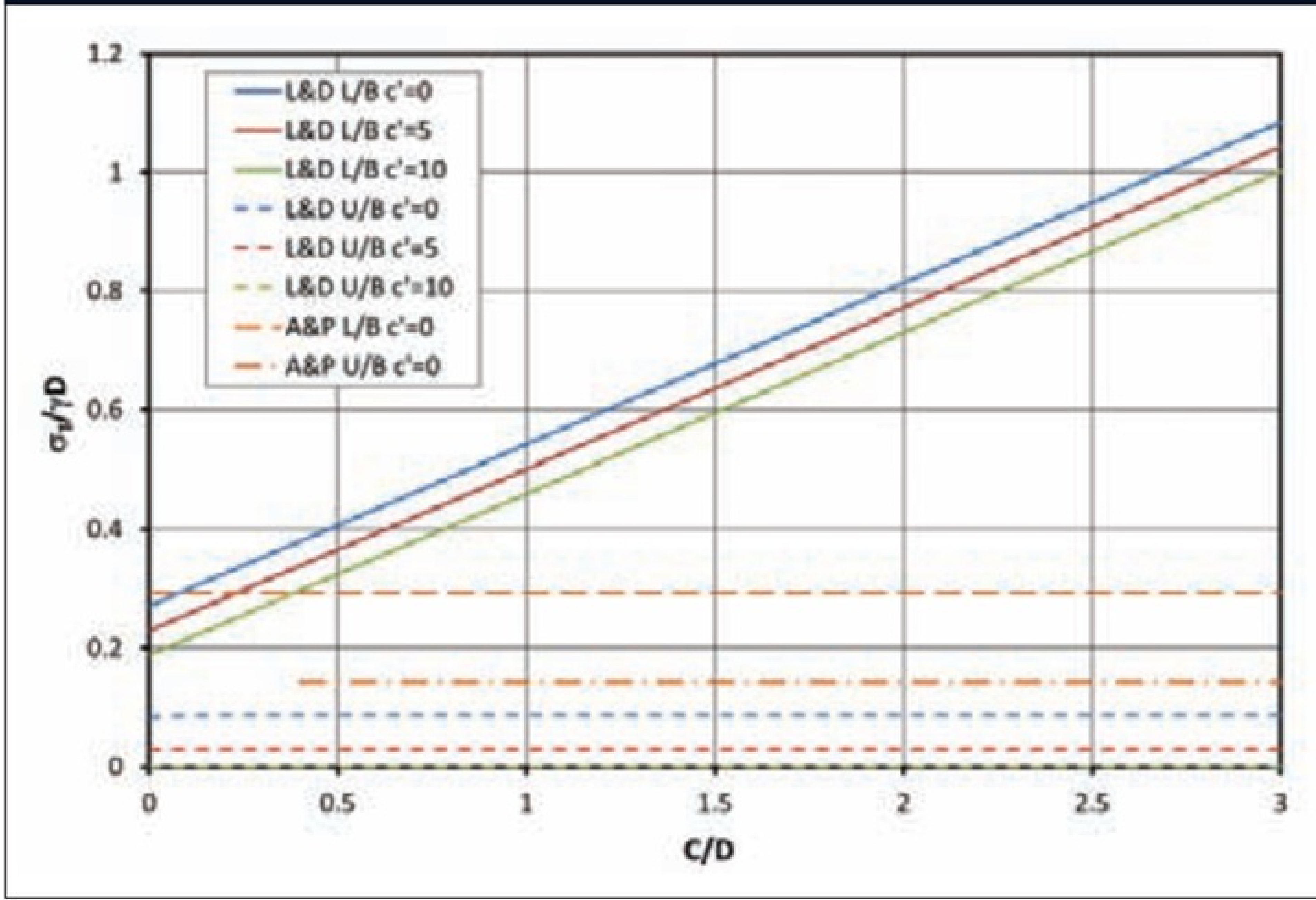
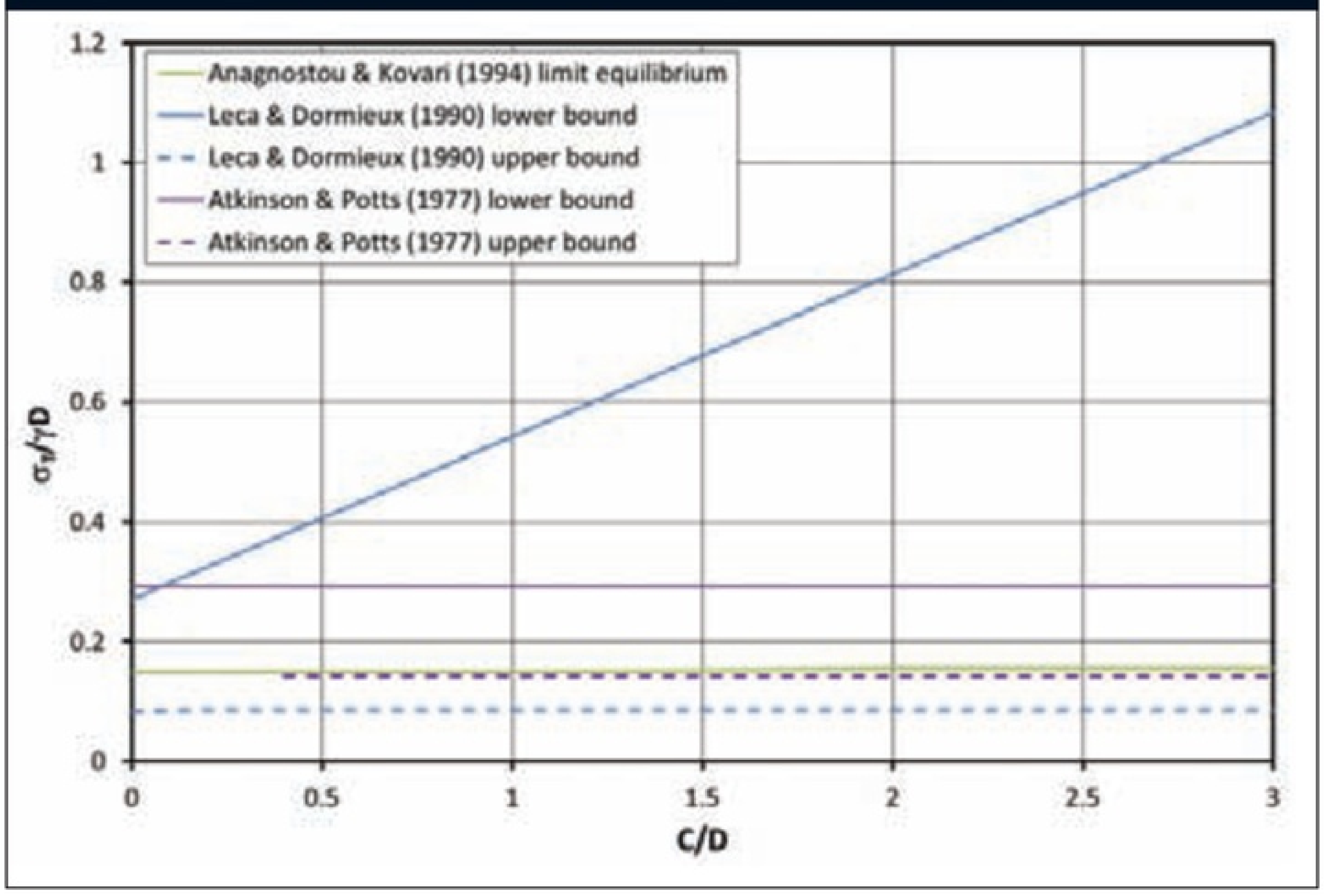



**Figure 7: Comparison of Anagnostou & Kovári's (1994) limit equilibrium solution with limit state solutions by Leca & Dormieux (1990) and Atkinson & Potts (1977) for a dry cohesionless soil with Ø = 35°.**

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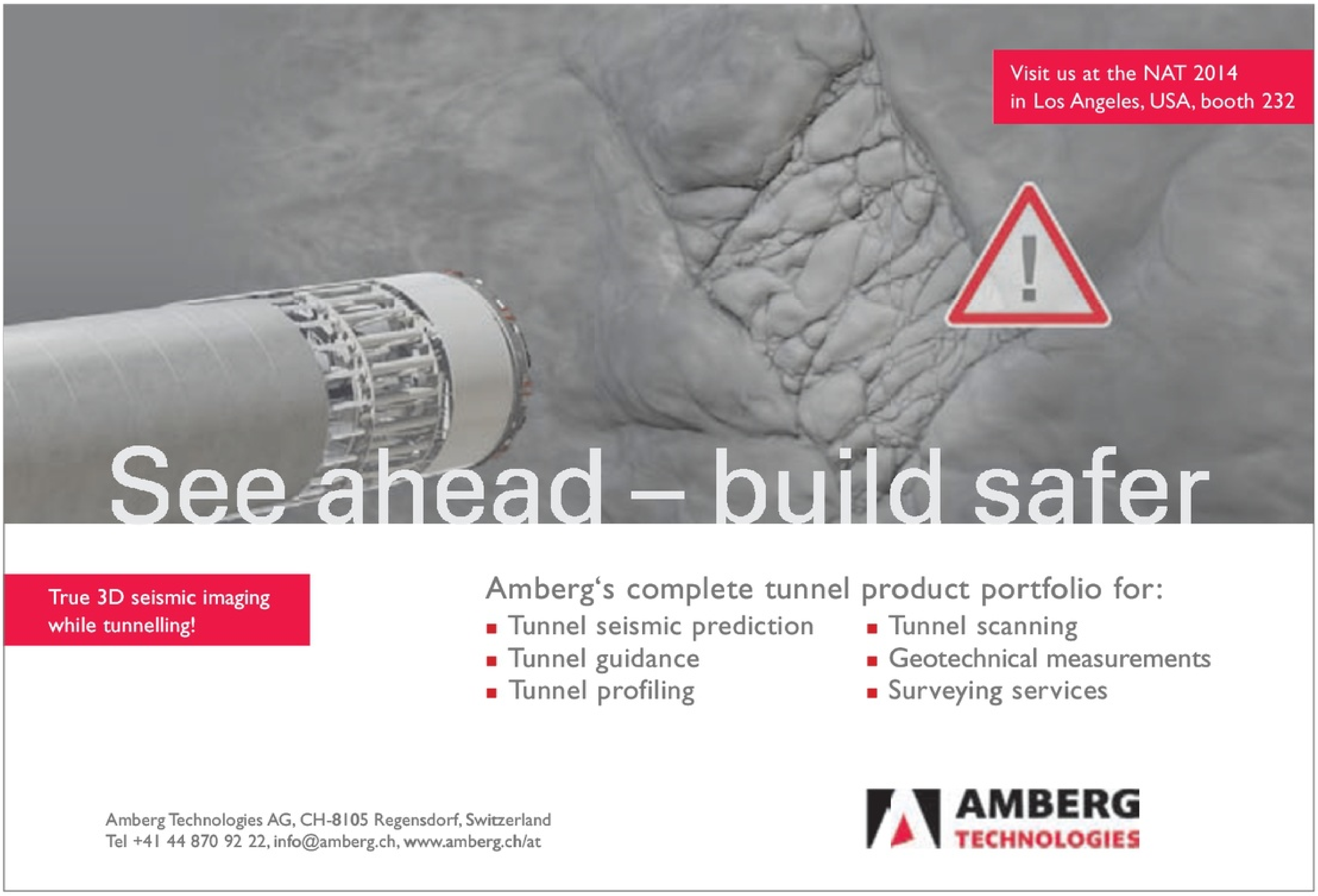
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**Figure 8: Comparison of centrifuge tests by Messerli et al. (2010) on dry cohesionless sand with limit state solutions (L&D U/B = Leca & Dormieux (1990) Upper Bound, A&P U/B = Atkinson & Potts (1977) Upper Bound), a limit equilibrium solution (A&K LE = Anagnostou & Kovári (1994)) and finite element models (FEM = FE models by Vermeer et al. (2002)).**

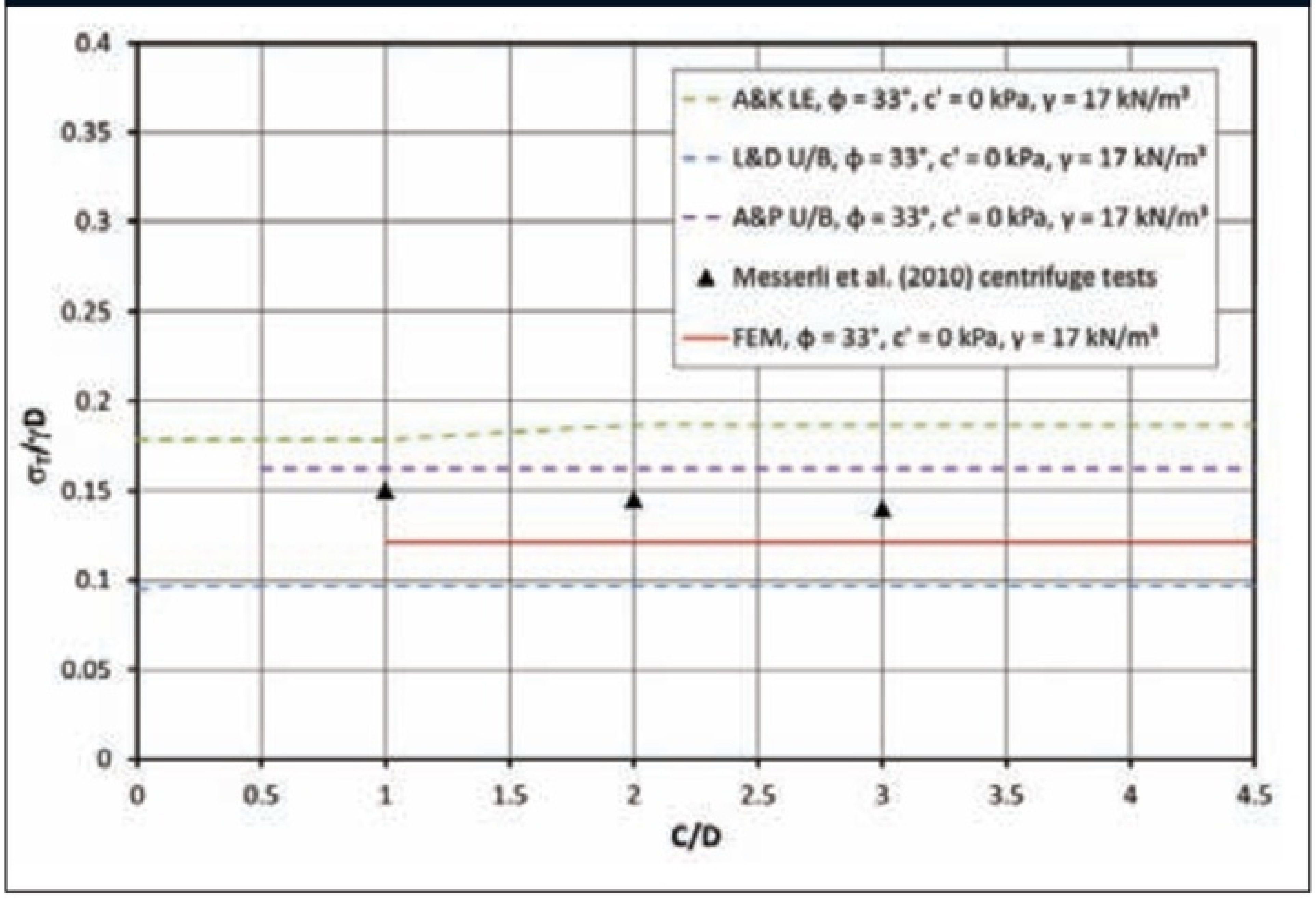
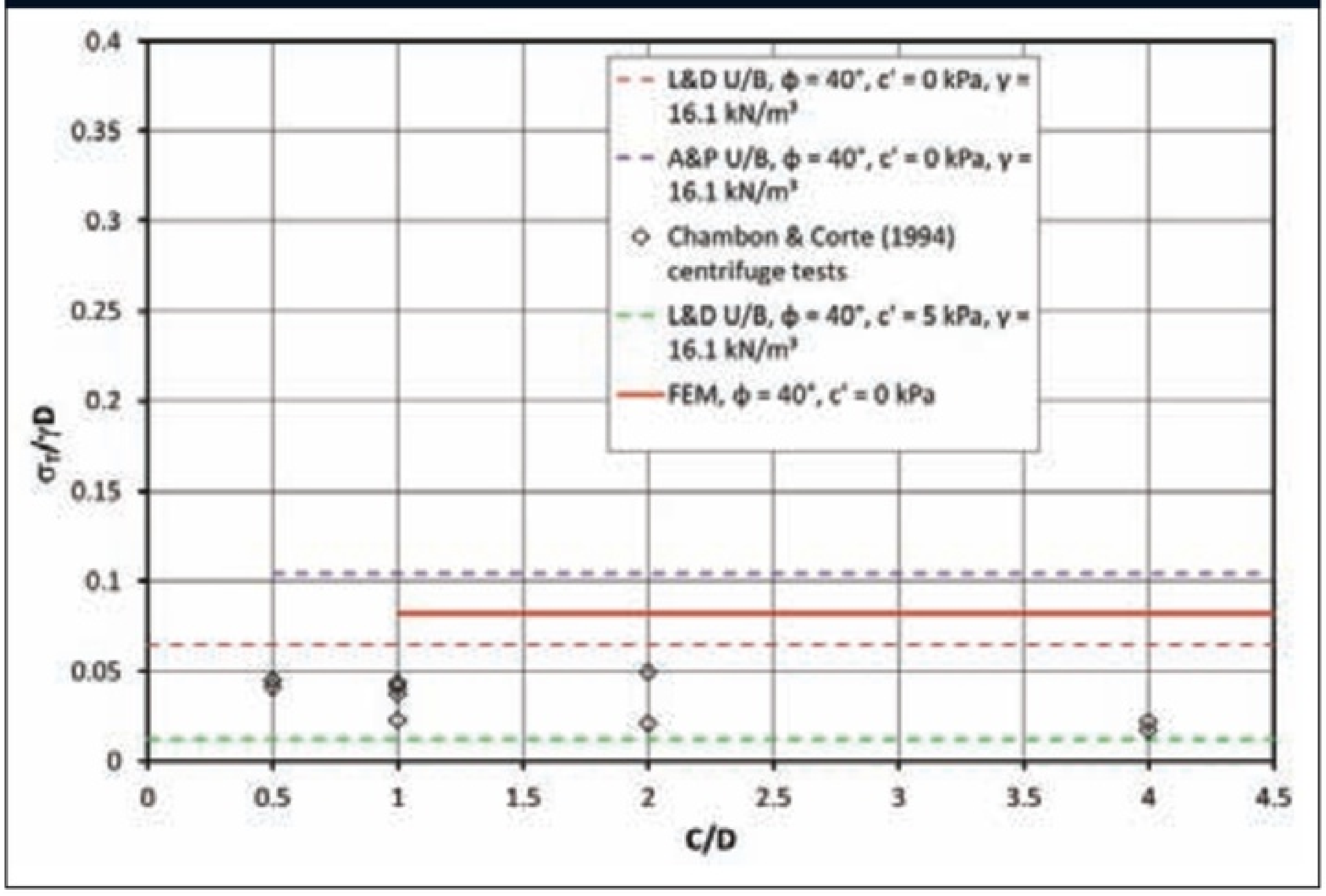



**Figure 9: Comparison of centrifuge tests by Chambon & Corté (1994) on dry cohesionless sand with limit state solutions (L&D U/B = Leca & Dormieux (1990) Upper Bound, A&P U/B = Atkinson & Potts (1977) Upper Bound), a limit equilibrium solution (A&K LE = Anagnostou & Kovári (1994)) and finite element models (FEM = FE models by Vermeer et al. (2002)).**




upper bound. Atkinson & Potts assumed a plane strain tunnel at both the upper and lower bounds (because this was what they were modelling in the 1g laboratory and centrifuge tests), essentially assuming an unlined infinitely long tunnel (see Figure 1 and Figure 2), whereas Leca & Dormieux assumed a three-dimensional failure at the face with unsupported length P = 0 for the upper bound, as shown in Figure 4, and three different stress fields for the lower bound, as shown in Figure 5, of which SI is usually the critical one.

Anagnostou & Kovári (1994, 1996) used a limit equilibrium solution they attributed to Horn (1961) and developed it to allow direct calculation of the required support pressure in any situation above or below the water table, with or without seepage. This solution is potentially much more useful for practical situations involving closed face tunnelling machines, although whether it represents the true collapse load is not guaranteed.

Figure 7 shows a comparison of Anagnostou & Kovári's limit equilibrium solution with the limit state solutions, for a dry cohesionless soil with Ø = 35°. Those of you familiar with Mair & Taylor's excellent 'Theme Lecture' paper at the 14th ICSMFE in 1997 will recognise Figure 3 as very similar to their comparison (their Figure 9). Interestingly, the support pressure calculated by the limit equilibrium solution is not completely constant and does increase very, very slightly with increasing *C/D*. The limit equilibrium solution is consistent with the limit state solutions in that the upper bounds (where the heading must fail) are below it, and the lower bounds (where the heading cannot fail) are above it.

**Comparison of analytical methods with centrifuge tests and finite element models**

Figure 8 and Figure 9 show comparisons between centrifuge tests by Messerli et al. (2010) and Chambon & Corté (1994) with the limit state and limit equilibrium solutions, and to finite element models by Vermeer et al. (2002).

Messerli et al.'s tests were on a uniform fine sand with angle of friction Ø = 33° and unit weight $\gamma$ = 17kN/m³. Analytical solutions are presented for comparison, with Ø = 33° and $\gamma$ = 17kN/m³ used as input parameters.

Vermeer et al. (2002) performed 25 finite element calculations and determined that the relationship between the soil weight stability number $N_\gamma$ ($= \sigma_T/\gamma D$) and angle of friction Ø is given by:

$$N_\gamma = 1/9\tan Ø - 0.05 = \sigma_T/\gamma D$$

This relationship was found to hold true as long as *C/D* > 1.

For Messerli et al.'s tests, Anagnostou & Kovári's limit equilibrium solution provides a good, though slightly conservative, estimate of the support pressure at failure. Atkinson & Potts's upper bound (below which the heading must fail) is in effect on the wrong side of the data points, but this is to be expected perhaps because the failure is three dimensional and their upper bound solution is for a 2D unlined plane strain tunnel. Leca & Dormieux's upper bound has a more realistic geometry, but for practical purposes the upper bound doesn't help decide what support pressure is required unless the lower bound brackets a small range of collapse loads, which it doesn't, as illustrated in Figure 7.

Chambon & Corté's centrifuge tests were on Fontainebleau sand. They estimated the angle of friction Ø was between 38 and 42° and the unit weight of the sand for the tests shown in Figure 9 was between 16.0 and 16.2kN/m$^3$. Anagnostou & Kovári's limit equilibrium solution is not shown in Figure 9 because the nomograms were used and these only show values for the coefficients up to Ø = 35°.

For Chambon & Corté's tests, both Atkinson & Potts's and Leca & Dormieux's upper bounds are on the wrong side. Chambon & Corté suggest in their paper that the Fontainebleau sand has a value of cohesion somewhere between 0 and 5kPa. If c' were closer to 5kPa, Leca & Dormieux's upper bound would drop below Chambon & Corté's data. The relationship based on finite element models (Vermeer et al., 2002) overestimates the support pressure at failure compared to Chambon & Corté's tests, but would be closer if a value of c' ≠ 0 were used.

**Summary**

The support pressure at failure for a drained soil is very sensitive to the value of cohesion c', but not so sensitive to the angle of friction Ø, although it does have an effect.

All methods suggest that the depth of the tunnel has negligible effect on the support pressure at failure. Vermeer et al. (2002) have found that the only exception to this is when a surcharge is applied to the surface above a shallow tunnel with $C/D < 2$ and Ø < 25°.

The support pressures required to prevent failure in drained c' – Ø soils may be normalised by the unit weight of the soil $\gamma$ and the diameter $D$. The proportion of $\gamma D$ required is of the order of 0.05 to 0.2 for most sands. In order to decide what face pressure to apply, the groundwater pressure must also be added to this value, and a factor of safety introduced. How this is done will be covered in the next issue when I'll go into more detail on the effect of seepage on stability calculations for closed face tunnel boring machines, and what the results tell us about how to operate these machines.

Both the relationships based on parametric finite element studies by Vermeer et al. (2002) and the limit equilibrium solution proposed by Anagnostou & Kovári (1994, 1996) seem to provide a reasonable, and slightly conservative, estimate of the collapse load when compared to centrifuge tests.

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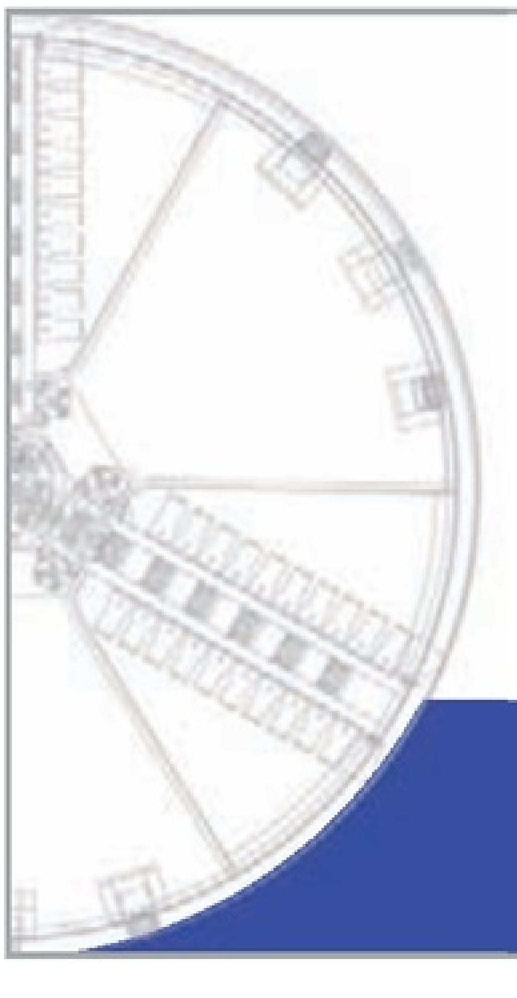

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