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PULLOUT RESISTANCE OF BURIED ANCHOR IN SAND[i)]

Closure by Kozo Tagaya[ii)], Ronald F. Scott[iii)] and Hisao Aboshi[iv)]

The writers wish to express their appreciation to discussers who have taken an interest in the subject of the pullout resistance of a buried anchor in sand.

Professor Matsuo, one of the discussers, proposed the theoretical formula for the vertical uplift resistance of shallow footing and the convenient approximate formulas (Eqs. (20) to (23)) (Matsuo, 1967 and 1968). Elucidating that log R and log D have a linear relation, he has demonstrated the Meyerhof's proposal Eq. (1) yields a similar result (Fig. 16).

In the application of the estimation formulas to an actual anchor, it is important to evaluate the soil parameters which may be determined by laboratory element tests, in-situ tests, or synthetic judgement of these tests. However, on account of the scatter in data, limitation in information, etc., it is difficult to accurately evaluate soil parameters. Consequently, discussers have proposed the method of obtaining the soil parameters to estimate the anchor pullout resistance by the pullout test of a small-sized model at the actual site (plate uplift test). By this test, the soil failure pattern is similar to that of an actual anchor, and thus the variations in soil parameters can be reduced (Fig. 19). To be concrete, circular plates of 200 mm to 300 mm in diameter are buried horizontally into a relative depth (D_t/B) of an extent of 1-2, and the vertical pullout test is executed to measure the ultimate uplift resistance, R_u. In view of the small error in the evaluation of ϕ, C for the design of an actual anchor can be obtained through Eqs. (20) to (23) with the measured R_u.

Expressing the opinion about the comments by discussers, the authors will classify the problems in the future on the anchor pullout resistance as follows : -

1) *Estimation formula for the anchor pullout resistance*

Fig. 20 gives the relationships between D_f/B and $\frac{Q_u}{A\gamma G_L D_f}$ given by Eqs. (20) to (23) and Eq. (1) assuming $\phi=35.1$ and $C=0$. Also shown in the figure are the experimental values obtained by the centrifugal

i) Vol. 28, No. 3, September, 1988, pp. 114-130 (Previous discussion by M. Matsuo and T. Shogaki, Vol. 30, No. 1, March, 1990, pp. 180-184).

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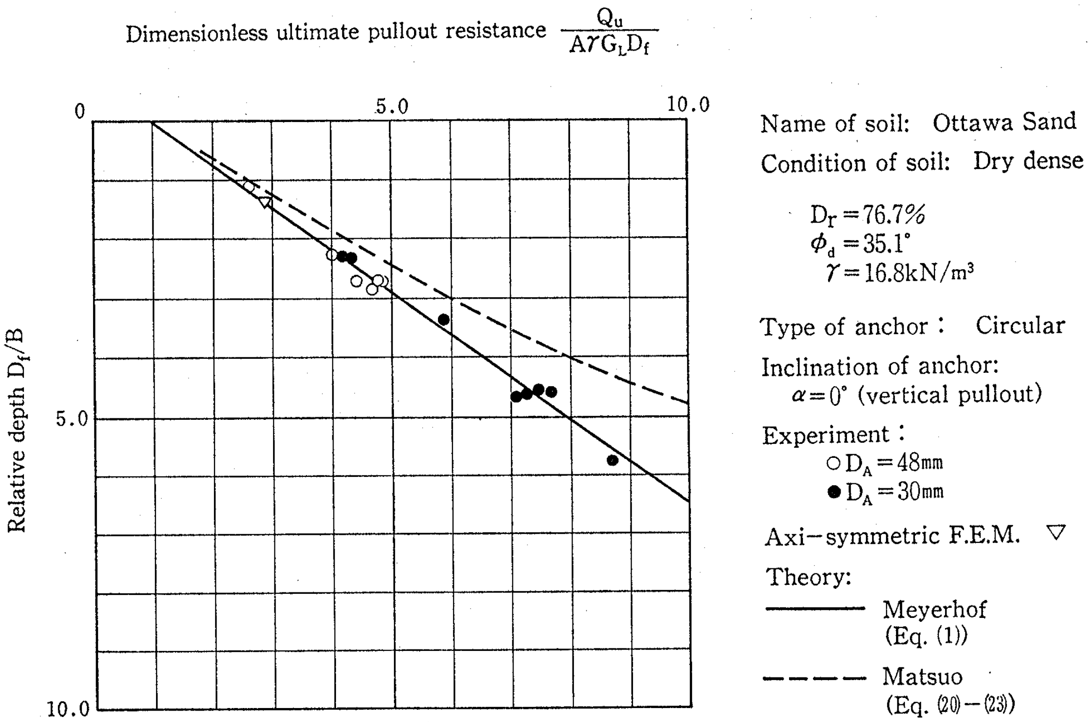

Fig. 20. Comparison of formulas by Meyerhof and Matsuo, present test data and F. E. M. analysis

technique and an analytical value by the axi-symmetric F. E. M. In this figure, Eqs. (20) to (23), Eq. (1) and the experimental value are consistent well with each other when the relative depth, D_f/B, is small. However, with an increase in the relative depth, Eqs. (20) to (23) offer a large dimensionless ultimate pullout resistance. Although the analytical value based on F. E. M. is given only for $D_f/B=1.13$, it is consistent well with the calculated value and experimental value. The cause of the difference in Eq. (1) and Eqs. (20) to (23) for the larger relative depth is not clear, but it may be due to the scale effect. Detailed investigations may be necessitated furthermore in the future.

2) *Plate uplift test*

The soil parameters can be obtained by an inverse analysis of the loading test with the scaled model(s) in addition to the element test with soil samples. With this method, the failure pattern must be similar to that of the prototype. Therefore, the scale effect has become an important problem. Tagaya, Scott and Aboshi (1988) demonstrated that, in the case of dry dense Ottawa Sand, the dimensionless ultimate pullout resistance, $\frac{Q_u}{A\gamma G_L D_f}$, and the dimensionless ultimate displacement, δ_{max}/D, become constant when $\gamma G_L D_A \geq 3.0\,N/cm^2$. With $\gamma G_L D_f$ becoming smaller than this value, the dimensionless ultimate pullout resistance becomes larger, and the dimensionless ultimate displacement becomes smaller. Namely, in the case of Ottawa Sand, both of the phenomena of the scaled model and the actual anchor may be regarded as the same, for $\gamma G_L D_A \geq 3.0\,N/cm^2$. When both of $\gamma G_L D_A$ of a scaled model and an actual anchor or that of a scaled model alone is smaller than $3.0\,N/cm^2$, the scale effect should be carefully examined. D_A at which $\gamma G_L D_A = 3.0\,N/cm^2$ with respect to Ottawa Sand is approximately 1800 mm at the $1\,g$ gravitational field.

The above discussion is concerned with sand (ϕ-material). In the case of the soil with cohesion, the problem is further complicated.

Both of the Meyerhof's generalized estimation formula (Meyerhof, 1973) and the

Matsuo's estimation formula are the functions of C and ϕ. However, it is a problem that which of C and ϕ predominates over the anchor pullout resistance. In the case of soil where C is predominant, the plate uplift test is considered to have a sufficient meaning, and in the case of soil where ϕ is predominant, the above mentioned scale effect must be taken into consideration.

Discussers also have pointed out importance of the scale effect in the plate uplift test.

As stated above, the authors have reported the evaluation method for the anchor pullout resistance of dry sand. However, there are several problems which must be investigated further in the future, as follows : -

(1) Pullout resistance of an inclined deep anchor

(2) Pullout resistance of an anchor buried in a saturated or partially saturated soil

(3) Pullout resistance in the case of C-material or C, ϕ-material

(4) Influence of a cyclic load

(5) Influence of an inclined ground surface

(6) Pullout resistance in the case of a regular or an irregular compound ground

These problems should be clarified by the sufficiently examined experimental method and analysis. In addition, it is necessary to confirm the phenomena of the prototype by actual anchor or large-sized model.

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INFLUENCE OF THE FOUNDATION WIDTH ON THE BEARING CAPACITY FACTOR[i)]

Closure by A. Hettler[ii)] and G. Gudehus[iii)]

Introduction

We thank the discussers for their detailed discussion including a lot of thorough experimental and theoretical work. They conclude from their test results, that at least for Toyoura sand at plain strain the influence of the grain size on the bearing capacity at the peak load cannot be neglected. This is explained by the strong anisotropic behaviour and the softening after the peak.

After investigating the bearing capacity factor it was surprising that the scale effect on N_γ could be explained by the pressure level alone. For piles in tension for example, a strong influence of the grain size on the pull-out load was observed (Hettler, 1982). According to the discussers' test results it seemed that the writers' founding was not general. But an additional calculation confirms that also for Toyoura sand under plain strain the influence of pressure level is dominating.

Some remarks on the formula for N_γ

It is known from the work of many authors that, depending on the assumptions, the bearing capacity factor N_γ can vary a lot. For example there is a big difference in N_γ for foundations with a rough or a smooth surface. Also the freedom to turn brings other results than in the case of a strongly guided foundation.

Eq. (9) is not the result of a theory with strict assumptions but some empirical consid-

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