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ELASTO-PLASTIC BEHAVIOUR OF REINFORCED CONCRETE BUILDING BASED UPON TEST RESULTS OF RESTORING FORCE CHARACTERISTICS OF PILE FOUNDATION DURING EARTHQUAKE LOADS

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SUMMARY

In testing laboratory, many full scale planar and space frame tests were carried out due to earthquake loading. However the foundations for these test specimens are quite different from those of real buildings. In the testing laboratory the foundation of test building is constructed as fix, however in reality it has some restoring force characteristics affected by soil and pile systems. (Ref. 1) In this paper, the elasto-plastic behaviour especially required ductilty foctor of each member of building structures which is set on the real pile foundation system is discussed at hinge mechanism.

INTRODCTION

In the analysis, mathematical modelling of load-deflection relationship for each member; beam, column and shear wall is able to be reasonably evaluated in analytical manner if its parameters such as geometical shape and amount of longitudinal and lateral reinforcements etc. are known. The longitudinal force-displacement relationship of pile system is derived from test results. Such mathematical modelling of the relationship between axial force and longitudinal displacement of pile was defined as tri-linear type according to the test result of full scale cast-in-place reinforced concrete pile specimen. This test was supervised by our Building Research Institute.

A non-linear frame analysis using such developped mathematical modells system was carried out in this paper. In the case considering the effect of pile foundation system, the ductility factor of each member of building at hinge mechanism was much larger than that in case ignoring pile foundation system which is often carried out in full-scale testing laboratory. From several cases of this analytical study considering pile condition, it became clear that the required ductility factor of member at hinge mechanism should have been larger than those in the condition of fixed foundation. The realistic required deformability of each member of the building system at hinge mechanism was obtained. And by some type of foundation condition, existing ultimate strength design system on foundation has to be drastically improved.

ALTERNATIVE PULL & LOAD TEST ON CONCRETE PILE

The test specimen is composed of concrete cast in situ pile and reinforced concrete footing as shown in Fig. 1. The size of test pile is 0.9m in diameter and 8m in embedded length. The footing is 1.5m in width, 1.9m in length and 1.7m in height. Subsoil condition at test site is the volcanic cohesive soil so-called Kanto loam was selected as test soil deposit as shown in Fig. 1. Comparatively uniform soil condition was clarified from the results of soil exploration to the depth of

8m expected as the level of test pile tip. The average N-values obtained from the standard penetration tests at the two test holes range from 3 to 6.

Hysteretic curve at pile head in small displacement region shows nearly elastic property with extremely small residual deformation as shown in Fig. 2. This figure suggests that the vertical spring of test pile is same both in tension and compression load conditions. In large displacement region, yield loads and displacements in tension and compression sides are 980KN at 0.95mm and 640KN at 0.67mm judged from logP-logS method, and the ultimate pullout load is 1,500KN at about 5cm. Practically skin friction is same in upward and in downward. The ultimate pullout load is corrected 1,320KN from the original 1,500KN on the basis of correction considering the effect of the weight of pile system. Figure 3 shows the relation between the skin friction and relative displacement. The skin friction is mobilized 50-60KN/m^2 at 3-5mm at each depth. Therefore the longitudinal stress distribution along pile was linear as shown in Fig.4. And after that it reaches the ultimate state with rapid increase of displacement.

MATHEMATICAL ROCKING MODEL

Pile test result indicates the necessity to take account of influence of effective weight of test specimen. Figure 5 illustrates such conditin schematically the weight of pile system acts already at the start of test. Considering such weight , movement of the origin from O' to O is made in Fig. 5. Then the yield loads in tension and compression come to be nearly equal in upward and in downward.

Fig. 6 shows the selected four cases of skeleton curves on rocking moment vs rotation angle. These curves are derived from the actual size of pile system based upon the test results mentioned above i, e.; diameter: 0.9m and pile length; 10 meters-25 meters. Such size of piles were assumed to be set at both sides of the bottom of shear wall. The longitudinal compressive stiffness of piles was assumed to be same as the tensile stiffness. Therefore the skeleton curve of shear wall at the base is affected by both conditions; flexural yield of shear wall and rocking yield. Case 4 is for the condition that the foundation is fixed.

MODELLING OF BUILDING MEMBERS AND FRAME ANALYSIS

Fig. 7 is the model building planar . This is similar to the test planar of the seven story reinforced concrete building which was introduced as US-Japan cooperative research program (Ref. 2,3 and 4). As shown in this reference, all of members including shear wall showed flexural yield at its hinge mechanism under the condition that the foundation is fixed in the full-scale testing laboratory. In this planar, the tri-linear skelton curves of rocking such as shown in Fig. 8 are connected to the base of shear wall.

Beams and columns were idealized as a perfectly elastic massless line element with two nonlinear rotational springs at the both ends. In orther words, all inelastic flexural deformations were assumed to concentrate at the ends of the flexible part of the member. The nonlinear axial spring was not included in a beam model. The stiffness properties of beams and columns under monotonically increasing load were evaluated by applying an imaginary antisymmetric moment distribution to the flexible part of the member. The elastic stiffness, used for the elastic line element of the one-component model, was evaluated as a T-shaped beam, with the effective width of the slab yield moment of a T-beam were calculated using idealized stress-strain relationship for the reinforcing steel. The slab was found to contribute to the resistance of a beam; the slab reinforcement yielded under beam negative moment, progressively spread with increasing beam deformation. In this test building the elastic stiffness properties of columns were calculated for the gross concrete section, ignoring the contribution of the reinforcement.

Cracking moment of column was evaluated for the caluculated constant axial load. Approximate expressions (Ref. 5) were used to evaluate yield moment M_y of beams andcolumns ; i. e.,

$$My = 0.9 A_s fyh \qquad \text{(beams)}$$

$$M_y = 0.8A_s f_y h + 0.5ph(1\text{-}P/bhf'_c) \qquad \text{(columns)}$$

Where b,h and h'=width ,overall and effective depth of section; f'_c=compressive strength of concrete; f_y=yield stress of longitudinal bars; A_s=area of tensile bars. The variation of axial loads due to the lateral loads was not considered in evaluating flexural resisting capacity. The rotation at member's end was evaluated by a simple emprical formula by Sugano (Ref.6) prepared for reinforced concrete beams and columns of rectangular section subjected to anti-symmetric bending. The ratio a_y of the secant stiffness at the yield point to the initial stiffness was proposed to be

$$a_y = (0.043 + 1.64np_t + 0.43M/Vh + 0.33P/bhf'_c)\,(d/h)^2$$

in which n=modular ratio of steel to concrete; p_t=tensile reinforcement ratio; M/Vh= shear span to depth ratio, where M and V are maximum moment and shear at the critical section. Ninety percent of test data studied fell within 30 percent range of the calculated(Ref. 6). The column-end rotation less the elastic deformation was assigned to the rotational springs of the one-component member model. The skelton moment-rotation curve was represented by a trilinear line with stiffness changes at cracking and yielding points. These data for each case are uniformly same in Case1,2,3 and 4.

In case of shear wall the modelling is quite same as that in case of columns. Because the shear wall of this test structure showed perfectly flexural failure until the hinge mechanism of the building.

Including the effect of actually evaluated rocking spring at the base of shear wall, inelastic frame static-response-analysis were carried out. The lateral force distribution was assumed to be reversed triangular shape. Fig. 8 shows the results of responsed story drift in each floor. In CASE 1 which has weak rocking spring, the total mechanism of the system was dominated by yielding of rocking model. Therefore the shear capacity was much smaller than those of the other three cases as shown in Fig. 8. In CASE 2 and 3, the story drift at the total hinge mechanism became larger than that in CASE 4 (fixed base). In CASE 2 and CASE 3, ductility factors of beams connecting to shear wall : mb larger by 20%-30% than those in CASE 4.

The ductibity factors of foundation beams: mF are also much effected by the condition of skeleton curve of rocking model as shown in Fig. 8.

CONCLUSIONS

1. From the survey of existing field test results of piles, it was possible to realize the model of rocking skeleton curve in order to evaluate an accurate mechanism of total structure.
2. Especially in case that frame structure includes shear wall system with rocking model, the reguired ductility factors were 20 % - 30 % larger than those in case of fixed foundation.

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REFERENCES

1. "Recommendation for Design of Building Foundation" Architectural Institute of Japan 1988 (in Japanese)

2. Nakata, S., S. Otani, T. Kabeyasawa, Y. Kai, and S. Kimura, "Tests of Reinforced Concrete Beam-Column Assemblages --U.S.- Japan Cooperative Research Program-", Report submitted to

Joint Techical Coordinating Committee, U.S.-Japan Cooperative Research Program, Building Research Institute and University of Tokyo, 1980

3 Okamoto, S., J.K. Wight, S. Nakata, M. Yoshimura, and T. Kaminosono, "Testing: Repair and Strengthening; Retesting", to be published in ACI Special Publication on U.S.-Japan Cooperative Research program, Phase-1: Reinforced Concrete Structures.

4. Otani, S., "Nonliner Dynamic Analysis of Reinforced Concrete Building Structures", Canadian Journal of Civil Engineering, Vol. 7, No. 2, April 1980, pp. 333-344.

5. Architectural Institute of Japan, " AIJ Standard for Structural Calcuration of Reinforced Concrete Structure(in Japanese)" revised in 1982

6. Sugano, S., "Experimental Study on Restoring Force Characteristics of Reinforced Concrete Members(in Japanese)", Doctor of Engineering Thesis, University of Tokyo, 1970.

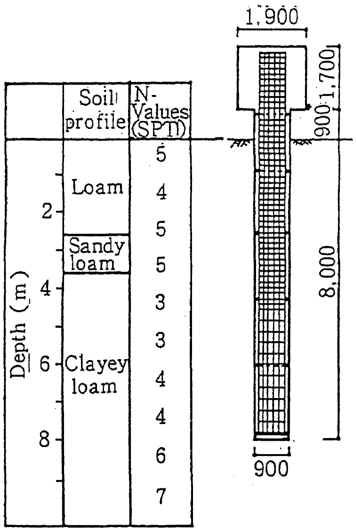


Fig. 1 Test Pile & Soil Condition

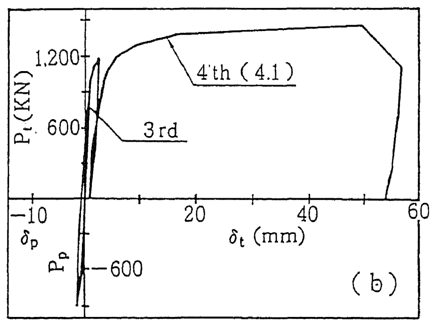
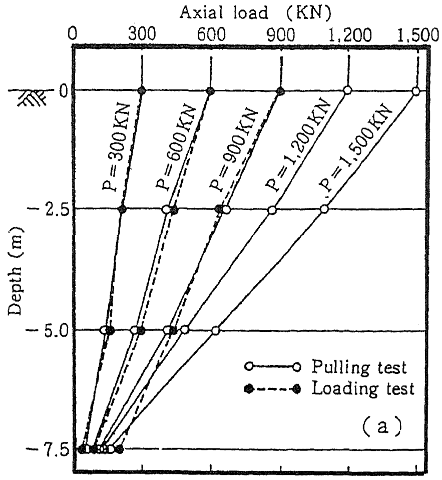


Fig. 2 Longitudinal Hysteresis by Pile Test



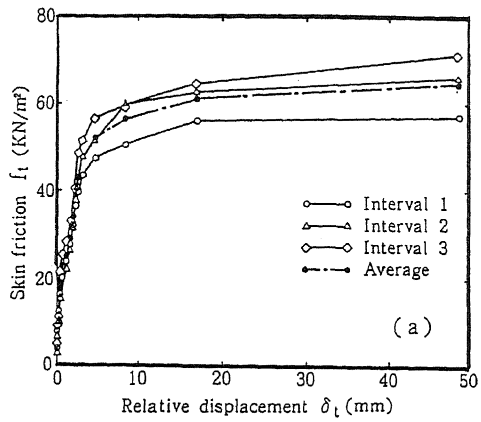


Fig. 4 Skin Friction & Relative Slide

Fig. 3 Distribution of Longitudinal Stress of Pile

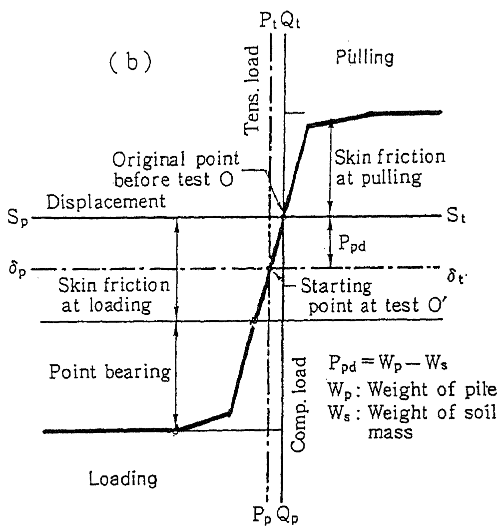


Fig. 5 Mathematical Modelling of Pile Deformation

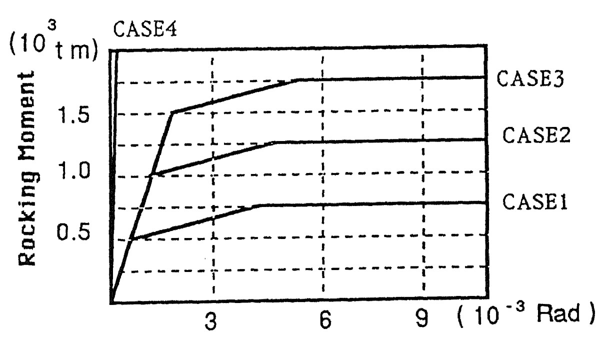


Fig. 6 Selected Four Cases of Rocking System

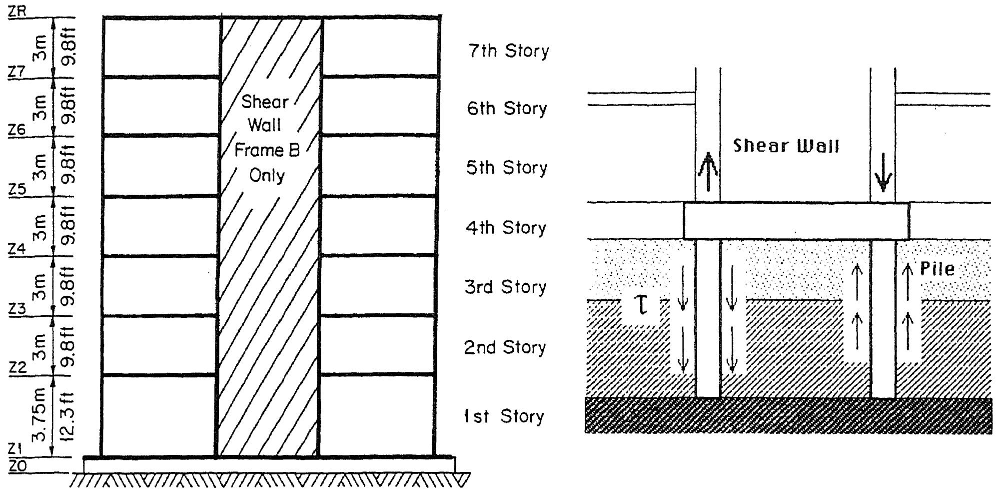


Fig. 7 Planar Model and Shear Wall- Pile System

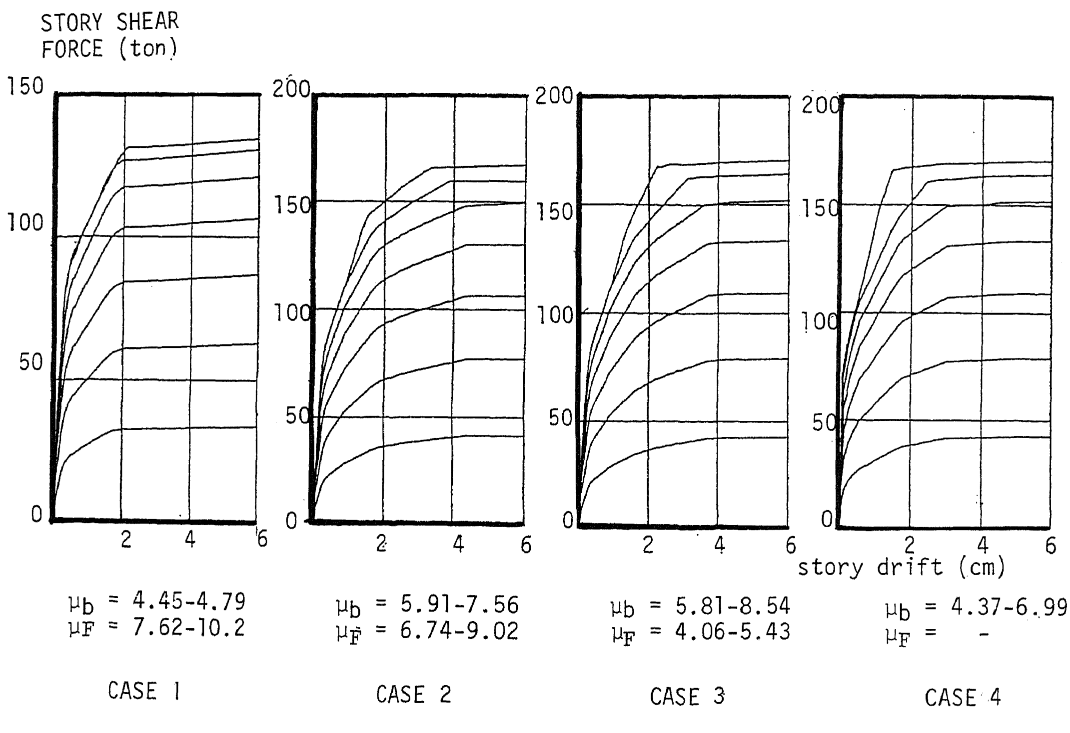


Fig. 8 Results of Inelastic Response Analysis