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ASEISMIC PROPERTY OF SHELL TYPE FOUNDATION

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SUMMARY

Shell Foundation is a type of foundation construction which provides a cylindrical thin hollow shell under an ordinary spread foundation to confine the lateral movement of encompassed soil. By model vibration test using dry sand, the relationship between decrease in shear strength of model ground and the effect of stabilization by shell has been discussed. By model loading test, the bearing capacity increase ratio of various diameter and embedded length of shell and the bearing mechanism has been discussed. The dynamic and static bearing capacity of Shell Foundation is much higher than that of ordinary spread foundation as the result of transferring the load to the bottom of hollow shell through encompassed soil.

INTRODUCTION

Having observed the detrimental settlement of foundation caused by liquefaction during the Tonankai Earthquake (1944) and Fukui Earthquake(1948), Dr. J.K. Minami proposed Shell Foundation method(1949, 1956). From the results of number of statical loading tests and field tests, Dr. Minami verified the effectiveness of Shell Foundation in minimizing settlement and increasing bearing capacity. And also, based upon Terzaghi's Bearing Capacity Theory, Dr. Minami explained theoretically the bearing capacity mechanism of Shell Foundation.

Recently, study on shell type foundation method has been made by Broms(1981) and Bhandari(1986), and studies concerned with it has increased in numbers. But the principal design concept of Shell Foundation dates back about a half century, and has already been introduced in practice in those days. In 1927, when the first subway in Japan was about to be constructed, Rudolf Briske, a German engineer, pointed out the possibility of stabilizing a foundation by encompassing it with sheet piles, during his discussions with Dr. Tachu Naito.

During the construction of Niigata City Hall (RC, 9F, 1BF) and Nagai Denki Building (RC, 3F), Dr. Naito gave instructions to keep sheet piles used for retaining wall remaining as to confine the loose granular soil beneath the footing. In Niigata Earthquake (1964), while a number of buildings and structures was damaged by liquefaction, the two buildings did not suffer any damage due to the subsoil deformation.

Even though this type of foundation method has already been used in practice, down to present, no experiment has been conducted to analyze the effectiveness of Shell Foundation in preventing liquefaction of sandy ground during earthquake.

To verify the effectiveness of Shell Foundation in preventing liquefaction, an experiment using saturated sand should be conducted. But it is difficult to maintain saturated sand in a condition of liquefaction for a long period. Thus, model vibration experiment, where a large amplitude of acceleration is applied to dry sand, has been conducted, which basically employs the theory presented by Mogami(1953). Mogami confirmed experimentally that the shearing strength of dry sand diminishes considerably with increasing acceleration of vibration.

Previous researches has not been able to evaluate analytically and experimentally the effectiveness of shell in confining the subsoil beneath a footing. This study determined quantitatively the subsoil displacement, circumferential stress of shell and stress distribution of subsoil by means of statical loading test, photoelasticity method and Finite Element Method, and further indicated the contribution of these factors in increasing the bearing capacity of Shell Foundation.

MODEL VIBRATION TEST

Model test using dry sand has been conducted based on the dynamic property of dry sand where the dynamic shear strength of dry sand decreases in proportion to the acceleration level.

Shearing stack box has been developed to verify the stabilizing effect of shell foundation in relationship to the shear strength and also the modulus of shear deformation of dry sand under vibration.

Fig.1 shows the test apparatus. Shearing stack box is a stack of 23 hollow vinyl chloride rings with rod-bearings linking one another. After covering the inner surface of the container with rubber membrane of 0.5mm thickness, model ground was prepared by means of vibratory compaction method.

The material properties were; D_{50}=0.58mm, Uc=1.5, e_{max}(maximum void ratio)= 0.840, e_{min}(minimum void ratio)= 0.541, Gs≒2.65. Dr (relative density of ground) is about 98%. Table 1 lists the value of Vs(shear wave velocity) of the model ground.

The model foundation is 80mm in diameter and 75mm in height and weighs 1.3kgf. The embedded length of shell was fixed at the length of either 50mm(L50) or 100mm(L100), and for comparison, experiment on a plain foundation(L0) has been conducted.

Based upon the frequency sweep up test, the acceleration level had been fixed at the following four steps, 50, 100, 150, 200, varying from the level which no liquefaction occurred to the level which absolute liquefaction occurred. The sinusoidal motion has been inputted to the shaking table with the above base acceleration level and 25Hz in frequency level.

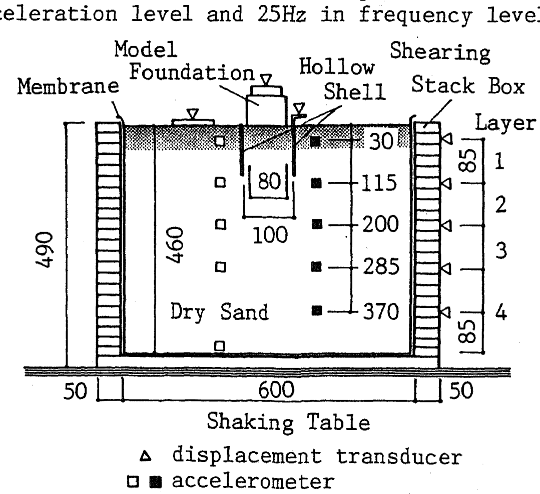


Fig.1 Test Apparatus for Model Vibration Test

Table.1 Shear Wave Velocity of Model Ground

Layer	Depth(mm)	Vs(m/sec)
1	30-115	36
2	115-200	68
3	200-285	111
4	285-455	120

Fig.2 shows the displacement mode of the shearing stack box. The displacement increases near the surface, especially when the acceleration exceeds 100gal.

Fig.3 presents the shear stress in relation to shear strain at each acceleration level. The hysteresis loop of the Layer 1, near the surface, becomes large, indicating the plastic behavior of soil. The shear stress and strain has been obtained from the following equations.

$$\tau(t)=\sum_{j=1}^{i} a_j(t)\cdot\rho_j\cdot l_j \ldots\ldots\ldots\ldots\ldots(1)$$

a:acceleration, u:displacement

$$\gamma(t)=(u_i(t)-u_{i-1}(t))/l_i \ldots\ldots\ldots(2)$$

ρ:density, l:thickness of layer

Fig.4 shows the relation of ratio of shear strength under vibratory condition and that at rest to input acceleration (on a log scale) for Layer 1, the layer which actual liquefaction had been observed. Shear strength has been measured using the portable vane tester(H=10mm, D=40mm).

According to Tanimoto(1959), angle of internal friction of sand decreases proportionally to the logarithm of the acceleration level. Fig. 4 reconfirms the results obtained by Tanimoto.

Fig.5 compares the settlement of foundation without hollow shell with foundation provided with the shell of L50 and L100 in relation to the acceleration level(on a log scale). Under the acceleration level of 50gal, there is no distinguished difference in settlement of foundations. But at the acceleration level larger than 100gal, where significant decrease in shear strength and shear modulus is observed, the settlement of plain foundation(L0) increases

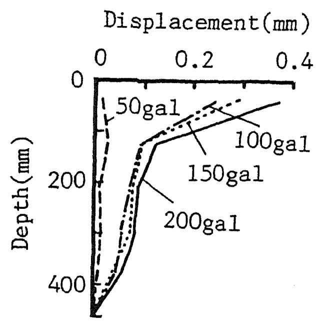


Fig.2 Displacement Mode of Shearing Stack Box

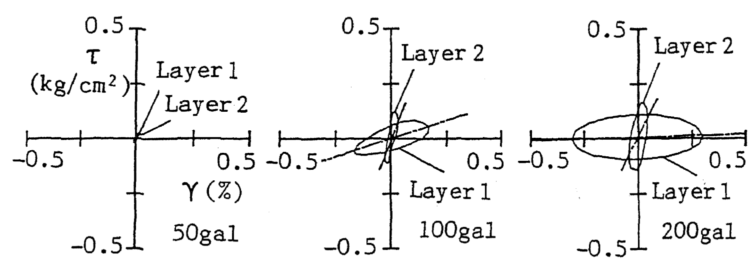


Fig.3 Hysteresis Loop

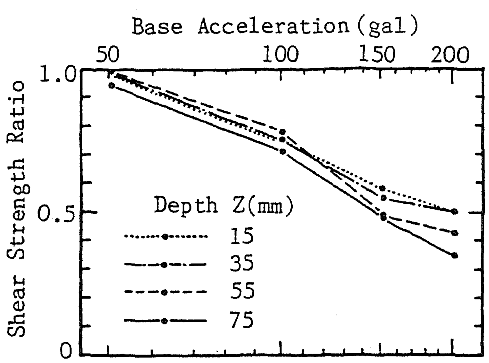


Fig.4 Shear Strength Ratio under Vibration to that of Static Condition versus Acceleration

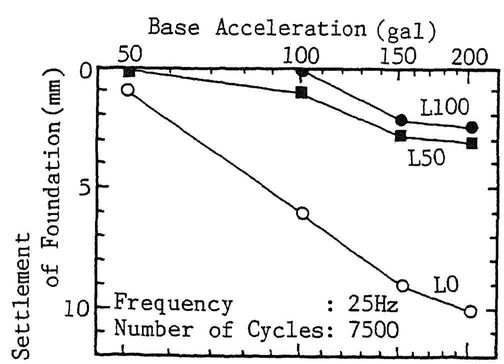


Fig.5 Settlement of Foundation versus Acceleration

proportionally to the logarithm of acceleration level. The same trend had been observed for shear strength of soil.

Settlement of foundation provided with shell(L50) is far smaller than that of plain foundation, even when the acceleration exceeds 100gal. When the acceleration level reaches 200gal, the settlement of Shell Foundation is 1/3 of that of plain foundation. L100, with sufficient embedded length, settles little compared to L50, which indicates the effect of embedded length.

MODEL LOADING TEST

Model vertical loading test has been performed to clarify the mechanism of increase in bearing capacity of Shell Foundation under statical loading conditions.

The test has been conducted on a model ground prepared by means of pluviation through air in a circular sand container(D=1500mm, H=1200mm). Major material properties of sand are as follows. D_{50}=0.19mm, Uc=1.77, e_{max}= 0.979, e_{min}= 0.606. Dr(relative density) of ground is 26% near the surface, and 56% near the bottom.

12 types of circular shell, with the length variations of three and the diameter variations of four, were used for the experiment. And for comparison, plain foundation has been loaded.

Table 2 lists the dimensions of all the foundation loaded.

Fig.6 shows the load-settlement curve for shell having the diameter of 315mm with different embedded length. The settlement of Shell Foundation is smaller than that of plain foundation under the same load. The figure states the significant effect of embedded length in minimizing the settlement and increasing the bearing capacity.

From the results of the loading test conducted on the foundation with the shell 450mm in diameter(Ds450) and 150mm in length(Ls150), almost no recognizable difference was observed between the Shell Foundation and plain foundation. In that particular case when the diameter of shell was large compared to that of the bearing plate and the embedded length was short, the shell showed little effectiveness.

Fig.7 shows the displacement of

Table.2 List of Model Shell Foundation

As/Ao	Ds(mm)	As(cm^2)	Ls/Do
1.10	315	779.3	0.5
1.44	360	1017.9	1.0
1.82	405	1288.2	1.5
2.25	450	1590.4	

As: Enclosed Area by Shell
Ao: Area of Bearing Plate
Ds: Diameter of Shell
Ls: Embeded Length of Shell
Do: Diameter of Bearing Plate

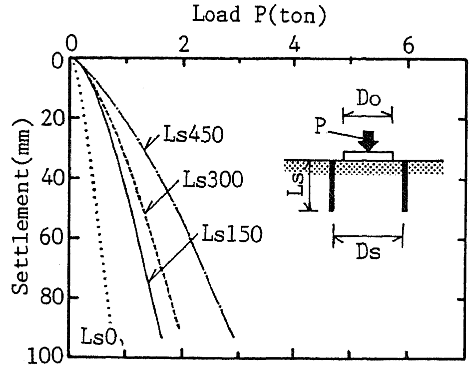


Fig.6 Load-Settlement Curve of Different Length of Shell (Ds=315)

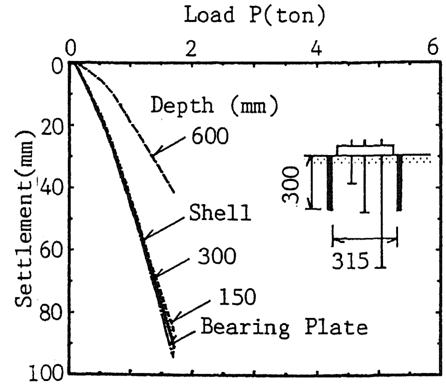
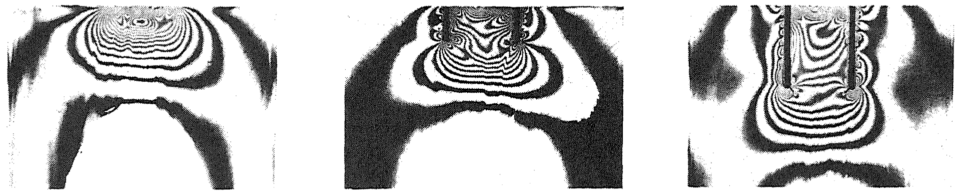
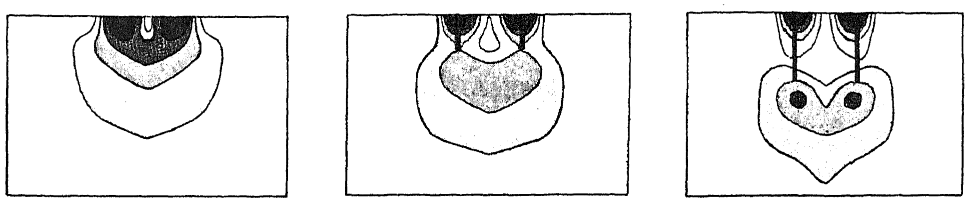


Fig.7 Load-Settlement Curve at Different Depth in Subsoil



L0 L12.5 L25

Photo.1 Fringe Pattern of Photo-elasticity Experiment



L0 L12.5 L25

τ_{max} (kg/cm²) : 0.5-1.0 1.0-1.5 1.5-2.0 2.0-

Fig.8 Distribution of Maximum Shear Stress by F.E.M

soil beneath the bearing plate, together with the settlement of shell and that of the bearing plate. When the subsoil is encompassed with shell, the settlement of the bearing plate is equivalent to that of subsoil displacement, indicating that the encompassed soil subsided as one unit. In the case of plain foundation, it has been observed that the subsoil displacement is smaller than that of bearing plate, and its amount decreases in proportion to depth.

Because it is difficult to obtain stress distribution of the subsoil by model loading test experimentally, the stress distribution has been analyzed by means of photoelastic method and F.E.M. Analysis under axi-symmetrical condition.

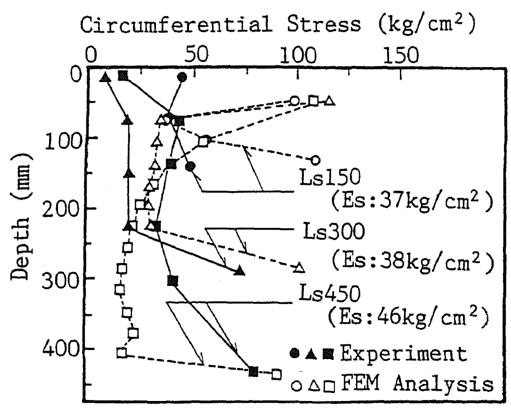


Fig.9 Circumferential Stress on Shell Foundation

Photo 1 is the fringe pattern which appeared on plain foundation and Shell Foundation. The fringe represents the deviator stress($\sigma_1-\sigma_3$), proportional to the maximum shear stress.

Subglobular distribution of stress beneath the bearing plate, highly concentrating on its perimeter, is observed for the case of plain foundation. When the shell is installed, the stress radiates from the horizontal plane at the bottom of shell.

Closely packed fringes is observed on the outer rim of the shell which represents the frictional stress and the end bearing stress of ground.

Fig.8 shows the analytical result obtained from F.E.M. analysis. The distribution of maximum shear stress coincided with the fringe patterns obtained from the photoelastic experiment. Thus, it can be concluded from the F.E.M. analysis and photoelasticity experiment that Shell Foundation, whereas the shell confines the lateral movement of soil, has the same effect as placing the bottom of the footing deep into subsoil.

Fig.9 shows the distribution of circumferential stress of shell obtained from the model loading test using shell 315mm in diameter(Ds315). Also the value obtained by inverse calculation using the experimental data is indicated in the figure.

The elastic modulus of soil(Es) has been adapted analytically as to satisfy the load-settlement curve. The modulus(Es) increase in direct proportion to the length of shell.

The results indicate that the length of shell should be regarded as an important factor in confining the subsoil. Irrelevant to the length of shell, the circumferential stress of shell reaches maximum value near the tip of shell. The reason for this can be designated to the fact that high degree of load is transferred from the lower edge of shell and therefore stress to confine the soil reaches maximum near the tip of shell.

CONCLUSION

A comparison of bearing capacity of Shell Foundation with that of ordinary spread foundation under dynamic and static condition has been discussed. Following conclusions were obtained from the model vibration and vertical loading tests.

1)The installation of shell results in minimizing the settlement when the shear strength and shear modulus is low under vibratory conditions.
2)The effect of minimizing settlement shows its significance with the extended length of shell.
3)Shell Foundation transfers the ground surface load deep into subsoil through a media of encompassed soil.

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