The Secretary,
The Institution of Civil Engineers.

DEAR SIR,

The correspondence from Mr T. Cameron Kenney in *Géotechnique* (11 : 1 : 54) uses a Paper by Osterberg and myself as a reference. I would like to add some comments that seem appropriate.

The shear strength components usually referred to as true cohesion and true angle of internal friction are not defined in the same way as the cohesion and friction components in the reference. The definitions used by Hvorslev and subsequent workers involves the assumption that at the same void ratio a given soil has the same cohesion. Any strength differences between specimens with different histories which result in different values of effective stress on the failure plane at the failure strain, but at the same void ratio on that plane, are then assumed due to the influence of the friction component of strength. It is on the basis of this assumption that the experimental determination of c_e and ϕ_e is carried out.

On the other hand, the definitions used in the reference are based on the manner in which the ability of a specimen to sustain deviator stress varies with imposed changes of effective stress—at any given strain condition. The effective stress is changed by means of controlled pore-pressure changes. Duplicate specimens with different histories are not required because a single specimen, either remoulded or undisturbed, is adequate. These definitions do not assume constant cohesion at constant void ratio. Comparison with the above shows that they are not similar. However, the Writer agrees that the use of the terms "cohesion" and "friction" in the reference may be confusing. It may help the reader to think of the definitions used in the reference as the "pore-pressure insensitive" and "pore-pressure sensitive" components rather than as cohesion and friction.

In view of the above, the manner in which Mr Kenney used this reference could cause some confusion. However, the Writer believes that strain should be included as an independent variable in any definition of the components of soil strength and agrees with the philosophy of Mr Kenney's comments and questions.

Yours faithfully,

JOHN H. SCHMERTMANN,
Assistant Professor of Civil Engineering.

Department of Civil Engineering,
University of Florida, U.S.A.
11 May, 1961.

The Secretary,
The Institution of Civil Engineers.

DEAR SIR,

I have read with great interest the Paper "Tunnel stress analysis" by H. D. Morgan (*Géotechnique*, 11 : 1 : 54).

To calculate the maximum bending moment in the lining, the Author has assumed that the lining is distorted into the shape of an ellipse; he has also assumed the active loading conditions on the lining.

In 1957, I developed a method of designing tunnel linings which does not require these assumptions to be made. The results are given by the following formulae.

Symbols

D = depth, vertical distance.
E, E_m = modulus of elasticity of a mass of soil.

E_b = modulus of elasticity of the lining.
F_b = area of section of lining per unit of length.
I_b = moment of inertia of lining per unit of length.
Ko = coefficient of earth pressure at rest.
W_b = section modulus of lining per unit of length.
γ = unit weight of a mass of soil.
μ = Poisson's ratio of the mass of soil.
ξ_2 = coefficient (= 0 for max shear stress on the extrados of lining as in the soil in the initial undeformed state of elastic equilibrium of the mass; = 1·0 for shear stress equal zero on the lining).

The radial stress p against the extrados of the lining is a function of $p(KoE_bI_bE\xi_2)$ and is given by:

$$p = \frac{1 + Ko}{2}\gamma D - \alpha_2 \frac{1 - Ko}{2}\gamma D \cos 2\theta$$

where
$$\alpha_2 = \frac{-1 + \xi_2 + 6(5 - \mu + 2\xi_2(2 - \mu))\dfrac{E_bI_b}{\eta_1{}^4r_1{}^3E}}{2 + \dfrac{6(5 - \mu)E_bI_b}{\eta_1{}^4r_1{}^3E}}$$

The bending moment M_{x-x} in the lining is given by:

$$M_{x-x} = -\eta_1{}^2r_1{}^2\frac{3(5 - \mu + (1 - \mu)\xi_2)}{3(5 - \mu) + \dfrac{\eta_1{}^4r_1{}^3E}{E_bI_b}}\frac{1 - Ko}{4}\gamma D \cos 2\theta$$

The total vertical earth pressure P_v against the tunnel lining is:

$$P_v = [1 + Ko + \tfrac{1}{3}(\alpha_2 + 2(1 - \xi_2))(1 - Ko)]\eta_1r_1\gamma D$$

The horizontal earth pressure P_h against the tunnel lining is given by:

$$P_h = [1 + Ko - \tfrac{1}{3}(\alpha_2 + 2(1 - \xi_2))(1 - Ko)]\eta_1r_1\gamma D$$

The extreme fibre stresses σ_R (from bending moment and axial force) are:—

For $\theta = 0°$:

$$\sigma_R = [1 + Ko + \tfrac{1}{3}(\alpha_2 + 2(1 - \xi_2))(1 - Ko)]\frac{\eta_1r_1\gamma D}{2F_b} \mp$$
$$\mp\ \eta_1{}^2r_1{}^2\frac{3(5 - \mu + (1 - \mu)\xi_2)}{3(5 - \mu) + \dfrac{\eta_1{}^4r_1{}^3E}{E_bI_b}} \cdot \frac{1 - Ko}{4W_b}\gamma D$$

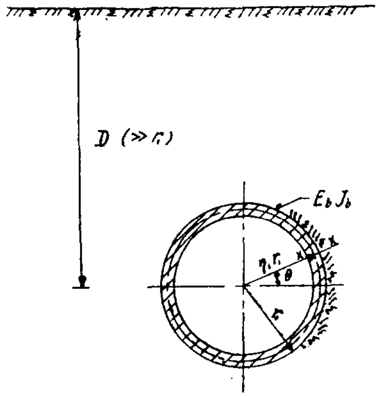
For $\theta = 90°$:

$$\sigma_R = [1 + Ko - \tfrac{1}{3}(\alpha_2 + 2(1 - \xi_2))(1 - Ko)]\frac{\eta_1r_1\gamma D}{2F_b} \pm$$
$$\pm\ \eta_1{}^2r_1{}^2\frac{3(5 - \mu + (1 - \mu)\xi_2)}{3(5 - \mu) + \dfrac{\eta_1{}^4r_1{}^3E}{E_bI_b}} \cdot \frac{1 - Ko}{4W_b}\gamma D$$

Having a preliminary clearance u_k between the ground and the tunnel lining, a reduction $\Delta\sigma_R$ in the stresses is obtained in the lining equal to:

$$\Delta\sigma_R = \eta_1\frac{E_mu_k}{(1 + \mu)F_b}$$

The radial deformation u_F of the lining is:



$$u_F = -\frac{5 + \xi_2 - \mu(1 + \xi_2)}{4\eta_1{}^4 r_1{}^3 E + 12(5 - \mu)E_b I_b}\,\eta_1{}^4 r_1{}^4 (1 - Ko)\gamma D \cos 2\theta$$

The relative circumferential sliding v between the lining and the ground is given by:

$$v = \left[\frac{(4 - 2\mu)(1 - \alpha_2) + (5 - \mu)\xi_2}{E} - \eta_1{}^4 r_1{}^3 \frac{2\alpha_2 + 1 - \xi_2}{12E_b I_b}\right] r_1 \frac{1 - Ko}{6}\gamma D \sin 2\theta$$

The resulting radial stress $\sigma_{r\ \mathrm{tot.}}$ in the vicinity of the tunnel is given by:

$$\sigma_{\theta\ \mathrm{tot}} = \frac{1 + Ko}{2}\gamma D - \left[1 - \frac{\dfrac{15 + 3\xi_2 - 3\mu(1 + \xi_2)}{5 - \mu}}{2 + \dfrac{6(5 - \mu)E_b I_b}{\eta_1{}^4 r_1{}^3 E}}\left[2\left(\frac{r_1}{r}\right)^2 - \left(\frac{r_1}{r}\right)^4\right] + \frac{2\xi_2}{5 - \mu}\left[3\left(\frac{r_1}{r}\right)^4 - (1 + \mu)\left(\frac{r_1}{r}\right)^2\right]\right]\frac{1 - Ko}{2}\gamma D \cos 2\theta$$

The resulting circumferential stress $\sigma_{\theta\ \mathrm{tot}}$ in the vicinity of the tunnel is:

$$\sigma_{\theta\ \mathrm{tot}} = \frac{1 + Ko}{2}\gamma D + \left[1 + \left(\frac{\dfrac{15 + 3\xi_2 - 3\mu(1 + \xi_2)}{5 - \mu}}{2 + \dfrac{6(5 - \mu)E_b I_b}{\eta_1{}^4 r_1{}^3 E}} + \frac{6\xi_2}{5 - \mu}\right)\left(\frac{r_1}{r}\right)^4\right]\frac{1 - Ko}{2}\gamma D \cos 2\theta$$

Yours faithfully,
KNUD ENGELBRETH

Sandvika, Norway.
18 May, 1961.

Note: The formulae given above are printed in the Norwegian publication *Teknisk Ukeblad, Oslo.*

The Secretary,
The Institution of Civil Engineers.

DEAR SIR,

Having been unable to discover similar previous work on the subject, I submit the following suggestions so that they may either be of assistance to others or that they may be corrected.