Limit state design in geotechnical engineering

INTERNATIONAL SYMPOSIUM

COPENHAGEN 26-28 MAY 1993

ISLSD 93 Vol 2 / 3

Sponsor: Danish Geotechnical Institute

May 1993

dgf-Bulletin **10**

DANISH GEOTECHNICAL SOCIETY

DANSK GEOTEKNISK FORENING

A Simple Method for the Design of Pile Groups

Singo GOSE, CTI Engineering Co., Ltd., Tokyo 113, JAPAN
Jyun YAMADA, Metropolitan Expressway Public Co., Tokyo, JAPAN
Akira SAITO, Metropolitan Expressway Public Co., Tokyo, JAPAN
Michinori HANKO, Metropolitan Expressway Public Co., Tokyo, JAPAN
Feng YI, CTI Engineering Co., Ltd., Tokyo 113, JAPAN

SYNOPSIS

In the current *Specifications for Highway Bridges* of Japan, no pile-soil-pile interaction is considered in the design of pile groups when the ratio of pile spacing, s/D, is larger than 2.5. An average value of the lateral load is taken as the load acted on each pile. However, the in-situ test results show that, as the lateral load acted on pile groups increases and large deformation occurs, strong interaction appears and the load acted on each pile is far from mean distributed. In this paper, a new concept that the stiffness of soil in the regions between the rows of piles degrades during deformation has been proposed. In-cooperating with Poulos' definition of interaction coefficient of piles and Randolph's equations, the new method gives satisfactory agreement of the load distributions with measured data.

INTRODUCTION

In recent years, there has a remarkable increasing tendency of the use of piles in civil engineering as the development in traffic system, especially the elevated expressways in urban areas where foundations are usually relatively poor. As the foundation of the pears of elevated expressways, piles are usually used in groups. The important roles taken by piles in the highway system make that not only the bearing capacity but also the lateral loading behavior of piles need to be widely understood. It is well known that, when the piles are used in groups, their behaviors are relatively different from that of single piles if the pile spacing is not larger enough. The decrease of bearing capacity and lateral resistance of pile groups is usually called as group efficiency.

The characteristics of laterally loaded pile groups have been investigated by individuals (Poulos, 1971; Tamaki, *et al., 1971*, Randolph, 1981, Shibata, *et al.*, 1989). Although these researchers have paid attentions to the group efficiency, no special attentions have been paid to the redistribution of load of laterally loaded pile groups. Researches by Poulos (1971) and Randolph (1981) based on the assumption that soil was an ideal, elastic, homogeneous, isotropic semi-infinite mass gave symmetric distribution of load in the loading direction among piles. In the current *Specifications for Highway Bridges* (1990) of Japan, an average value of the lateral load is taken as the load acted on each pile when the ratio of pile spacing, s/D, is larger than 2.5. The group efficiency also does not considered in this spacing condition in the design of pile groups. However, both in-situ (Hanko, *et al.*, 1992) and laboratory (Shibata, *et al.*, 1989; Takaki, *et al.*, 1991) test results of pile groups showed that even at the ratio of pile spacing of 2.5, strong group effect occurred. Loads acted on each pile are far from mean distributed. As the lateral load increases, the distributions of load in the loading direction change from symmetric to asymmetric. Loads acted on first row of piles were several times larger than that on last row of piles when the pile groups were near the state of yielding. This means that, if the pile groups are designed based on the current specifications, damages are possible to occur when the lateral loads acted pile groups are large enough, for instance, in case of large earthquake. Therefore, it is necessary to study the redistribution of loads among piles and make design according to this redistribution.

In this paper, a simple method for computing the load redistribution of pile groups has been proposed based on the assumptions that 1) the stiffness of soils in the regions between the rows of piles degrades during deformation, 2) the definition of interaction coefficient of Poulos is suitable and 3) the equations of Randolph are conditionally usable. It is presented by first, reviewing the elastic theory of the deformation of laterally loaded piles; second, extending the theory to elasto-plastic conditions; and finally giving the comparison of the predicted and measured data.

2. RANDOLPH'S ANALYSIS

2.1 Deformation of Single Piles

According to the theory of elastic beam, for a pile of bending rigidity $(EI)_p$, embedded in soil with coefficient of horizontal subgrade reaction k_H[1], there is a critical length of pile beyond which the pile behaves as if it is infinitely long. This critical length is given as

$$l_c \simeq 4\left(\frac{(EI)_p}{k_H}\right)^{1/4} \tag{1}$$

Hetenyi (1964) has given a solution, for a pile longer than the critical length and loaded by a lateral H and bending moment M, as

$$u = \sqrt{2}\frac{H}{k_H}\left(\frac{l_c}{4}\right)^{-1} + \frac{M}{k}\left(\frac{l_c}{4}\right)^{-2} \tag{2}$$

[1] k_H is defined here as the ratio of the load per unit length of pile to the local deflection, i.e., $N/m/m$

$$\theta = \frac{H}{k_H}\left(\frac{l_c}{4}\right)^{-2} + \sqrt{2}\frac{M}{k}\left(\frac{l_c}{4}\right)^{-3} \tag{3}$$

Dimensional analysis shows that, if the pile length is unimportant, any particular influence factor will be a function solely of the stiffness ratio E_p/G, where E_p is the effective Young's modulus of the pile

$$E_p = \frac{(EI)_p}{\pi r_0^4/4} \tag{4}$$

and the Poisson's ratio ν for the soil (r_0 is pile radius). For a soil of shear modulus G, the effect of the variation of Poisson's ratio ν can be taken account by the use of a new parameter G^* which is defined as

$$G^* = G\left(1 + \frac{3}{4}\nu\right) \tag{5}$$

For homogeneous soil profile and soil deposits with stiffness proportional to depth, the responses of a single pile loaded by lateral load H and moment M have been analyzed by Randolph (1981) by means of finite element method. According the results of FEM analysis, the deformation of pile at ground level was modeled by Randolph (1981) as

$$u = \frac{(E_p/G_c)^{1/7}}{\rho_c G_c}\left[0.27H\left(\frac{l_c}{2}\right)^{-1} + 0.3M\left(\frac{l_c}{2}\right)^{-2}\right] \tag{6}$$

$$\theta = \frac{(E_p/G_c)^{1/7}}{\rho_c G_c}\left[0.3H\left(\frac{l_c}{2}\right)^{-2} + 0.8(\rho_c)^{1/2}M\left(\frac{l_c}{2}\right)^{-3}\right] \tag{7}$$

where

$$G_c = G^*_{l_c/2} \tag{8}$$

$$\rho_c = \frac{G^*_{l_c/4}}{G_c} \tag{9}$$

and

$$l_c = 2r_0\left(\frac{E_p}{G_c}\right)^{2/7} \tag{10}$$

The definition of G_c and ρ_c are as shown in Fig. 1.

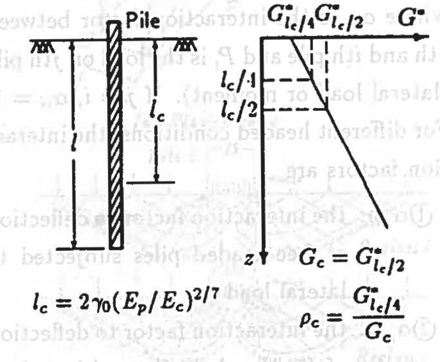


Figure 1: Definition of G_c and ρ_c

Eqs. 6 and 7 are the deformations of piles had general headed conditions. For free-headed pile, the deformation at ground level may be readily calculated by these equations for any combinations of H and M. For fixed-headed piles, the rotational angle at pile head is 0. Thus, the fixing moment is gotten from Eq. 7 as

$$M_f = -0.375\left(\frac{1}{\rho_c^{1/2}}\right)H\left(\frac{l_c}{2}\right) \tag{11}$$

2. Deformation of Group Piles

The solution of single piles was extended to closely spaced group piles by the use of interaction factors (Poulos, 1971). The interaction factor is defined as the ratio of additional deformation due to adjacent pile to the deformation of pile due to its own loading. If k represents the stiffness (load divided by deformation, H/u) of a single isolated pile, the deformation of the ith pile in a group of n piles may be expressed as

$$\delta_i = \frac{1}{k}\sum_{j=1}^{n} \alpha_{ij} P_j \qquad (12)$$

where α_{ij} is the interaction factor between jth and ith pile and P_j is the load on jth pile (lateral load or moment). If $j = i$, $\alpha_{ii} = 1$. For different headed conditions, the interaction factors are

①$\alpha_{\rho H}$: the interaction factor to deflection of free-headed piles subjected to lateral load

②$\alpha_{\rho M}$: the interaction factor to deflection of free-headed piles subjected to moment loading

③$\alpha_{\theta H}$: the interaction factor to rotation of free-headed piles subjected to lateral load ($\alpha_{\rho M}=\alpha_{\theta H}$ from the reciprocal theorem)

④$\alpha_{\theta M}$: the interaction factor to rotation of free-headed piles subjected to moment loading

⑤$\alpha_{\rho F}$: the interaction factor to deflection of fixed-headed piles subjected to lateral load

According to the results from FEM analysis of fixed-headed piles, the interaction factor to deflection of fixed-headed piles may be expressed as

$$\alpha_{\rho F} \simeq 0.6\rho_c \left(\frac{E_p}{G_c}\right)^{1/7} \frac{r_0}{s}(1+\cos^2\psi) \qquad (13)$$

where s is the pile spacing and ψ is the angle between the line joining the pile centres and the direction of loading (Fig. 2).

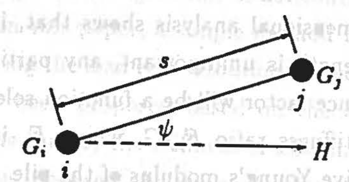


Figure 2: Definition of s and ψ

If the value given by Equation 13 is larger than 0.5, it is necessary to replace $\alpha_{\rho F}$ by $1\text{-}(4\alpha_{\rho F})^{-1}$. For free-headed piles, there has

$$\alpha_{\rho H} \simeq 0.5\rho_c \left(\frac{E_p}{G_c}\right)^{1/7} \frac{r_0}{s}(1+\cos^2\psi) \quad (14)$$

For other three interaction factors, the following relations exist

$$\alpha_{\rho M} = \alpha_{\theta H} = \alpha_{\rho H}^2 \qquad (15)$$

$$\alpha_{\theta M} = \alpha_{\rho H}^3 \qquad (16)$$

3. EXTENSION OF RANDOLPH'S METHOD

In Randolph's method, the consideration of the interaction of pile groups followed the ideal of Poulos (1971). While Poulos' theory is based on the assumption that soil is elastic material or the deformation of pile groups is so small that soil behaves as elastic material. Therefore, the method gives the symmetric distribution of loads in loading direction. This is obviously shown in the interaction factors $\alpha_{\rho F}$ or $\alpha_{\rho H}$ that gives the same values in pull direction ($\psi = 0°$) and push direction ($\psi = 180°$). However, the in-situ test results (Saitou, *et al.*, 1993) show

that the load distribution of pile groups in loading direction changes from approximately symmetric to asymmetric as the applied load increases from small to large. The loads acted on the first row of piles were as large as several times of that acted on the last row of piles when the laterally loaded pile groups came to yielding state. The asymmetric distribution of loads in loading direction is considered as the change of resistance from soil. When the pile groups bear a large amount of load and the deformation of group piles is large enough, there yields a certain range near ground surface in which soil behaves as plastic material. While below this zone, soil is still in elastic state. Therefore, soil profile is separated into plastic range and elastic range. The movement of soil around pile in different zone is as shown in Fig. 3. In

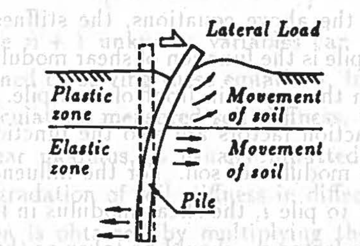
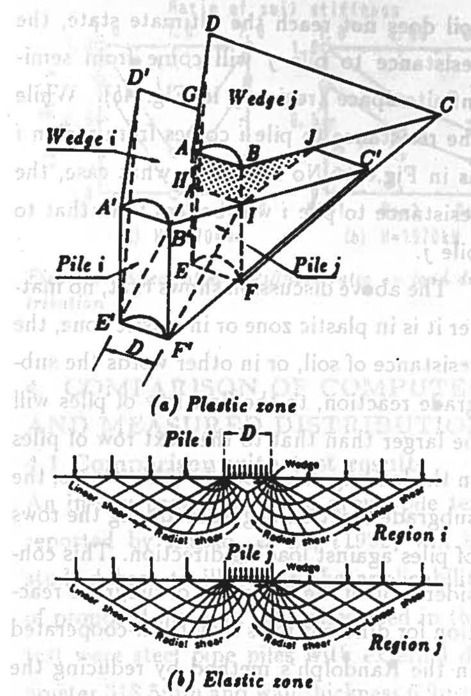


Figure 3: Movement of soil around pile

the plastic range, soil deforms as a wedge-shaped mass (Fig. 4*a*). The ultimate resistance from soil is equal to the passive earth pressure from the wedge-shaped mass. It can be seen from Fig. 4*a* that, if the pile spacing is not large enough, the wedge-shaped soil mass, $A'B'C'D'E'F'$, before pile i partly heaps with that, $ABCDEF$, before pile j. In most of the case, the heads of piles



(a) Plastic zone

(b) Elastic zone

Figure 4: Failure pattern

in the group are connected by a rigid footing. Therefore, the head and near head deformations of each pile in the group are the same. In other words, the deformations of wedges $A'B'C'D'E'F'$ and $ABCDEF$ can be considered as the same. Because $ABJGHI$ laps over $ABCDEF$, the ultimate resistance of soil deposit to pile i will be less than that to pile j.

On the other hand, in the elastic range, the ultimate state of deformation of the soil in front of the pile is similar to that of infinite long loading plate in vertical direction as shown in Fig. 4*b*. Similarly, if the pile spacing is not large enough, the failure zones will partly lap. If the deformation of soil does not reach the ultimate state, the resistance to pile j will come from semi-infinite space (region j in Fig. 4b). While the resistance to pile i comes from region i as in Fig. 4b. No matter in what case, the resistance to pile i will be less than that to pile j.

The above discussion shows that, no matter it is in plastic zone or in elastic zone, the resistance of soil, or in other words the subgrade reaction, to the first row of piles will be larger than that to the next row of piles in the loading direction. In other words, the subgrade reaction degrades among the rows of piles against loading direction. This consideration of the difference of subgrade reaction for different piles may be in-cooperated in the Randolph's method by reducing the shear moduli of the soil in the regions between the rows of piles.

By use of Eqs. 6 and 7, Eq. 12 can be rewritten, for free-headed piles, as

$$\delta_i = \frac{1}{(k_{\rho H})_i}\sum_{j=1}^{n}(\alpha_{\rho H})_{ij}H_j + \frac{1}{(k_{\rho M})_i}\sum_{j=1}^{n}(\alpha_{\rho M})_{ij}M_j \qquad (17)$$

$$\theta_i = \frac{1}{(k_{\theta H})_i}\sum_{j=1}^{n}(\alpha_{\theta H})_{ij}H_j + \frac{1}{(k_{\theta M})_i}\sum_{j=1}^{n}(\alpha_{\theta M})_{ij}M_j \qquad (18)$$

For fixed headed piles

$$\delta_i = \frac{1}{(k_{\rho F})_i}\sum_{j=1}^{n}(\alpha_{\rho H})_{ij}H_j \qquad (19)$$

$$\theta_i = 0 \qquad (20)$$

where

$$\frac{1}{(k_{\rho H})_i} = 0.27\left[\frac{(E_p/G_c)^{1/7}}{\rho_c G_c}\left(\frac{l_c}{2}\right)^{-1}\right], \qquad (21)$$

$$\frac{1}{(k_{\rho M})_i} = 0.3\left[\frac{(E_p/G_c)^{1/7}}{\rho_c G_c}\left(\frac{l_c}{2}\right)^{-2}\right], \qquad (22)$$

$$\frac{1}{(k_{\theta H})_i} = \frac{1}{(k_{\rho M})_i} \qquad (23)$$

$$\frac{1}{(k_{\theta M})_i} = 0.8\left[\frac{(E_p/G_c)^{1/7}}{\rho_c G_c}(\rho_c)^{1/2}\left(\frac{l_c}{2}\right)^{-2}\right], \qquad (24)$$

and

$$\frac{1}{(k_{\rho F})_i} = \left[\frac{(E_p/G_c)^{1/7}}{\rho_c G_c}\left(0.27 - \frac{0.1125}{\rho_c^{1/2}}\right)\left(\frac{l_c}{2}\right)^{-1}\right], \qquad (25)$$

In the above equations, the stiffness of each pile is the function of shear modulus of soil in the region in front of that pile. The interaction factors are also the function of shear modulus of soil. For the influence of pile j to pile i, the shear modulus in interaction factor α_{ij} should be taken as the one corresponding to pile i because this influence is caused by the deformation of pile j. While the deformation of pile j depends on the stiffness of soil corresponding to pile j as well as the its own stiffness. If $i \neq j$, $\alpha_{ij} \neq \alpha_{ji}$.

For pile groups, if the heads of the piles are connected, the head deformation of each pile will be the same. If P represents the general load, $i.e.$, lateral load H or moment M, and δ the general deformation, $i.e.$, deflection u or rotation of pile θ, there has

$$\delta = \delta_i = \frac{1}{k_i}\left(P_i + \sum_{\substack{j=1\\ j\neq i}}^{n}\alpha_{ij}P_j\right)(i = 1, 2, \cdots n) \qquad (26)$$

The total load P acted on the group piles is distributed to each pile,

$$P = \sum_{i=1}^{n} P_i \qquad (27)$$

Eqs. 26 and 27 give $n+1$ equations for $n+1$ unknown variables as

$$\begin{pmatrix} \frac{1}{k_1} & \frac{\alpha_{12}}{k_1} & \cdots & \frac{\alpha_{1n}}{k_1} & -1 \\ \frac{\alpha_{21}}{k_2} & \frac{1}{k_2} & \cdots & \frac{\alpha_{2n}}{k_2} & -1 \\ \vdots & \vdots & \ddots & \vdots & \vdots \\ \frac{\alpha_{n1}}{k_n} & \frac{\alpha_{n2}}{k_n} & \cdots & \frac{1}{k_n} & -1 \\ 1 & 1 & \cdots & 1 & 0 \end{pmatrix}\begin{Bmatrix} P_1 \\ P_2 \\ \vdots \\ P_n \\ \delta \end{Bmatrix} = \left\{\ 0\ \ 0\ \ \cdots\ \ 0\ \ P\ \right\}^T \qquad (28)$$

The $n+1$ unknown variables can be obtained by solving these equations. In a real calculation, measured soil stiffness, usually shear modulus, is usually inputted. The degradation of soil stiffness in different region is obtained by multiplying the measured soil stiffness profile by a coefficient. The effect of the ratio of soil stiffness on load distribution is illustrated in Fig. 5 for a 3 x 3 pile groups. In the figure, the piles that the line joining their centres paralleled load direction are called as *Line* piles, represented by L. While *Row* piles mean that the line joining their centres is perpendicular to load direction. It is obvious that the degradation of soil stiffness affects the load distribution significantly.

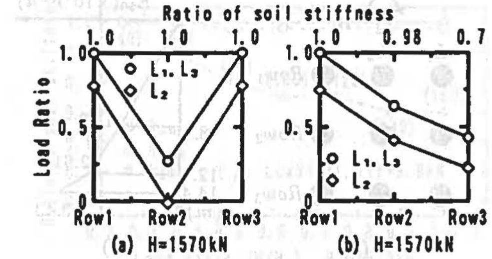


Figure 5: Effect of soil stiffness ratio on load distribution

4. COMPARISON OF COMPUTED AND MEASURED DISTRIBUTION

4.1 Comparison with test results

An in-situ prototype 3 x 3 group pile test reported by Hanko, *et al.*, (1992) will be studied here to illustrate the applicability of proposed method. The piles used in this test were steel pipe piles with external diameter 318.5mm and wall thickness 6.9mm. They were driven into a deposit with sandy and clayey layers to a penetration of 14.4m. The Young's modulus of pile was $E = 2.06\times 10^8 kPa$. Pile heads were connected by a 0.8m thick and 2.4 x 2.4m reinforced concrete footing. The arrangement of piles is shown in Fig. 6a. The simplified profile of soil stiffness based on penetration tests is shown in Fig. 6b.

The loads acted on each pile head were back-calculated at each load step based on the measured axial strains on both sides of piles. The typical results are shown in Fig. 7 that illustrates the average loads in each *row* normalized to that in the first *row*. It is obvious that the distribution of loads at pile heads changes from approximately symmetric to asymmetric as the deformation of pile

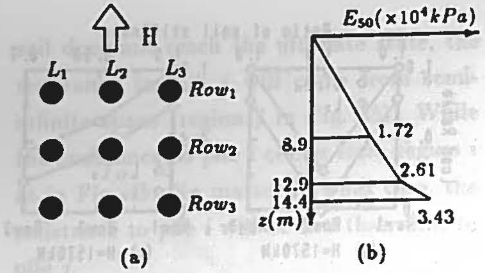


Figure 6: (a). Arrangement of piles and (b). Profile of E_{50}

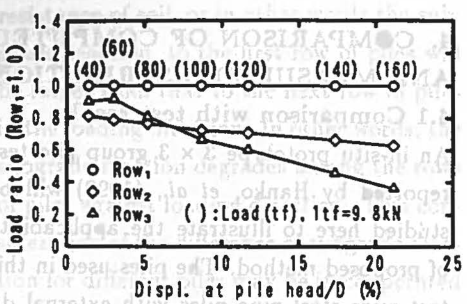


Figure 7: Load distribution along loading direction

head increases to a large value.

The load distribution on pile heads may be calculated based on Eq. 28. The distribution of soil stiffness shown in Fig. 6b is used. For the pile groups were loaded till an ultimate state of deformation, strong nonlinearity was shown in soil properties. This nonlinearity should be considered in the computation. Because the stress strain relationships of the soils do not known, the nonlinearity of soil stiffness is considered by multiplying a factor to the stiffness shown in Fig. 6b. For each loading step, the magnitude of the factor is such chosen, by try and error, that the calculated deformation of pile head agrees with that obtained from in-situ test. The calculated load distributions at loads 392, 785, 1177 and 1570 kN are illustrated in Fig. 8. It can be seen that, although divergence exists between theoretical results and test data when the applied loads are not large enough, the computation generally can give satisfactory predictions to the load distribution, especially at large deformation (ultimate state). It is considered that the divergence is caused by over-consideration of the interaction of piles at elastic condition. On the view point of design, the load distribution of pile groups at small and middle level deformation will not be as important as that at large deformation. Therefore, the above method will be good enough for computing the load distribution of pile groups at limited state.

The change of the ratio of soil stiffness as a function of applied load is illustrated in Fig. 9. It is obvious that the degradation of soil stiffness between the rows of piles increases as the applied load increases. This is because that the larger the applied loads or the larger the deformation, the deeper the plastic zone and the larger the plastic wedge, further the larger the lapped area and the larger the degradation of soil resistance to the behind rows of piles. The fact that at load ratio $H/H_y \leq 0.875$ (where H_y is yielding load), the ratios of soil resistance of 1:0.98:0.7 can satisfactorily predict the test results, indicates that it is necessary to consider the degradation of soil stiffness in the regions between rows of piles when consider the group effect of pile groups at large deformation state.

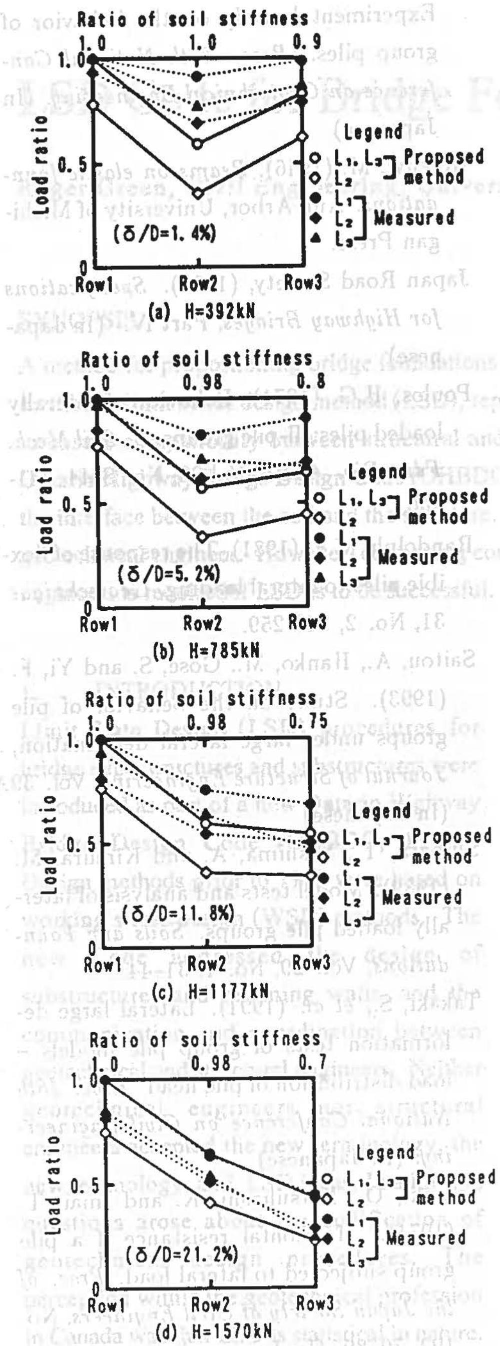


Figure 8: Load distribution ratio

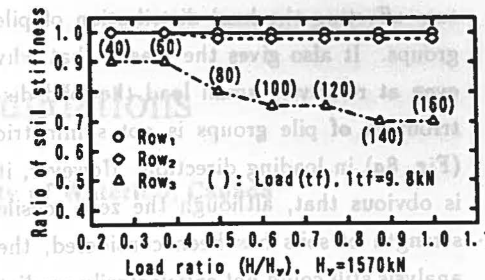


Figure 9: Change of the ratio of soil stiffness

4.2 Comparison with FEM results

Three dimensional finite element analysis has been done at a load 1570kN (yielding load). In the analysis, it is assumed that soils are elastic materials but with zero tensile strength. Soil moduli used are corresponding to E_{50}. The outcomes together with test data and the results from above proposed method are illustrated in Fig. 10. It can be seen that, with the consideration

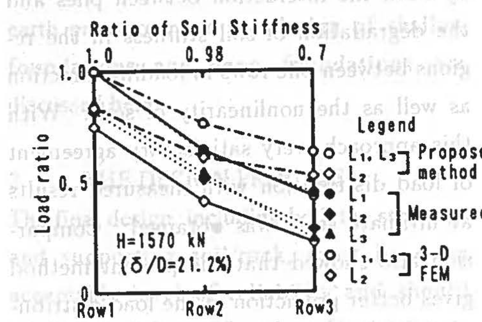


Figure 10: Comparison with FEM results

of zero tensile strength of soil, the load distribution of pile groups shows asymmetric along loading direction. This means that the anisotropic feature of soils in compression and tension is one of the main factors affecting the load distribution of pile groups. It also gives the reason that why even at relatively small load the load distribution of pile groups is not symmetric (Fig. 8a) in loading direction. However, it is obvious that, although the zero tensile strength of soils has been considered, the analysis still could not satisfactorily predict the test results. This means other factors, which influence the load distribution of pile groups at large deformation, exist. One of the factors is the degradation of soil stiffness in compression region as shown in the proposed method.

5. CONCLUSIONS

A simple method for prediction of load distribution of pile groups has been proposed in this paper. It was considered that the load distribution of pile groups was caused by both the interaction between piles and the degradation of soil stiffness in the regions between pile rows in loading direction as well as the nonlinearity of soils. With this approach, very satisfactory agreement of load distribution with measured results at ultimate state was obtained. Comparison also showed that the present method gives better prediction of the load distribution than three dimensional finite element analysis which assumes that soils are elastic materials but with zero tensile strength.

6. REFERENCE

Hanko, M., Gose, S. and Seki, K. (1992). Experimental study on the behavior of group piles. *Proc. 27th National Conference on Geotechnical Engineering.* (In Japanese)

Hetenyi, M. (1946), *Beams on elastic foundations.* Ann Arbor, University of Michigan Press.

Japan Road Society, (1991). *Specifications for Highway Bridges,* Part IV. (In Japanese)

Poulos, H.G. (1971), Behavior of laterally loaded piles: II-pile groups. *J. Soil Mech. Fdn. Div. ASCE,* Vol 98, No. SM4, 341-360.

Randolph, M.F. (1981), The response of flexible piles to lateral loading, *Géotechnique* 31, No. 2, 247-259.

Saitou, A., Hanko, M., Gose, S. and Yi, F. (1993). Study on the behavior of pile groups under large lateral deformation, *Journal of Structure Engineering,* Vol. 39A. (In Japanese)

Shibata, T., Yashima, A. and Kimura, M. (1989). Model tests and analysis of laterally loaded pile groups. *Soils and Foundations,* Vol. 29, No. 1, 31-44.

Takaki, S., *et al.* (1991). Lateral large deformation tests of group pile models – load distribution of pile head. *Proc. 46th National Conference on Civil Engineering.* (In Japanese)

Tamaki, O., Mitsuhashi, K. and Imai, T. (1971). Horizontal resistance if a pile group subjected to lateral load. *Proc. of the Japan Society of Civil Engineers,* No. 192, 79-89. (In Japanese)

LSD Code for Bridge Foundations

Roger Green, Civil Engineering, University of Waterloo, Canada

SYNOPSIS

A method for proportioning bridge foundations and retaining walls of bridges is described. The method, a limit states design method (LSD), replaces a factor of safety design method to attempt to ensure compatibility between structural and geotechnical design procedures. LSD, of the Ontario Highway Bridge Design Code (OHBDC), clarifies the calculation procedures describing the interface between the soil and the structure. No new technical procedures are required of the geotechnical engineer. However, continuing communication between geotechnical and structural engineers is required if LSD is to be successful.

1. INTRODUCTION

Limit State Design (LSD) procedures for bridge superstructures and substructures were introduced as part of a new Ontario Highway Bridge Design Code (OHBDC, 1979). Design methods prior to 1979 were based on working stress design (WSD) methods. The new Code addressed the design of substructures and retaining walls, and the communication and coordination between geotechnical and structural engineers. Neither geotechnical engineers nor structural engineers accepted the new terminology, the new technology, and LSD ideas. In addition, questions arose about the codification of geotechnical design procedures. The perception within the geotechnical profession in Canada was that LSD is statistical in nature. This negative reaction was unexpected.

Changes in the design process from working stress design to limit states design, problems of code writing, the selection of earth pressures, and the design of shallow foundations and deep foundations are discussed here.

2. THE DESIGN PROCESS

The final design, including both the structure and supporting soil/rock, must have an acceptable level of reliability and should minimize any loss of function. Uncertainty is present because of the variability of load, material characteristics, resistance predictions, imperfections of analysis and an incomplete knowledge of the system. There is a perception that structural design is an "exact"

Limit state design in geotechnical engineering

INTERNATIONAL SYMPOSIUM

COPENHAGEN 26-28 MAY 1993

ISLSD 93 Vol 2 / 3

Sponsor: Danish Geotechnical Institute

May 1993
dgf-Bulletin **10**

DANISH GEOTECHNICAL SOCIETY
DANSK GEOTEKNISK FORENING