SHELL TYPE FOUNDATION CONSTRUCTION FOR EARTHQUAKE RESISTANCE

by J. K. Minami*

INTRODUCTION

Structural damage to industrial plants and other structures located conveniently to harbors, railways, and other arteries of transportation occurs frequently during destructive earthquakes due to detrimental movement of the footings. In some cases, such displacements amount to several feet in both the vertical and horizontal directions. Foundation movements of such magnitude cause wooden columns to break at the junction with the knee-braces and cause rivets to snap and steel members to distort with resulting damage to the total structure. Damage to industrial structures of wood and steel construction, due to foundation subsidence, was widespread at the time of the To-nan-kai earthquake of December 7, 1944. The epicenter of this earthquake was located 80 miles south of Nagoya at sea. This tremor caused extensive damage in the prefectures of Aichi and Shizuoka, the number of totally or partially damaged buildings amounting to fifty thousand. Rehabilitation of damaged structures due to the sinking of the foundations is troublesome and costly.

Damage of this nature may be minimized by increasing the effective depth of footings and, in addition, by preventing lateral escape of the soil immediately beneath the loaded area. This may be accomplished effectively by installing a thin-walled hollow-shell under and around the footing to be reinforced and protected, without actually increasing the depth of footings. The basic concept of shell type foundation construction is shown in Figure 2. It is evident that the bearing capacity of this construction would be greater than for the footing without the shell due to: 1) the soil surcharge above the lower level of the shell; 2) skin friction between the shell and the soil on the shell surface; 3) mobilization of the shearing resistance of the soil in the case of general shear failure; and 4) increased bearing capacity of the soil with increased depth resulting from the use of the shell.

This paper presents some of the results of investigations on the shell type foundation performed in the laboratory at Waseda University and in the field, with the hope that this type of foundation construction may prove useful under suitable soil conditions in regions outside of Japan.

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* Professor of Structural Engineering, Waseda University, Tokyo (on leave); Fulbright Visiting Lecturer at Univ. of California and Univ. of Illinois during leave of absence.

EARTHQUAKE EFFECTS ON SOILS AND FOUNDATIONS

RESEARCH PROGRAM

Exploratory experiments in the laboratory were performed during the summer of 1947 with empty beer cans and dry sand as the test soil. The general trends and relations were so encouraging that a program of investigation was established and performed during 1948-1949. The program included the following projects:

1. General Study on Models of Shell Foundation Construction, Including Photo-elastic Study.
2. Determination of Optimum Area and Length Ratios for Bearing Capacity Increase.
3. The Case of Eccentric Loads.
4. The Case of Inclined Loads, Including the Use of Projecting Stabilizing Connectors.
5. Comparison of Bearing Capacity of Shell Foundation and Pier Foundation.
6. Double-Shell Construction and Miscellaneous Tests.
7. Pressure Distribution on the Inside Surface of Shells Measured with Magnetostriction Pressure Gages.
8. Field Tests at Okuma Garden, Waseda University, Using a Rotating Type Loading Machine.
9. Field Tests at Kudan-Ue, Tokyo.
10. Field Tests at Toyama-cho Site, Waseda University.

LABORATORY INVESTIGATIONS

A simple lever-type testing machine was used for small model tests. A wooden box of inside dimensions of 65 x 65 x 65 cm provided with a front surface of glass for observation purposes was filled with sand to an approximate depth of 60 cm, in three layers of 20 cm each, which was tamped 100 times per layer with a steel rod of 16 mm diameter. The sand used had an effective diameter of 0.19 mm and an uniformity coefficient of 2.4. Pieces of steel angle shapes of approximately 60 cm lengths were used to apply the load and the counterweight consisted of a box filled with old nails and bolts. (See upper two photographs, Fig. 1; lower photograph shows the rotating type loading machine used in field tests).

Round steel bearing plates of 50 and 100 sq cm, designated R50 and R100, respectively, were found most convenient to use. Shells of four different enclosed areas, 40, 80, 120, and 160 sq cm, and four different lengths, 5, 10, 15, and 20 cm, designated A, B, C, and D, respectively were made from thin steel sheets. The designation R50C160 would indicate that a round bearing plate of 50 sq cm was used with a shell of an enclosed area of 160 sq cm and 15 cm length. (See Fig. 3).

A micro-cathetometer capable of measuring both vertical and horizontal displacements to 1/20 mm was used to follow the movement of the bearing plate with the target point fixed on the loading post of the test equipment.

In order to perform tests under conditions as similar as possible with one another, the sand was emptied after each test and refilled and tamped in the predetermined manner.

Two or more tests were performed for each combination of a given bearing plate and a shell of given dimensions, and the load-settlement curve constructed using average values.

SUMMARY OF TEST RESULTS

Project 1 indicated much greater bearing capacity for the shell type construction than for the ordinary footing construction. Bearing plates, R50, R100, and R200 were used with shells of small, medium, and large sized, designated S, M, and L, respectively, in three different lengths for each shell size. Only the effect of shell length, with area-ratio constant, was studied in this series of tests. It was found that the bearing capacity increases with longer shells, at a constant value of the area-ratio, due to the fact that the shell transmits the foundation load to the lower open end of the shell and also prevents lateral escape of the soil within the shell.

Project 2 was undertaken to study the effect of the area-ratio and the length-ratio on the bearing capacity increase relationship. The test results are shown in the figures as follows:

Fig. 4	Settlement Curves for	R50A(40, 80, 120, 160)
Fig. 5b	" " "	R50B(40, 80, 120, 160)
Fig. 5c	" " "	R50C(40, 80, 120, 160)
Fig. 5d	" " "	R50D(40, 80, 120, 160)

The influence of the area-ratio and length-ratio on the bearing capacity relations was studied separately at first. The effects of the two ratios on the bearing capacity increase relationship was further analyzed and it was possible to combine the effect of the two ratios into one chart, named BCIC-Chart, which is shown in Fig. 7. The values of BCIC have been computed for a settlement of 6 mm which corresponds approximately to 0.075 of the plate diameter.

The Bearing Capacity Increase Coefficient (BCIC) is defined as the ratio of the bearing capacity of the shell foundation supporting a footing of a given area to the bearing capacity of the ordinary footing of the same area (without the shell), at a given value of the settlement for both cases which is well below the yield point on the load-settlement curves for both types of construction.

It has been found that there are two factors which influence the bearing capacity increase coefficient values, namely, the area-ratio and the length-ratio. The area-ratio, A_r, is defined as the ratio of the enclosed area of the shell to the area of the bearing plate at the upper level of the shell; and the length-ratio, L_r, is the ratio of the length of the shell to the diameter of the bearing plate which the shell reinforces. (In the case of a square or a rectangular plate, the diameter of a round plate of the same area to be used).

The curves of equal BCIC values have been drawn from the computed BCIC for shell foundations using 16 shells of differing area and length ratios shown in Fig. 3. The slope of these curves increases as the area ratio is increased up to an optimum value but, for short shells, the slope decreases after the critical area ratio is exceeded. This is apparent from the settlement curve for R50A160 in Fig. 4. In the lower stress range, a short shell of large enclosed area is initially effective but at higher stress ranges, the short shell cannot restrain the lateral escape of the soil from the under edge of the shell. The phenomenon of a sudden loss of support noted for R50A160 no longer occurs when the shell length is increased to 10 cm, as shown by the settlement curve for R50B160 in Fig. 5b.

BCIC-Chart was constructed from tests using a larger bearing plate, R100, and the general shape and characteristics were found to be similar to the BCIC-Chart shown in Fig. 7. It has been found possible to construct similar charts for other soil types having different soil properties encountered in the field. Such charts enable reliable design of shell foundations when the desired value of BCIC has been established.

Another point of interest that may be mentioned is the fact that in order to obtain significant increase in the bearing capacity from piles, it would be necessary to use a large number due to the low values of the area-ratio of the piles under large footings which they support.

Actual foundations are generally subjected to bending moments due to horizontal loads or from the eccentricity of foundation loads. Project 3 for the case of eccentric loadings gave results indicating that the shell foundation settles less than the ordinary footing and also the inclination of the bearing plate is reduced by shells. The shells also impart greater resistance to horizontal displacement which is reflected in smaller bending moments resulting from eccentricity.

Project 4 on inclined loadings also gave results indicating less settlement for shell foundations. Insertion of a connector with projecting wings between the bearing plate and the shell provided greater stability to the construction by preventing overturning, thus permitting application of greater loads. The inclination of the plate is greatly reduced with shells, especially when provided with stabilizing connectors. The projecting wing outside the shell need not be great to be effective.

As experimental investigation progressed, it appeared more and more likely that a shell foundation should possess a bearing capacity as high as that of a pier of the same outside dimensions. Project 5 was undertaken to investigate this problem. A series of tests were conducted using solid wooden piers of oak of four different lengths and 80 sq cm in cross section. The bearing capacities of pier and shell foundations were compared and the bearing capacity increase relationship curves have been plotted as shown in Fig. 8, as follows:

Fig. 8a Load-Settlement Curves for Pier Foundation.
Fig. 8b " " " " Shell "
Fig. 8c Comparative BCIC Values for Pier and Shell Foundations.

The results of this investigation are of practical significance in indicating the possibility of effecting economy in the use of cement for the core of concrete piers. The replacement of concrete by incompressible soil mixture in the core would result in reduced dead load and would increase the supporting capacity. The transmission of the load by the pier to the soil may be visualized as one of "hard" transmission from concrete to the soil whereas the load transmission in shell type construction is a "soft" type transmission, from the soil to soil.

The study of double shell construction in Project 6 disclosed that only the outside shell is effective and the smaller and longer inside shell is ineffective.

The horizontal pressure distribution on the inside surface of shells was measured with magnetostriction pressure gages. The maximum average values did not exceed 20 per cent of the vertical pressure, resulting from the applied load on the bearing plate. Concentration of pressure was noticeable near the top of the shell. An adoption of 0.33 and 0.25 as coefficients of horizontal pressure for the end portions and the center, respectively, seems adequate for practical engineering purposes.

FIELD INVESTIGATIONS

The results of laboratory investigations far exceeded the expectations of the author and it was deemed desirable to investigate further in the field using larger bearing plates and shells. Projects 8, 9, and 10 were undertaken with this view in mind. In order to **facilitate** conducting numerous tests to study the influence of different area-ratio and length-ratio on the bearing capacity, a rotating type loading machine was conceived and constructed which would permit testing six or more areas with one setting up of the equipment. It is essentially an enlargement of the lever type equipment used in the laboratory. The machine under working conditions is indicated in the lower photograph in Fig. 1. Such a machine could be improved and made mobile by mounting on a truck for soil investigations.

Bearing plates of 800, 1600, and on some occasions 2000 sq cm and shells of more than 100 cm length and 3000 sq cm enclosed area were used. The BCIC-chart may be constructed by performing a test with the bearing plate without the shell and the load-settlement curve for this case would become the standard settlement curve. Three additional loading tests using shells of different area and length ratios would be necessary to obtain BCIC-values for them. Smooth curves for equal values of BCIC may then be plotted, based upon previously determined BCIC-values. An example of shell foundation design based on the use of a BCIC-chart is shown in Fig. 10.

DESIGN, WITHOUT THE USE OF BCIC-CHARTS

There are instances when shell foundations must be designed, although results of loading tests to construct a BCIC-chart are not available. The procedure would be to compute the bearing capacity of foundations using several shells of different sizes and select the one which is most suited to the conditions of the problem. The approximate equation for the bearing capacity of a shell foundation may be expressed by the following equation: (see lower portion of Fig. 2)

$$Q_s = mQ_o = \pi R_s^2 f'_e + \gamma\pi(n^2-1)\, R_s^2 D_s + 2\pi R_s L_s f_s + 2\pi n R_s D_s\, \tau$$

wherein

Q_s = bearing capacity of the shell foundation

Q_o = bearing capacity of the ordinary footing

m = BCIC = Q_s/Q_o

R_s = radius of the shell

f'_e = bearing capacity of the soil at the level of shell bottom

γ = unit weight of the soil

nR_s = radius from the center line of the shell to the plane of rupture of the soil at failure

D_s = the depth to the lower edge of the shell from the ground level

L_s = length of the shell

f_s = unit skin friction of the soil on the shell surface

τ = unit shear resistance of the soil at failure

The first term on the right-hand side of the equation represents the bearing capacity component at the lower end of the shell; the second term represents the bearing capacity increment due to soil surcharge; the third term is the bearing capacity component due to skin friction on the shell surface; and the fourth term represents the bearing capacity component due to shear resistance on the plane of rupture at failure.

Most building codes do not permit the addition of the bearing and friction components of bearing capacity in computing the design bearing capacity of foundations, except under specific conditions. A friction type foundation is generally less reliable than a bearing type foundation. This applies to both the shell type or the pile type foundations, although friction type foundations can be designed conservatively to give satisfactory performance.

In designing a bearing type shell foundation, i.e., when the lower edge of the shell has reached ordinarily satisfactory soil layer, such as loam, it would be safe and conservative to consider only the first and second terms and neglect altogether the third and fourth terms on the right-hand side of the equation.

The allowable bearing capacity of a shell foundation should be based on the use of allowable unit bearing capacity of the soil at the lower level of the shell and a conservative value for the unit weight of the soil and a proper value of n. A value of $(n^2-1) = 10$ appears reasonable and safe.

Shell type foundations for four story reinforced concrete apartment houses in the Tokyo-Yokohama area, courthouses of reinforced concrete construction in Fukui and Maebashi, and others which have been designed on this basis have given satisfactory performance to date without detrimental settlement and no earthquake damage. Cylindrical reinforced concrete pipes manufactured by utilizing centrifugal forces which are commonly used for water-mains and sewage work have been used with good results.

A suitable mechanical device for the removal of the inferior soil in-situ, placing of the shell in the excavated hole, and filling the inside of the shell with a dense mixture of incompressible sand and gravel on which to place the footing would make for easy and good construction.

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NOMENCLATURE

Explained as they first appear in the text.

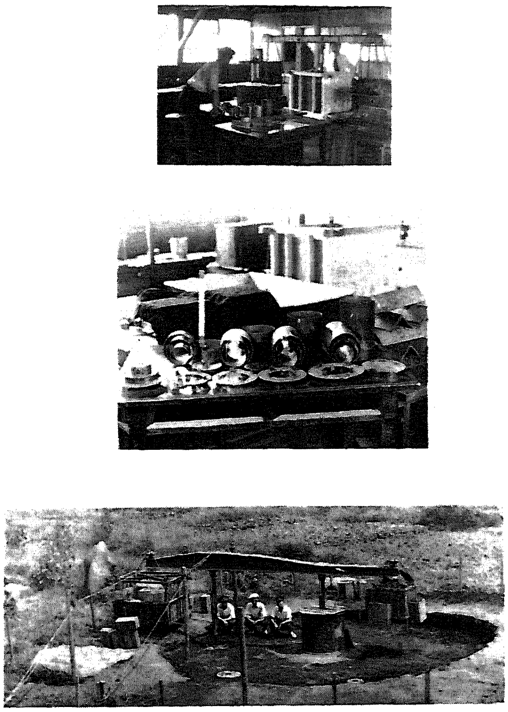
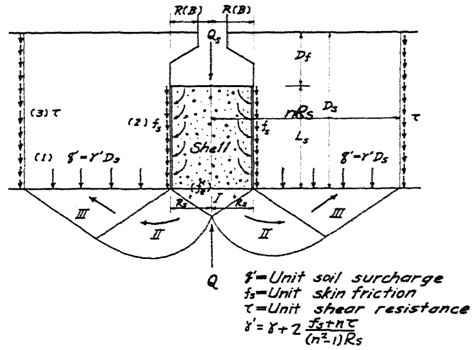


Fig. 1. Laboratory and Field Testing Equipment



The bearing capacity of shell foundations is increased due to:

1) *Soil surcharge above the lower level of the shell;*
2) *Skin friction between the shell and the soil on the shell surface;*
3) *Mobilization of the shearing resistance of the soil in the case of general shear failure.*

For approximate equation for the bearing capacity, in general shear failure.

$Q_s = mQ_o - \pi R_s^2 f_s' + \gamma\pi(n^2-1)R_s^2 D_s + 2\pi R_s L_s f_s + 2\pi n R_s D_s \tau$

where m *- BCIC,* Q_o *= B.C. of Footing at Surface*

Fig. 2 Concept of Shell Foundation

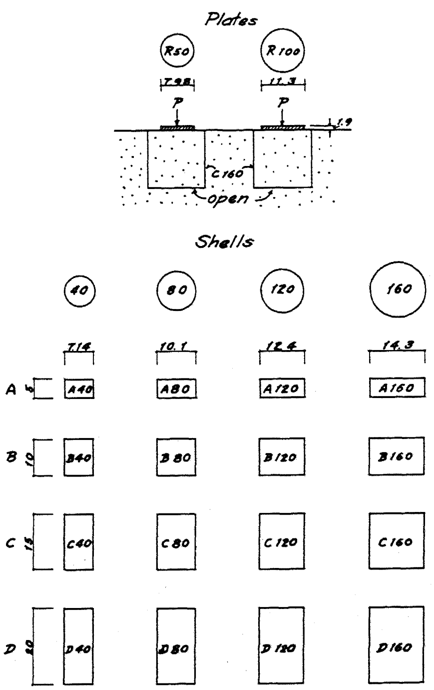


Fig. 3 Dimensions of Plates and Shells (Unit: cm²; cm)

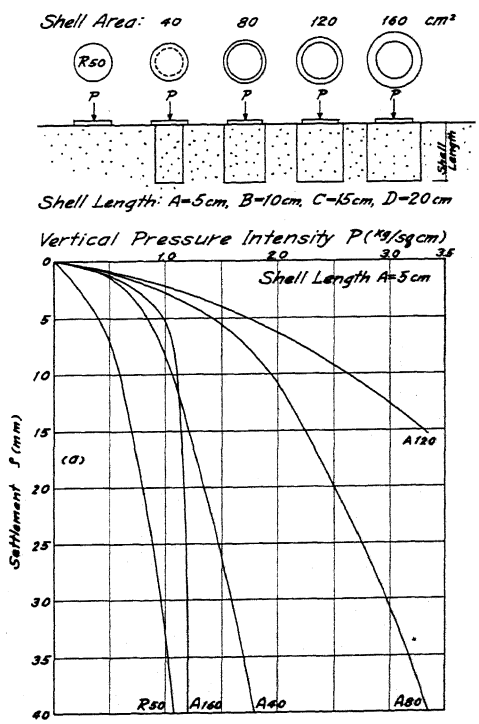


Fig. 4 Settlement Curves for R50 A(40, 80, 120, 160)

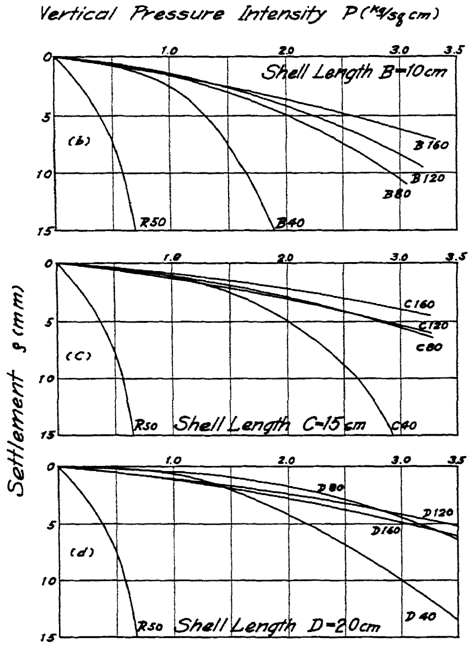


Fig. 5 Settlement Curves for
b) R50 B (40. 80. 120. 160)
c) R50 C (40. 80. 120. 160)
d) R50 D (40. 80. 120. 160)

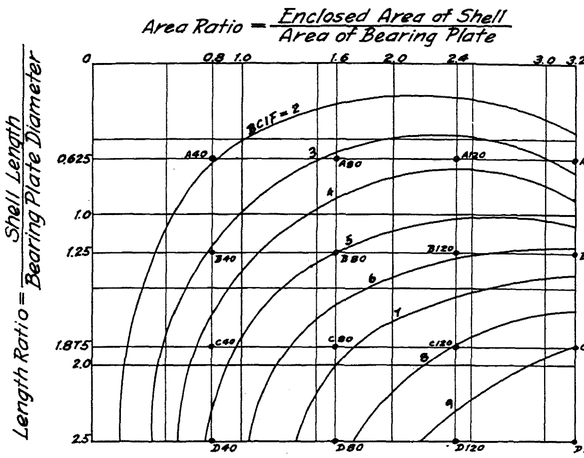


Fig. 7 Bearing Capacity Increase Coefficient Chart

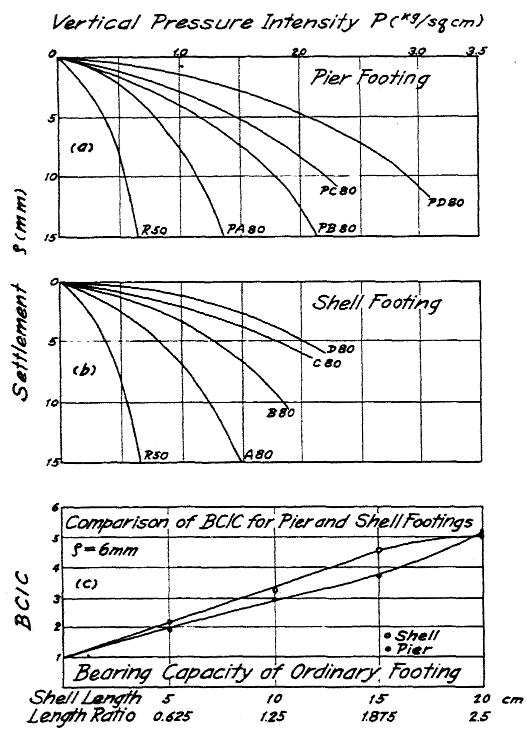


Fig. 8 BCIC for Pier and Shell Foundations

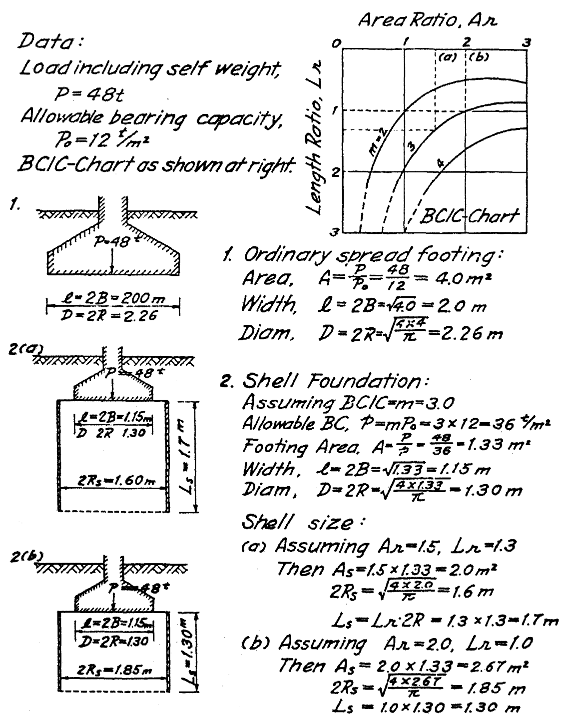


Fig. 10 Shell Foundation Design