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Numerical study on uplift bearing capacity of caisson type pile with reinforcing bars

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ABSTRACT: A caisson type pile foundation with reinforcing bars has been proposed by Matsuo and Ueno (1989). This foundation was developed for increasing uplift bearing capacity of transmission towers and others. In the present study, the mechanism of reinforcing effect of such types of foundation is numerically investigated by finite element analysis. The present analysis takes into account not only elastoplastic characteristics of geomaterials but also the stiffness of reinforcements and the frictional behavior between soil and reinforcement and soil and pile. The analytical results clarify the mechanism of the uplift bearing capacity and give information about the most suitable arrangement of reinforcements.

1 INTRODUCTION

Effective and economic pile foundations of the electric transmission towers have been expected with increase in size of the tower. For this purpose, a new caisson type pile foundation for transmission tower was developed and put into practice, in which reinforcing bars are driven from the pile into the ground (Matsuo and Ueno, 1989). The foundation should usually be stable against vertical load, uplift load and horizontal load. However, uplift load is considered to be the most serious in the foundation of transmission tower, so that model tests have been performed to confirm the efficiency of reinforcing bars against uplift load. It was also confirmed experimentally that reinforcing bars protruded off diagonally downward is more effective against uplift load than those protruded off horizontally (Tokyo Electric Power Company and Dai Nippon Construction, 1990).

One of the authors investigated the behavior of reinforced sand by carrying out triaxial tests of reinforced sand and the corresponding finite element analyses in which elastoplastic characteristics of sand, stiffness of reinforcements and slip between sand and reinforcements are considered (Nakai, 1992). In the present study, the authors aim to investigate numerically the reinforcing mechanism of such type of pile foundation against uplift load, by performing finite element analyses with different stiffness of reinforcements and different insertion direction of the reinforcements. The stress-strain behavior of sand is described by t_{ij}-sand model (Nakai, 1989), and the frictional behavior between sand and pile and sand and reinforcing bars by the elastoplastic joint elements (Nakai, 1985) in the same way as the previous study.

2 METHODS OF ANALYSIS

As shown in Fig. 1, the following four types of pile foundations with length of 6.0m and diameter of 1.5m were assumed paying attention to the previous model tests:

(a)case 0 : pile foundation without reinforcement
(b)case 1 : pile foundation with reinforcements which are protruded off horizontally from the pile
(c)case 2 : pile foundation with reinforcements which are protruded off diagonally downward from the pile
(d)case 3 : pile foundation with reinforcements which are protruded off diagonally upward from the pile

Here, in every case, the length of reinforcing bar is 1m, and six reinforcing bars in each level are set up at six levels with an equal spacing (1m). Figure 2 shows the finite element mesh for case 1 as an example. The ground is under axisymmetric condition, having the depth of 6.5m and the distance of 6.75m from the center, divided into 208

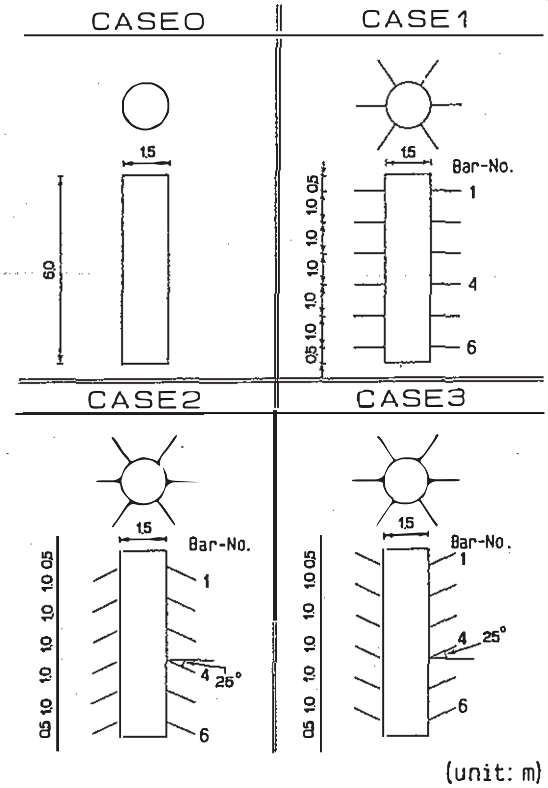


Fig. 1 Conditions of analysis

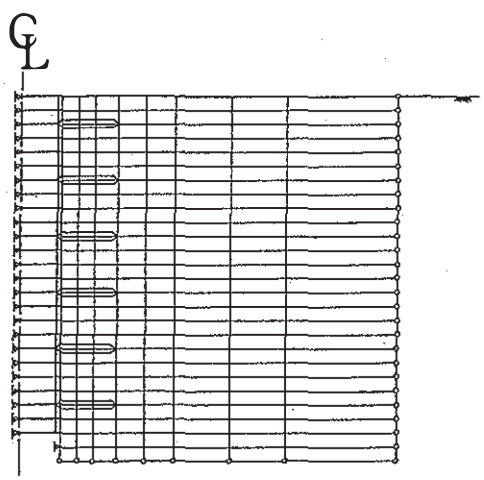


Fig. 2 Finite element mesh for case 1

Table 1 Values of soil parameter used in analysis

C_t	0.84×10^{-2}
C_e	0.60×10^{-2}
m	0.3
$R_f\equiv(\sigma_1/\sigma_3)_{f(comp.)}$	4.7
$D_f\equiv(d\varepsilon_v/d\varepsilon_1)_{f(comp.)}$	-0.6
α	0.85

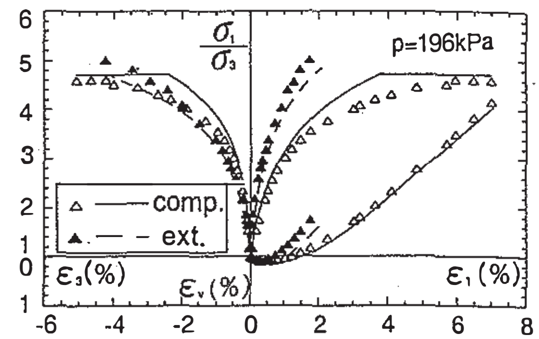


Fig. 3 Stress-strain relation on triaxial compression and extension tests on Toyoura sand

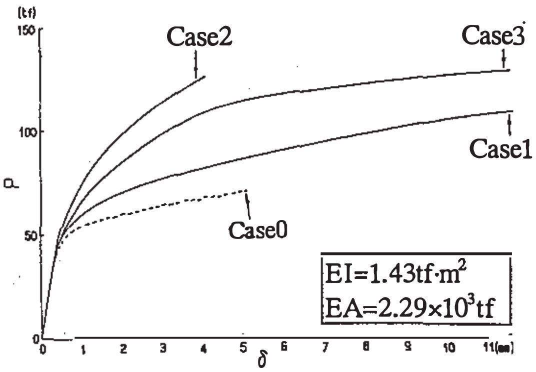


Fig. 4 Computed relation between uplift load and pile displacement

quadrilateral elements. The bottom and right-hand lateral boundaries are assumed to be fixed. The pile foundation which is 6.0m in length and 0.75m in radius is installed in the ground. The reinforcing bars in each level is replaced with a disc-shaped element, whose tensile and bending stiffnesses are assumed to be equivalent to those of mass of reinforcing bars with reference to the effective area of reinforcing bars. Joint elements are placed around the pile and both side of reinforcements to take into consideration the slip on the skin of the pile and the reinforcements and the skin resistance. This frictional behavior is described by elastoplastic joint property matrix (Nakai, 1985).

Medium dense Toyoura sand (G_s=2.65, e_0≈0.68, Dr=72%) is assumed as the material of ground. Upward uniform load p is increasingly applied on the top of the pile for every case. Elastoplastic constitutive model for sand named t_{ij}-sand model (Nakai, 1989) is used. This model is capable of description of the following typical characteristics of sand:

① Influence of intermediate principal stress on the deformation and strength of sand (unified description of stress-strain curves and failure condition in general three-dimensional stresses)

② Influence of stress path on the direction of plastic flow (stress path dependency of stress-dilatancy relation)

③ negative and positive dilatancy (volume contraction and successive volume expansion of sand under shear loading)

The values of soil parameters of t_{ij}-sand model for medium dense Toyoura sand are listed in Table 1, which can be determined from a drained conventional triaxial test after isotropic compression. Figure 3 shows the results (triangles) of triaxial compression and extension tests under constant mean principal stress together with the calculated curves, arranged with respect to principal stress ratio (σ_1/σ_3), principal strains (ε_1 and ε_3) and volumetric strain (ε_v). It can be seen that the model describes well the observed stress-strain behavior in three-dimensional stresses including dilatancy characteristics.

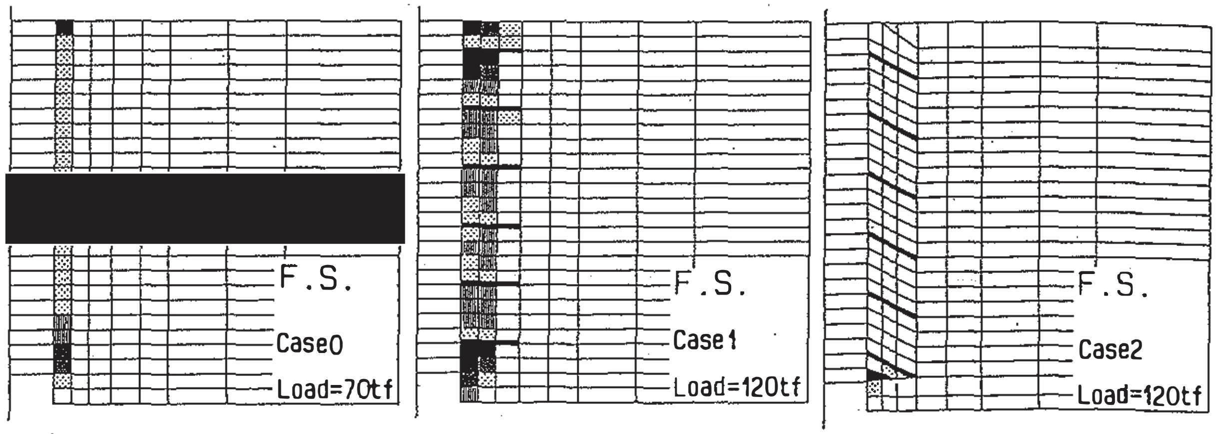
The initial stresses of the ground are calculated from the unit weight of sand (γ=1.58tf/m^3) and coefficient of earth pressure at rest (K_0=0.57). The value of K_0 can be theoretically determined from the constitutive model used. The stiffness of converted element for reinforcements is assumed as the bending stiffness EI=1.43tfm^2 and the tensile stiffness EA=2.29×10^{3}ft. To investigate the influence of the stiffness of reinforcements, the analyses in which either the bending stiffness or the tensile stiffness is assumed to be almost zero are also performed. The friction angle of the joint elements is assumed to be 40° in all the analyses. In computations, if a tensile stress is generated in soil element, the computation is not continued after then.

3 RESULTS AND DISCUSSIONS

We will firstly discuss about the results with EI=1.43tfm^2 and EA=2.29×10^{3}tf. Figure 4 shows the relation between uplift load p and upward displacement δ of the pile foundation. Here, uplift load dose not include the self-weight of the pile. The results without reinforcement (case 0) is described by broken curve. It is obvious that though uplift loads of reinforced piles (case 1 to case 3) are larger than unreinforced one (case 0), reinforcements protruded off diagonally downward (case 2) are the most effective to increase uplift resistance. Computed distributions of local factors of safety (F. S.), computed distributions of shear strain (ε_d) and computed distributions of volumetric strain (ε_v) in case 0 to case 2 are indicated in Figs. 5 to 7 respectively. Here, factor of safety (F. S.) is defined as F. S.=X_f/X (X_f:shear-normal stress ratio at failure; X: shear-normal stress in each soil element). The distributions of F. S. in every case varies widely as shown in Fig. 5. In unreinforced pile (case 0), F. S. of soil elements around pile alone decreases due to slippage between pile and soil at p=70tf. Although failure zones and zones of lower safety factor develop in case 1 at p=120tf, F. S. of soil elements in case 2 do not decrease at the same uplift load. The distributions of shear strain in Fig. 6 show the same tendency as the distributions of F. S. in Fig. 5. Figure 7 shows the distributions of zones where the soil element is compressive or expansive. It can be seen from this figure that the soil around the pile expands in case 0 and case 1 but contracts in case 2. Namely, the reinforcements in case 2 restrain the soil in the vicinity of reinforcements to expand, and strengthen the soil itself.

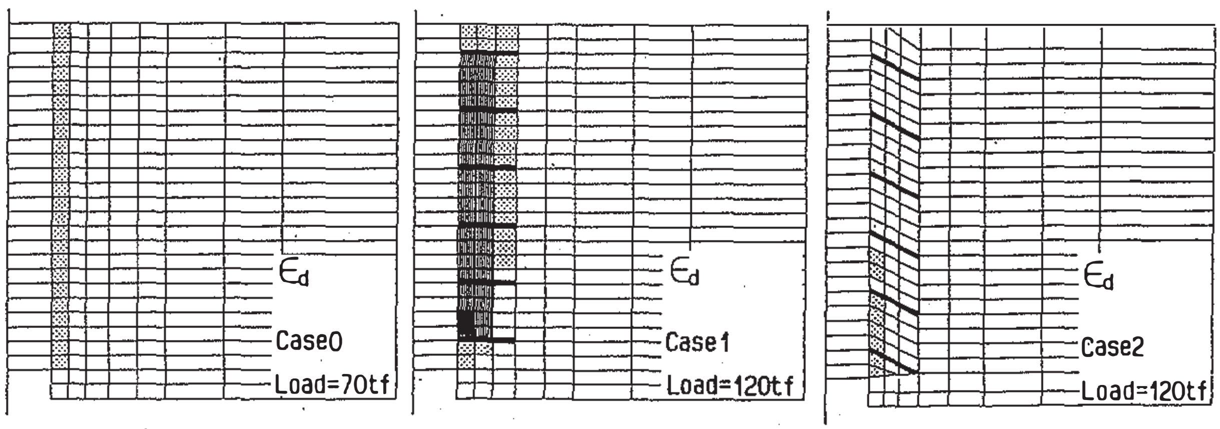
Figure 8 shows the distributions of axial force in reinforcements at p=120tf (tensile force is positive). The locations of the reinforcements (No.1, 4 and 6) are indicated in Fig. 1. Figure 9 shows the distributions of bending moment in the same reinforcements. Tensile axial force in every reinforcement in case 1 is not very large (even compression force is exerted in the lower reinforcements). On the other hand, large tensile force is exerted in all the reinforcements in case 2 where the reinforcements are protruded diagonally downward. We can see from the tendency of volumetric strain in Fig. 7 and axial force in Fig. 8 that the reinforcements in case 2 are acting as the tensile reinforcement. It is also seen from Fig. 9 that the bending moment of reinforcements in case 2 is not as large as those in case 1 and case 3. Through the above results, we can understand that even if the same reinforcements are used, the resistance of reinforced pile depends on the protruded direction of reinforcements.

For better understanding of the reinforcing mechanism, analyses for extreme cases are carried out. Figure 10 shows the relation between uplift load and upward displacement of the pile foundation for the case that though the tensile stiffness of reinforcements is the same as that in the above analyses (EA=2.29×10^{3}tf), the bending stiffness is almost zero (EI≈0.0tfm^2) like geotextiles. Comparing these results with the results in Fig. 4, we can see that the reinforcements protruded off diagonally downward (case 2) are the most effective in the same way as in Fig. 4, and uplift load-displacement curve of case 2 in Fig. 10 is the almost same as that in Fig. 4. Therefore, when the



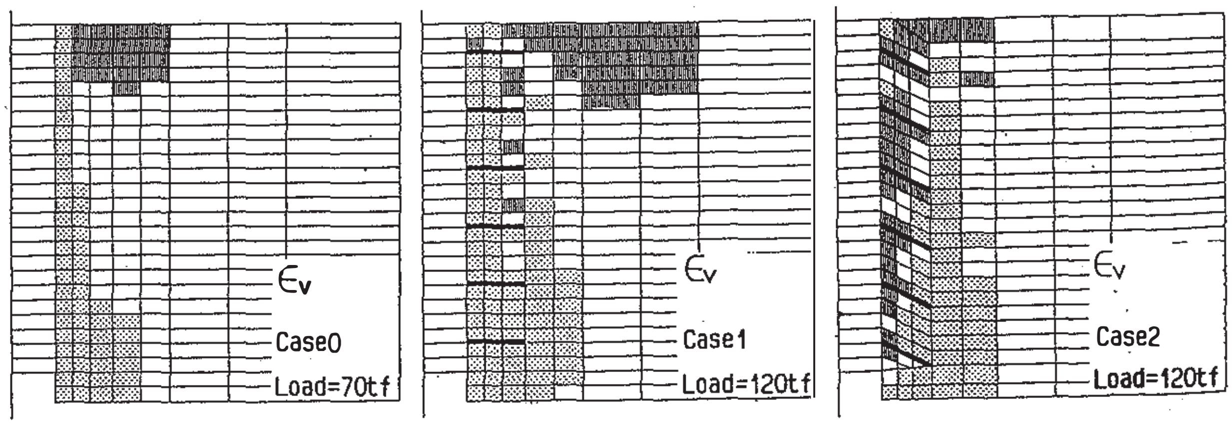
■ F.S.<1.0 ▩ 1.0≤F.S.<1.1 ▥ 1.1≤F.S.<1.2 □ 1.2≤F.S.<1.3 □ 1.3≤F.S.

Fig. 5 Computed distributions of local facters of safety in ground



■ 4.0%$\leq\varepsilon_d$ ▩ 2.5%$\leq\varepsilon_d$<4.0% ▥ 1.5%$\leq\varepsilon_d$<2.5% ░ 0.5%$\leq\varepsilon_d$<1.5% □ 0.5%$\leq\varepsilon_d$

Fig. 6 Computed distributions of shear strains in ground



▥ $0.1\times10^{-3}\leq\varepsilon_v$ □ $-0.1\times10^{-3}<\varepsilon_v<0.1\times10^{-3}$ ░ $\varepsilon_v\leq-0.1\times10^{-3}$

Fig. 7 Computed distributions of volumetric strains in ground

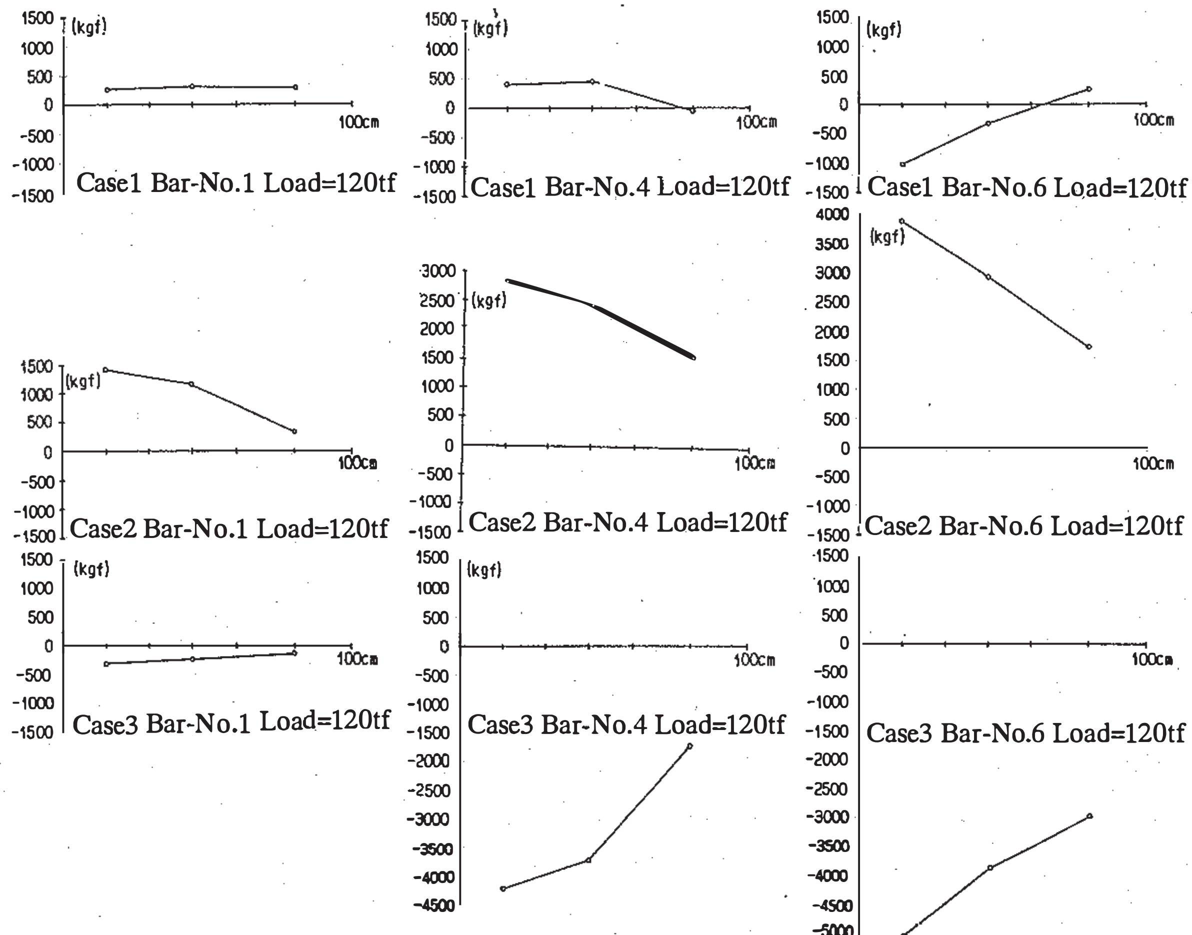


Fig. 8 Computed distributions of axial force in reinforcements

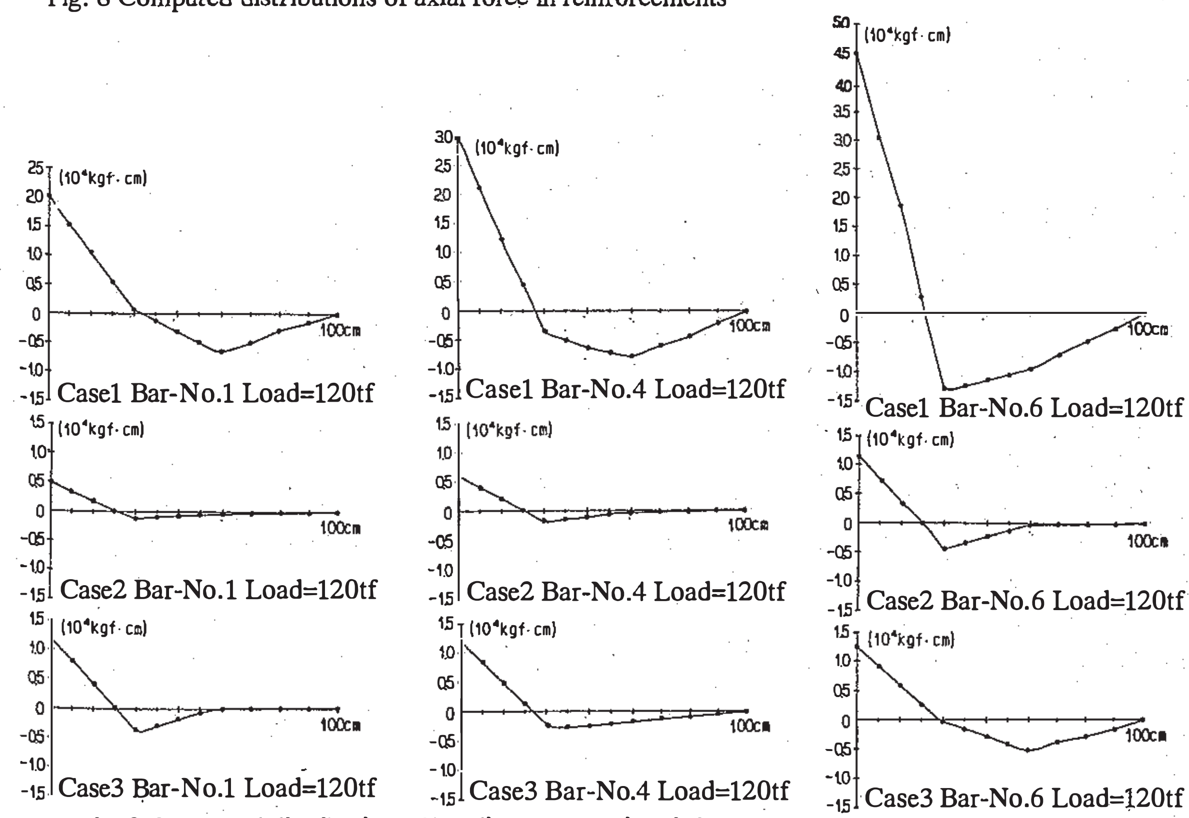


Fig. 9 Computed distribution of bending moment in reinforcements

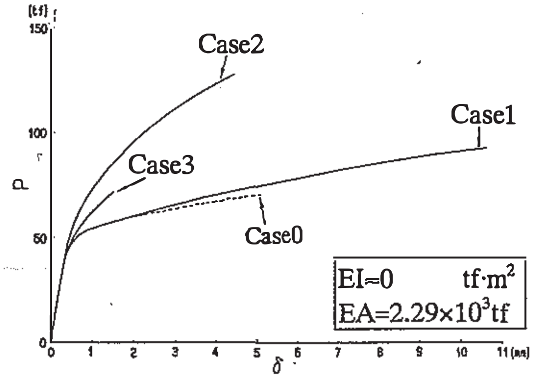


Fig. 10 Computed relation between uplift load and pile displacement in case of EI≈0.0tfm^2

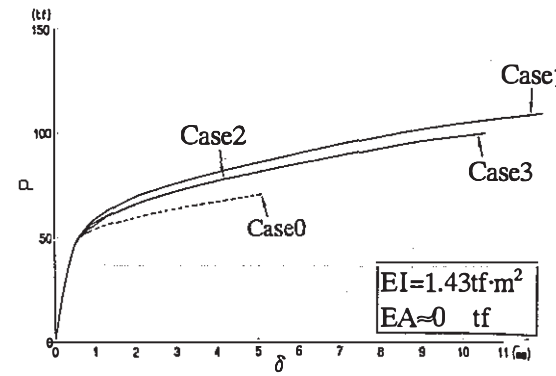


Fig. 11 Computed relation between uplift load and pile displacement in case of EA≈0.0tf

reinforcements are protruded off diagonally downward (case 2), they work as tensile reinforcements alone and the bending stiffness of the reinforcements hardly influence on the reinforcing effect. This can be understandable naturally from the fact that the roots of wood and grass, which have little bending stiffness, spread diagonally downward and stand against the uplift force caused by the wind. On the other hand, uplift load-displacement curve of case 1 is almost same as that of case 0 in case of EI≈0.0tfm^2. When the reinforcements are protruded off horizontally, reinforcing effect is expected not as tensile reinforcement but as bending reinforcement. Figure 11 shows the results in case that the tensile stiffness alone is almost zero (EA≈0.0tf, EI=1.43tfm^2), contrary to Fig. 10. Here, case 1 in which reinforcements are protruded off horizontally is the most effective in this figure. This is because the reinforcements function as the bending reinforcing as mentioned before. It can, however, be seen from Figs. 4, 10 and 11 that the reinforcements as the tensile reinforcing are more effective than those as the bending reinforcements.

4 CONCLUSIONS

The main results of the present numerical investigation on uplift bearing capacity of caisson type pile foundation are summarized as follows:

(1) The uplift bearing capacity and the slope of load-settlement curve of pile with reinforcing bars are larger than those of unreinforced pile. The reinforcing bars which are protruded off diagonally downward (case 2) are the most effective. These numerical results qualitatively correspond to the previous experimental results by Matsuo and Ueno (1989) and others.

(2) The mechanism of reinforcement depends on the protruded direction of reinforcements. Namely, the reinforcements protruded off diagonally down work as tensile reinforcement, the reinforcements protruded off horizontally as bending reinforcement. The tensile reinforcements is more effective than the bending reinforcements.

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