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DISCUSSIONS

EXTENDED DESIGN METHOD FOR MULTI-ROW STABILIZING PILES AGAINST LANDSLIDE*

Discussion by E. De Beer** and R. Carpentier***

It is with outmost interest that we read the papers by Ito, Matsui and Hong "Design methods for stabilizing piles against landslide—one row of piles," Soils and Foundations, Vol. 21, No. 1, pp. 21-37, and "Extended design method for multi-row stabilizing piles against landslide," Soils and Foundations, Vol. 22, No. 1, pp. 1～13.

In the second paper the authors give a very clear and easy method for taking into account the effect of multiple pile rows on the improvement of the stability of landslides, while at the same time the stability of the piles is checked.

In the computations a mobilization factor α_m, being the ratio of the lateral force to be delivered by the pile, to the maximum lateral force which can act on the pile is introduced, which is certainly a handy way of proceeding. The maximum lateral force on a pile is determined in function of the interval between the piles and the shearing strength characteristics of the soil around the pile, according to theoretical deductions presented by Ito and Matsui in their previous publications (1975, 1977, 1978). We already made some reserves concerning the absolute validity of these deductions (De Beer and Carpentier, 1977). This remains in our opinion a weak point in the practical validity of the results.

In the numerical applications for the case shown on Fig. 3, for the soil modulus below the sliding surface, and based on the N values a modulus $E_s=4\times10^3\,\text{kN/m}^2$ is introduced. This seems a rather very low value.

For the Shiranozawa landslide slope, strength parameters along the sliding surface of $c_u=9, 8\,\text{kN/m}^2$ and $\varphi=7°$ are introduced. Also these values are very low, and of course have a large influence when calculating the maximum effort which can possibly be acting on a pile. We have shown (De Beer and Carpentier, 1977) that when the shearing strength parameters of the soil are small, the degree of incertitude in the determination of the maximum possible force on the pile, is rather small.

Therefore it can be concluded that the extended design method for multi-row stabilizing piles against slides, described by Ito, Matsui and Hong, can be used with confidence in case of landslides in soils with small shearing strengths, but that in case of potential slides in stronger layers, limits of confidence for the maximum possible force on the piles should first be established. Starting from these limits the method described by the authors can be applied without any other change.

Therefore the publication is to be considered as a very useful tool for the design of slopes with stabilizing piles.

* By Tomio Ito, Tamotsu Matsui and Won Pyo Hong, Vol. 22, No. 1, March 1982, pp. 1-13.
** Professor Emeritus, Univ. Ghent, Belgium.
*** Chief Engineer, Belgian Geotechnical Institute, Zwijnaarde, Belgium.

Reference

10) De Beer, E. and Carpentier, R. (1977) : Discussion on "Methods to estimate lateral force acting on stabilizing piles," Soils and Foundations, Vol. 17, No. 1, 1977, pp. 68—82.

VERTICAL OSCILLATION OF EARTH AND ROCKFILL DAMS: ANALYSIS AND FIELD OBSERVATION*

Discussion by Tatsuo Ohmachi**

In formulating the equation of motion followed by the solution, the author employes a simplified stress-strain relationship expressed as

$$\sigma_z=\frac{E(z)}{1-\nu^2}\varepsilon_z\equiv\frac{G(z)}{\eta}\varepsilon_z \qquad (4b)'$$

where

$$\eta=\frac{1-\nu}{2} \qquad (4c)$$

As is written in the footnote, this relationship can be derived by assuming

$$\varepsilon_x=0 \text{ and } \sigma_y=0 \qquad (23)$$

Under these assumptions, the equation of motion in the vertical direction can be reduced to an uncoupled form expressed as

$$\rho\ddot{v}=\frac{1}{z}\frac{\partial}{\partial z}\left[\frac{G(z)}{\eta}z\frac{\partial v}{\partial z}\right]+G(z)\frac{\partial^2 v}{\partial x^2} \qquad (3)'$$

The uncoupled equation of motion, however, can be derived by employing the following relationship which has been commonly used for longitudinal vibration of bars

$$\sigma_z=E(z)\varepsilon_z\equiv\frac{G(z)}{\eta^*}\varepsilon_z \qquad (24)$$

where

$$\eta^*=\frac{1}{2(1+\nu)} \qquad (25)$$

Since the difference between Eq. (24) and Eq. (4 b)′ is only in their coefficient term, the formulation process described by the author is valid even for Eq. (24) if η is substituted by η^*. Moreover, due to small numerical difference between η and η^* over a whole range of Poisson's ratio, the difference in vibration characteristics such as natural frequencies and vibration mode shapes between the two cases will not be significant. Thus, the choice may depend on personal opinion for practical purposes.

An advantage of Eqs. (24)-(25) will be disclosed, if Eq. (3)′ is compared with the simplified equation of motion in the dam axis direction which is expressed as (Abdel-Ghaffar & Koh, 1981; Gazetas, 1981 b)

$$\rho\ddot{u}=\frac{1}{z}\frac{\partial}{\partial z}\left[G(z)z\frac{\partial u}{\partial z}\right]+E(z)\frac{\partial^2 u}{\partial x^2} \qquad (26)$$

Note that Eq. (26) can be directly derived from Eq. (3)′ by exchanging $E(z)$ and $G(z)$ if η is defined as the ratio $G(z)/E(z)$ which is the definition of η^*.

* By G. Gazetas, Vol. 21, No. 4, Dec. 1981, pp. 56-68.

** Associate Professor of Environmental Engineering, The Graduate School at Nagatsuta, Tokyo Institute of Technology, Nagatsuta-cho, Yokohama, Kanagawa.