A Case History of Raft Foundation Behavior for a Large-Scaled Building Complex

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SYNOPSIS This paper describes the field observation of settlement of a raft foundation for a large scale building complex in Nagoya city in Japan. The complex consists of a 28 story hotel, a 15 story office building and a low-storied banquet hall in the middle all of which are supported on a single 80 m sqare basement on raft foundation. Since the load on the foundation is non-uniform, the differential settlement of the foundation was investigated analytically, taking into consideration the interaction between the ground and the raft. To confirm the validity of the design, measurements were conducted for about 2 years during the building construction. The results of the measurement are discussed below.

INTRODUCTION

The building complex being investigated like most high-rise buildings, 60 m or higher, constructed in the center of Nagoya City, is built on a raft foundation. The soil below consists of a diluvial deposit (the so-called Atsuta stratum,) which is made up of an alternating gravel layer on the top of a stiff silt layer. Although the Atsuta stratum is characterized by a large bearing capacity, some relatively large ground heaves and settlements have been recorded at some construction sites. The Geotechnical Data of Subsoil in Nagoya (1988) indicates that for designing buildings on raft foundations the average contact pressure should be less than 294 KN/m^2 and the differential settlement is allowed up to a deformation angle (θ cr) of 1×10^{-3} (rad).

For this particular complex, the local load on the raft exceeds 294 KN/m^2. Therefore, a detailed examination of the differential settlement is necessary, taking into consideration the ground rigidity and the interaction between the ground and the raft. In order to verify the design methodology of such a compound building, the behavior of the raft foundation was monitored during the course of excavation and construction.

BUILDING AND SOIL CONDITIONS

Fig. 1 shows a schematic of the building complex composed of a 110.5 m tall hotel building with 28 stories, a 64.5 m high office building with 15 stories and a low-storied banquet hall in between the two high-rise buildings. The hotel and office buildings are connected by an expansion joint and they are both supported on a single basement on a raft foundation. The excavation depth for the raft foundation is about 18 m.

Fig. 2 summarizes the soil investigation conducted at the construction site. The soil profile up to a depth of 36 m is the first diluvial stratum, the so-called Atsuta stratum, composed of an upper alternating gravel layer to a depth of 30 m and a stiff silt layer below. The second diluvial gravel stratum extends from a depth of 36 m to a depth of 55.5 m. Below this lies the Tertiary stratum. Groundwater was found at a depth of 8 m.

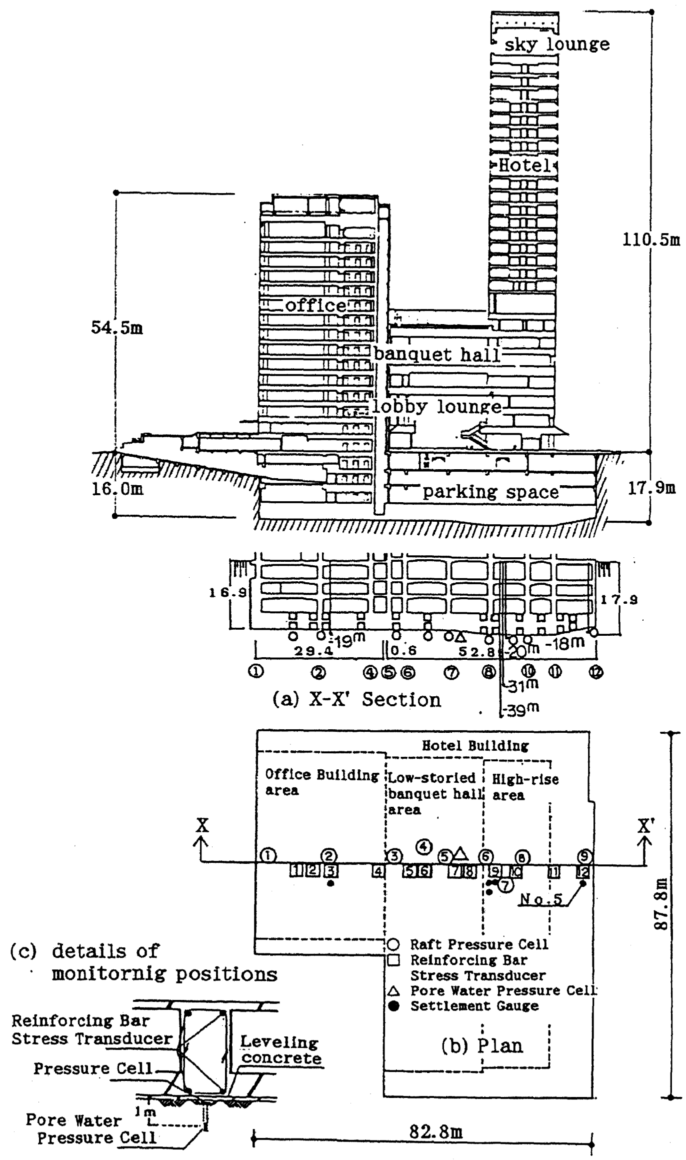

Fig-1 Schematic of building

FOUNDATION DESIGN LOAD

The bottom of the raft foundation is at a depth of 16.9 m to 17.9 m on the diluvial gravel stratum. As the building complex has two high-rise buildings, the building load is non-uniform. The design average contact pressure is 343 KN/m² for the hotel building, 166 KN/m² for the banquet hall, 294 KN/m² for the office building and 118 KN/m² for the lower part of the perimeter of the complex. The average contact pressure over the entire area is 230 KN/m² which is approximately equal to the weight of the removed soil, 217 KN/m², or less than the weight of the removed soil when buoyancy is taken into account (see Fig-13).

INSTRUMENTATION

The differential displacement of the bearing strata, the contact pressure and the pore water pressure on the underside of the raft, the foundation beam stress and the displacement of the foundation beam were measured in order to monitor the behavior of the foundation. The locations of the transducers are shown in Fig. 1. Transducers were placed so as to obtain a planar distribution of displacement for studying the differential settlement, the behavior of the upper diluvial layer at depths from 20 to 31 m and the stiff silt layer at depths from 31 to 39 m (see Figs. 1 and 2). The contact pressure at the underside of the raft was measured by earth pressure cells which were installed at 9 different points along section X - X' taking into account the non-uniform building load distribution. As the stratum below the foundation is composed of gravel, the earth pressure cells were positioned on a 50 mm high sand stratum which was thoroughly compacted, as shown in Fig. 1. The pore water pressure at the underside of the raft foundation was measured by pressure cells, which were positioned at a depth of 1 m under the raft foundation at the center of the site. The foundation beam stress was measured by strain transducers affixed to the reinforcing bars. A total of 48 strain transducers were used: 2 to each of the top and bottom main reinforcing bars along the 12 foundation beams of section X - X'. The differential settlement of the foundation beam was measured by a level, taking the settlement gauge No. 5 as the control point.

OBSERVATIONS

a) Differential settlement of the bearing strata

The measurement results of the differential settlements are shown in Figs. 3 and 4. The control points for the observation were chosen at two places in two buildings away from this construction site. Their levels were checked at each time, it was measured to confirm zero settlement at the control points. At the time of construction the groundwater level in the excavated portion had already fallen below 18 m. Consequently, the settlement associated with the fall of the groundwater level was not included in the measurements. The maximum ground heave after the final excavation was 42.0 mm. Based on the displacement distribution at the time of excavation (Fig. 4(a)) the lower stiff silt layer was found to have a greater inclination and a larger strain than the upper gravel layer. Also the heave at the depth of 39 m is about half that at the depth of 20 m. However, the displacement

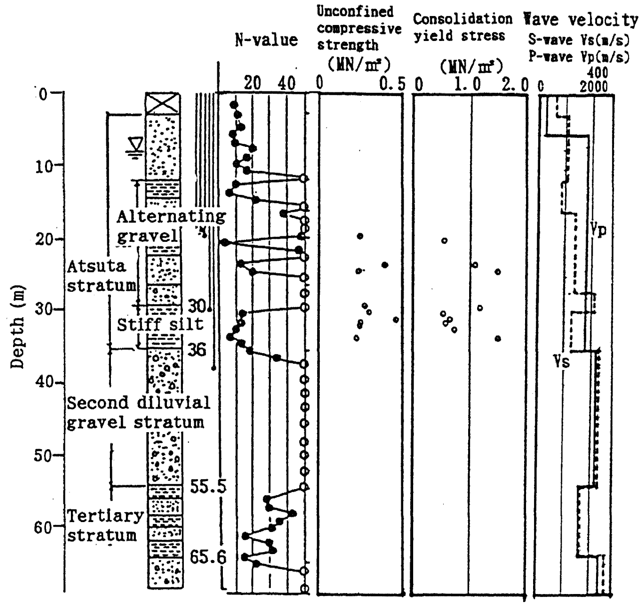

Fig-2 Summary of soil investigation

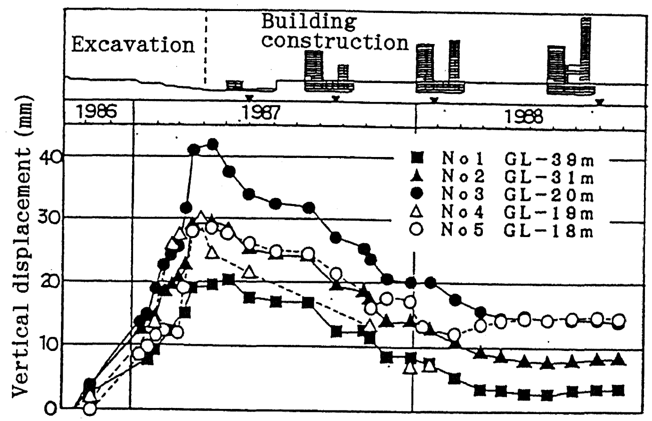

Fig-3 Measured vertical displacement

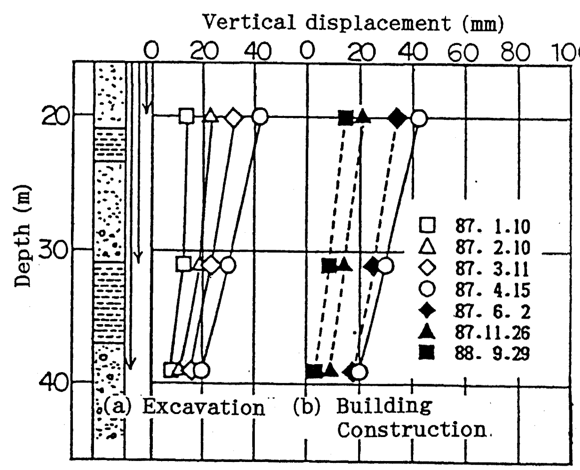

Fig-4 Measured differential displacement of bearing strata distribution (Fig. 4(b)) of the excavated ground during construction of the building shows that the upper and lower strata have similar inclinations, which were essentially straight lines.

b) Contact pressures of the raft foundation

Fig. 5 shows the changes over time in the earth pressure and the pore water pressure. The earth pressure cells were adjusted so that the measured pressure would be equal to the pressure calculated from the time that the foundation beam concrete was placed. The contact pressure gradually increased as the construction progressed and became larger below the office building (No. 2) and the high-rise hotel building (Nos. 6 and 8) than below the banquet hall (No.5). A change in contact pressure induced by buoyancy was, also, observed after the completion of the underground building frame when the groundwater level started to return (January 1988). Generally, the contact pressure increases with the water pressure when it is smaller than the average contact pressure, and decreases when it is larger than the average contact pressure. Such contact pressure fluctuations due to varied groundwater level was also seen from July through November, 1988, when the groundwater level changed as a result of its use.

c) Settlement at the foundation beam

The measurement of the settlement of the foundation beams was initiated on July 21, 1987, after the placement of the foundation beam concrete. Fig. 6 shows the distribution of the foundation beam settlement from the time of the excavation's completion which was derived from the measured differential settlement of displacement gauge No.5. In the early stages of the construction, the area around the office building had a greater settlement than the hotel building since its construction was completed ahead of the hotel structure. At the final stages of the construction, the settlement under the banquet hall, whose load was about 50-60% of that of the high-rise hotel building, was as much as 70-80% of the settlement under the hotel building. This implies that to a certain degee the settlement was distributed uniformly.

d) Stress in the reinforcing bar of the foundation beam

The measurements of the stress in the reinforcing bars of the foundation beams were begun on July 21, 1987. Fig. 7 shows the changes over time in the stresses in the reinforcing bars of the foundation beams at the office building (No.3), the banquet hall (No.6), and the hotel building (No. 10). The stresses in both the top and bottom reinforcing bars are shifted toward the compressive side at the high-rise hotel and the office buildings, while at the low-storied banquet hall the reinforcing bar stresses shifted toward the tensile side until the ground water level returned (January 1988). However, speaking as a whole, the stress pattern is relatively symmetric and it is judged that the concrete at the tensile side worked effectively throughout the construction period. Fig. 8 shows the changes over time in the bending moment of the foundation beam. The bending moment in the foundation beam was calculated based on the assumption that all the concrete in the tensile side was effective and that the

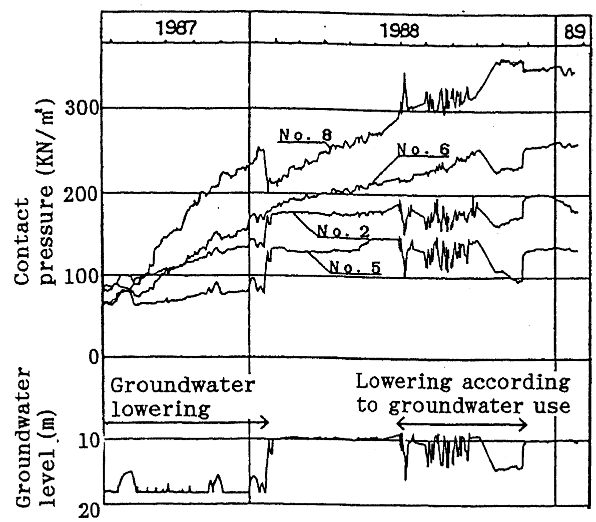

Fig-5 Measured contact pressure and groundwater level

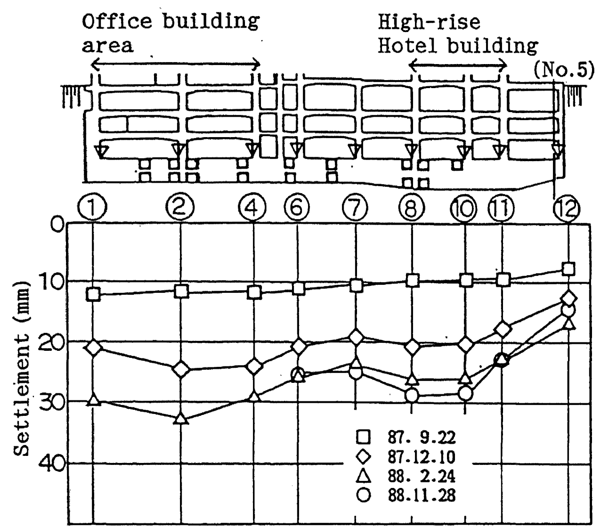

Fig-6 Measured settlement of raft foundation beam

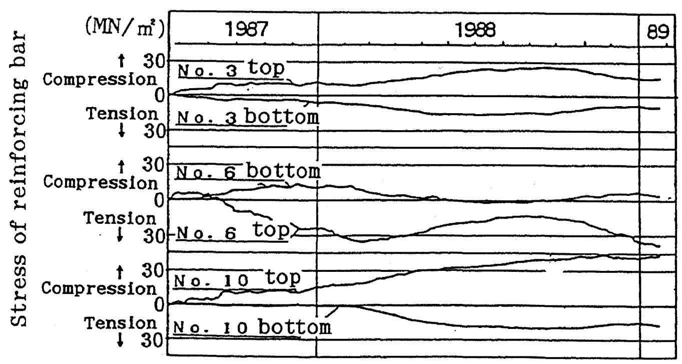

Fig-7 Development of reinforcing bar stress ratio of Young's modulus of the reinforcing bars to concrete is 15 to 1.

The bending moment at the high-rise office and hotel building increased as the construction progressed. On the contrary, at the low-storied banquet hall negative bending moment was observed. Buoyancy associated with the return of the groundwater level, seemed to dampen the rate of increase of the bending moment at the high-rise hotel building. A similar influence was seen in the rate of increase of the negative bending moment at the low-storied banquet hall. These changes in stress are relatively gentle compared to the changes in contact pressure and pore water pressure, shown in Fig. 5. The reason for this is a long term change in the stress level which cause a creep-like behavior. Fig. 9 shows the distribution of the stress in the reinforcing bars at the major stages of the construction. At the high-rise office and hotel buildings compressive stresses of about 20 MN/m² and 40 MN/m², respectively, were produced. At the low-storied banquet hall, a tensile stress of about 40 MN/m² was produced. It is found that the reinforcing bar stress is influenced more by the position in the whole building than by the position in an individual beam.

ANALYTICAL STUDY

a) Evaluation of ground rigidity

Fig. 10 shows the relationship between the elastic modulus of each soil layer, which was re-calculated from the measured ground displacement, and loading and unloading stress calculated at the middle depth of each layer. In calculating the displacement below the second gravel stratum, the displacement at the bottom of the stratum in the Tertiary (at the depth of around 66.0 m) is taken to be zero. The elastic modulus of the upper alternating gravel layer around the bottom of the excavated area rapidly decreased as the unloading stress increased but very little change in the elastic modulus was noted beyond the lower stiff silt layer. Under loading conditions, the elastic modulus was comparatively stable up to the level where it decreased because of the unloading. The elastic modulus of the upper alternating gravel layer was almost equal to that of the lower stiff silt layer under the loading conditions.

The most important factors which lead to a smaller measured elastic modulus than that obtained from PS logging are considered here; namely:

1) the effect of the mean principal stress, (σm)
2) the effect of the shear strain, (γ)

In this investigation, the results of the shear strain (γ) from cyclic tri-axial tests are used to define the shear modulus G which is used instead of the elastic modulus, E. As the effect of both factors acting simultaneously is difficult to predict, so the factors are examined separately. Hardin(1972) suggested that the effect of the mean principal stress on G is incorporated as shown in Eqs. (1) and (2). And Mayne(1982) suggested that the coefficient of earth pressure at rest (K_{or}) is as shown in Eq.(3).

$$G = A \cdot \sqrt{\sigma_m} \cdot F(e) \qquad (1)$$

$$\sigma_m = \frac{1}{3}(1 + 2K_{or})\,\sigma_z \qquad (2)$$

where F(e) is a function of the void ratio e and A is a constant.

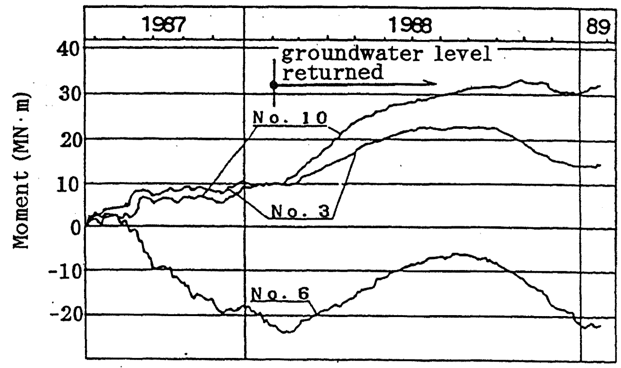

Fig-8 Development of bending moment in foundation beam

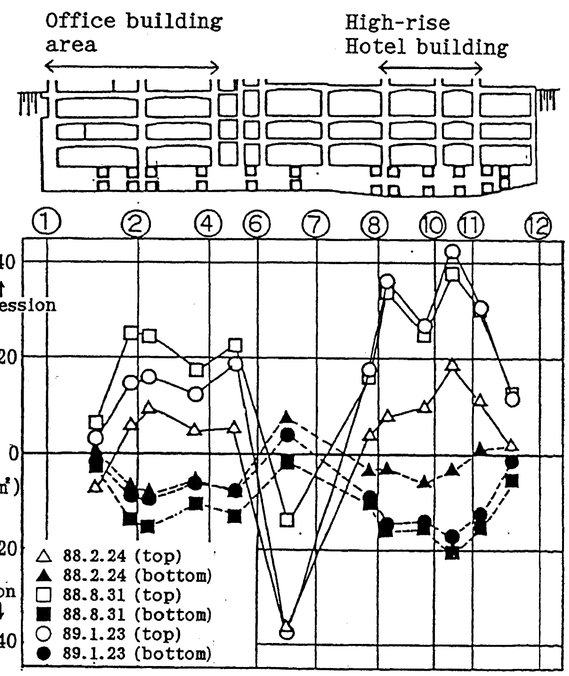

Fig-9 Measured distribution of reinforcing bar stress

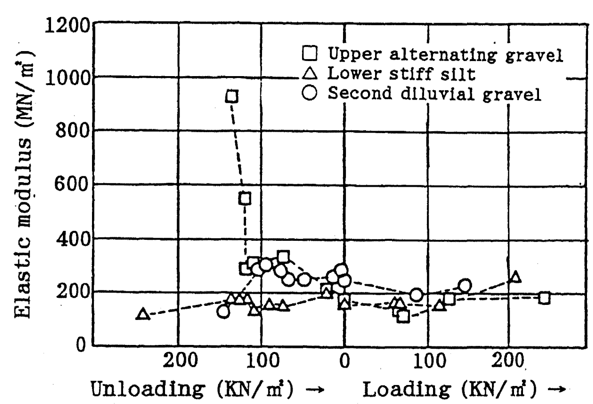

Fig-10 Relationship between increasing load and ground rigidity

$$K_{or} = (1-\sin\phi)\cdot OCR^{\sin\phi} \qquad (3)$$

where ϕ is a internal friction angle and OCR is a over consolidation ratio.
By substituting OCR=1.0, ϕ=30°, e=constant in Eqs. (1) to (3), the shear modulus ratio (Go'/Go) coused by mean principul stress change is obtained by Eq. (4).

$$\frac{G_o'}{G_o} = \sqrt{\frac{1+\sqrt{\sigma_{zo}/\sigma_z}}{2\cdot(\sigma_{zo}/\sigma_z)}} \qquad (4)$$

where σ_{zo}, σ_z are the vertical stress before and after the excavation.
Fig. 11 shows the relationship between the shear strain (γ), re-calculated from the measured differential displacement and G/Go'. Go' is obtained from Eq. (4) using the Go from the PS logging. Fig. 11 indicates that G/Go' shows a tendency to decrease as the shear strain (γ), increases. The curve in the figure is drawn based on the Ramberg-Osgood model Eq. (5), using the values of R and K obtained from cyclic tri-axial tests presented by Hatanaka (1982). The values that were used are: R=1.7 to 1.9, K=120 for cohesive soil, and R=2.0 to 2.2, K=17000 for sandy soil.

$$\gamma = \frac{\tau}{G_o'} + K\cdot\left(\frac{\tau}{G_o'}\right)^R \qquad (5)$$

The measured shear strain and shear rigidity show a relationship similar to that obtained from the data of the cyclic tri-axial tests.

b) Analysis of foundation settlement

Prior to designing the foundation, an analysis which incorporated ground and foundation beam interaction was carried out to examine the building load distribution. Fig. 12 shows a flowchart of the analytical procedure that was used. The steps in the analysis were as follows:

(1) The elastic modulus is first evaluated from PS logging data. The actual value used in the analysis is reduced according to a ratio which reflects the strain level.
(2) The elastic displacement of the ground is calculated based on loads being transmitted by columns and soil below the raft foundation is modeled by equivalent springs based on the theory of elasticity.
(3) The raft foundation supported on equivalent springs is analyzed and the displacements and reaction forces of the equivalent springs are calculated.
(4) Steps (2) and (3) are repeated until the displacement of the equivalent springs calculated in (3) converge to the elastic displacement of the ground calculated in step (2).

The elastic modulus used in the analysis is taken from the results of PS logging shown in Fig.2, with some modifications based on the measured value of G/G'o as shown in Fig. 11. As previously mentioned, the measured elastic modulus (E), decreased as the shear strain (γ), increased during the excavation (i.e. unloading), but under loading E remained at about the same value regardless of the strain level. Therefore, in the analysis the value of the elastic modulus of each stratum at the time of completion of excavation was adopted as the elastic modulus under loading.
The column loads used in the foundation analysis were the reaction forces that were calculated from an analysis of the superstructure with the columns assumed as being

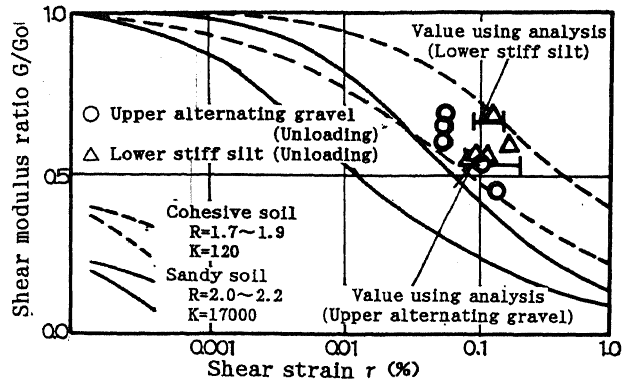

Fig-11 Relationship between shear strain and shear modulus ratio

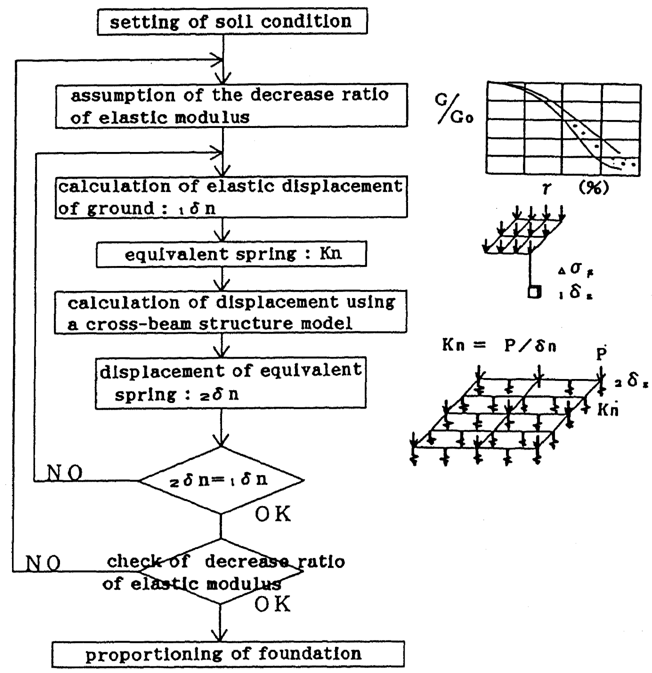

Fig-12 Flowchart of Numerical Calcuiation

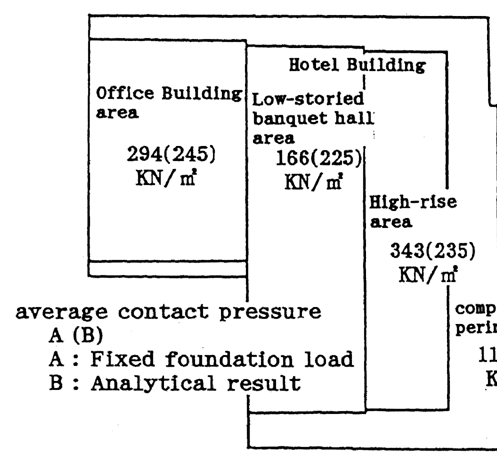

Fig-13 Comparison between Fixed Foundation Load and Analytical Load fixed at the base. The buoyancy was estimated to be 70 - 80 KN/m² on the assumption of a depth of 10 m for the groundwater level. Equivalent springs for modeling the soil below the raft were used under the columns and at the center of the foundation beams.

Fig. 13 shows the analytical results for the average contact pressure. The average contact pressure is 235 KN/m² at the hotel building and 245 KN/m² at the office building. The analysis (which incorporates the interaction between the soil and the foundation beam) revealed that the localized load of the building is to a large extent distributed uniformly throughout the foundation beams.

Fig.14 shows a comparison between measured and calculated displacements of the foundation beams after the completion of the building frame. The displacement of the foundation beam was obtained by measurements taken by levels at points on the foundation beams and the readings taken from the differential settlement gauge No. 5 which is the assumed control point. Fig.15 shows a comparison between the measured and calculated bending moment in the foundation beams. The measured bending moment is obtained from the stress calculated from the strain transducers on the reinforcing bar on the assumption that the concrete in the tensile side is effective. The first measured value was on Aug. 31, 1988, when the groundwater level was relatively stable, and the other was on January 23, 1989, at the time of the final measurement, indicated in Fig. 8, show a creep-like behavior resulting from the changes in the groundwater level. The measured moment distribution shown in Fig. 15 exhibited a behavior similar to that of a single beam supporting by itself both the office and hotel buildings. The analytical results agree well with the measured data.

CONCLUDING REMARKS

The in-situ behavior of a building complex, consisting of a high-rise hotel building and an office building with a low-storied banquet hall in the middle, all supported on a single basement, was observed. The results of the field measurements are summarized below.

1) The maximum ground heave caused by the excavation was 42 mm and the maximum settlement caused by the building construction was 28 mm. The changes in the elastic modulus of the ground agree well with a decrease ratio which in turn is evaluated based on a decrease of the mean principal stress caused by excavation and a decrease of ground rigidity caused by shear strain.
2) The measured distribution of the displacement and stress are in relatively good agreement with the results of an analysis which incorporates the interaction of the building frame and the soil.

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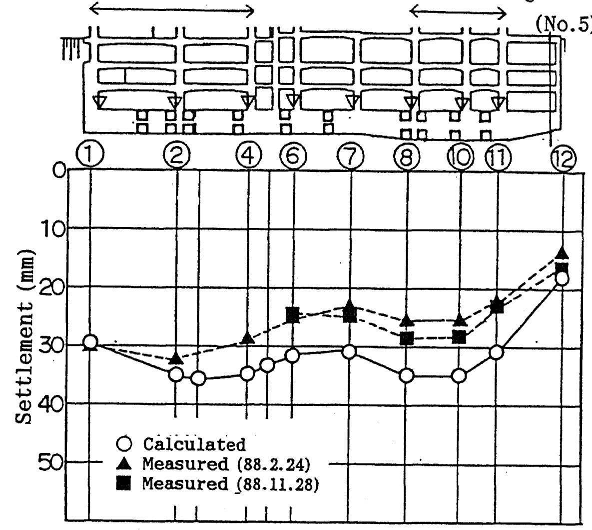

Fig-14 Comparison between calculated and measured settlement

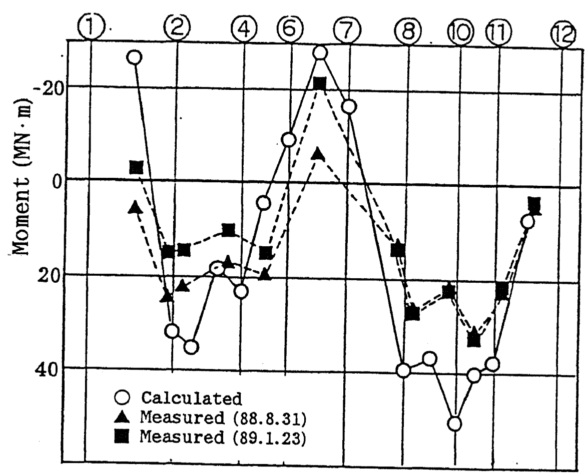

Fig-15 Comparison between calculated and measured bending moment in foundation beam

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