Strength and Ductility Characteristics of Highway Bridge Foundations

by

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ABSTRACT

The check of ductility and dynamic strength for highway bridge foundations has been proposed based on the results of research into the behavior of foundations during severe deformation. For seismic design method of foundations, analytical model of pile foundations and caisson foundations were proposed accounting for the non-linear properties of the horizontal ground resistance, vertical ground resistance and the bending stiffness of the foundation body. Horizontal loading tests of pile foundation model were performed to clarify the foundation ductility factor - degree of pile body damage relationship.

Key Words:Seismic Design, Bridges,
Strength, Ductility,
Pile Foundations,
Caisson Foundations,
Spread Foundations

1. INTRODUCTION

The Hyogo-ken Nanbu Earthquake of January 17, 1995 caused severe damage to highway bridges, including collapsed bridge piers and bridge falls at many locations. Shortly after the earthquake, the state of this damage was surveyed by visually examining the bridge piers and other above ground structures and emergency restoration work was performed to prevent secondary damage. Because the foundations are underground, it was impossible to directly assess their damage immediately after the earthquake. Surveys of damage to the foundations conducted in conjunction with the restoration work confirmed that little damage was suffered by the foundations(1).

After the earthquake, the "Committee for Investigation on the Damage of Highway Bridges Caused by the Hyogo-ken Nanbu Earthquake" was formulated in the Ministry of Construction to survey the damage and clarify the factors which contributed to the damage. On February 27, 1995, the Committee approve the "Guide Specifications for Reconstruction and Repair of Highway Bridges Which Suffered Damage due to the Hyogo-ken Nanbu Earthquake(2)", and the Ministry of Construction noticed on the same day that the reconstruction and repair of the highway bridges which suffered damage by the Earthquake shall be made by the Guide Specifications. The design method that accounts for the non-linear behavior of foundations based on the results of research at the Public Works Research Institute and elsewhere was included in the Guide Specifications.

This design method, one based on the fact that a foundation provides greater horizontal strength than a bridge pier body, is difficult to apply in a case where the ground around the foundation will liquefy or a case where the bridge pier body has ultimate horizontal strength sufficiently great to withstand the design horizontal seismic coefficient. Thus when the Specifications for Highway Bridges were revised, a model experiment concerning the strength and ductility

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of foundations was carried out to propose an seismic design method for foundations.

This paper describes experiments on strength and ductility characteristics of highway bridge foundations conducted by the Public Works Research Institute.

2. FUNDAMENTALS OF SEISMIC DESIGN OF FOUNDATIONS

Damage inflicted on foundations of highway bridges by the Hyogo-ken Nanbu Earthquake was relatively minor. In many cases, the damage was not severe because the horizontal strength of the foundation was greater than that of the bridge pier body or because the foundations provided sufficient ductility to withstand the decline in the horizontal strength of the foundation caused by liquefaction. So judging from the state of the damage, the seismic design method in the present Specifications is a satisfactory design method for foundations.

But during the Hyogo-ken Nanbu Earthquake, load greater than the conventional design seismic force acted on these structures, and the balance of the strengths of the various parts of the bridge and the strength of the foundations have come to differ as, for example, bridge pier bodies designed considering this provide greater strength than in the past. And because it is difficult both to survey the damage to underground foundations and to reinforce these foundations, it is necessary to design foundations so they will not suffer severe damage. So when performing the seismic design of foundations, it should be done in conformity with the following guidelines.

It should be based on the fact that the plastic deformation of the base of a bridge pier body, whose damage can be studied and which can be restored with relative ease, absorbs earthquake energy. For this reason, the horizontal strength of the foundation must be equal to or greater than that of the bridge pier body (See Figure 1 (a)).

In a case where a bridge pier body such as a wall-type bridge pier which has sufficiently great ultimate horizontal strength in the direction at right angles to the bridge axis against the design horizontal seismic coefficient for check of ductility and dynamic strength, it is not necessarily rational to design the horizontal strength of the foundation so that it is equal to or greater than the ultimate horizontal strength of the of the bridge pier. And when liquefaction occurs, the decline in the bearing capacity of the ground around the foundation causes a decline in the strength of the overall foundation, but even in this case, if the horizontal strength of the foundation is greater than the ultimate horizontal strength of the bridge pier body, the structural cross section is excessive, making this an irrational overall bridge design.

Consequently, when designing a foundation in such cases, it is more rational to allow for plasticization at and beyond yield within a range where excessive damage will not be inflicted on the foundation body even when the horizontal strength of the foundation is lower than the ultimate horizontal strength of the bridge pier body. In such cases, the response ductility factor (response horizontal displacement / yield horizontal displacement) of the foundation under the horizontal force corresponding to the design horizontal seismic coefficient for check of ductility and dynamic strength is calculated, and it should be checked to make sure it is below the limiting value (Figure 1(b)).

3. PILE FOUNDATIONS

3.1 Analytical Model of Pile Foundations

In order to check of dynamic strength and ductility for pile foundations, it is necessary to calculate the horizontal strength of pile foundations and its behavior during severe deformation. The displacement method stipulated in the present Specifications handles the behavior of the foundation as linear behavior by restricting the displacement of the foundation within a range where it is assumed to be elastic behavior. It is impossible to ignore the effects of the non-linear properties of the behavior of the foundation when studying the behavior of the bridge pier during severe deformation. So when check of ductility and dynamic strength for pile foundations, it is necessary to consider the following items as non-linear properties of the behavior of a pile foundation.

[1] Vertical resistance of the piles
[2] Horizontal resistance of the piles
[3] Bending moment - curvature relation of the pile bodies

The horizontal strength of a pile foundation and its behavior during severe deformation can, as shown in Figure 2, be analyzed by replacing the pile foundation with a rigid frame structure supported by a ground spring and considering this to be non-linear.

(1) Vertical Resistance of the Piles

The vertical force at pile top - vertical displacement relationship for a pile generally reveals non-linear behavior. The following method is considered as a method for modelling this kind of non-linear behavior for design purposes.

[1] Approximating the vertical force - vertical displacement curve at the top of the piles with a function (Weibull function curve formula for example).
[2] Modelling the pile tip resistance and the pile surface resistance respectively as elasto-plastic forms.
[3] Modelling the vertical force - vertical displacement curve at the top of the piles with a elasto-plastic form.

At the Public Works Research Institute, the results of vertical loading tests of many piles were organized to propose modelling based on the above methods(3). Considering convenience in actual design work and the precision of the design constant estimation, the following model was used for the design calculations.

The resistance properties in the vertical direction of the pile were considered to be elasto-plastic form, with the pile axial direction spring constant K_{VE} considered to be the initial gradient, and the ultimate bearing capacity P_{NU} and the ultimate pull-out bearing capacity P_{TU} treated as the upper limit values.

In this case, the pile's axial direction spring constant K_{VE} was assumed to be the pile axial direction spring constant K_V calculated as stipulated in the Specifications.

The ultimate bearing capacity P_{NU} and the ultimate pull-out force P_{TU} were considered to be the ultimate bearing capacity of the pile R_U calculated as stipulated in the Specifications and (ultimate pull-out force of the pile P_U) + (effective weight of the pile W). These values did not exceed the ultimate bearing capacity and ultimate pull-out force determined from the stress of the pile body.

(2) Horizontal Ground Resistance

1) Non-linear Properties of the Ground Resistance

The ground reaction force properties should be faithfully modelled so that the horizontal ground reaction force p - horizontal displacement y relationships at varying depths in the ground is the p - y curve used for DNV etc.(4). But when the ground survey precision and the convenience of the calculation in the design task are considered, it is not necessarily appropriate to apply this method to the design of bridge foundations.

Consequently, the horizontal ground resistance was estimated by modelling as a bilinear model of the kind shown in Figure 2 (c). The effectiveness of this analysis method was confirmed and confirming the effectiveness of this analysis method based on an analysis of the results of horizontal loading tests(5).

2) Effects of the Pile Group

Experiments concerning the effects of the pile group have been performed to clarify the horizontal resistance of the pile under various relationships. At the Public Works Research Institute, steel pipe piles with a diameter of 102 mm were arranged as single piles, in straight lines, side by side, and as vertical and batter pile groups in test ground prepared inside a soil tank, then horizontal loading testing was performed on these piles(6). When proposing a design method, the results of horizontal loading tests of pile groups of these kinds are analyzed, and the coefficients η_k of the coefficient of horizontal ground reaction and η_p of maximum value of horizontal ground reaction are introduced.

3) Calculation Method

Based on 1) and 2), the horizontal ground resistance is found as follows.

The horizontal ground resistance properties is considered to be elasto-plastic in form with the coefficient of horizontal ground reaction k_{HE} as the initial gradient, and the maximum value of the horizontal ground reaction force p_{HU} to be the upper limit value (same figure (c)). k_{HE} and p_{HU} can be found from the following formulae.

$$k_{HE} = \eta_k \alpha_k k_H \qquad (1)$$

$$p_{HU} = \eta_p \alpha_p p_U \qquad (2)$$

Where:

- k_{HE} : Coefficient of horizontal ground reaction used for the check of ductility and dynamic strength (kgf/cm^3)
- p_{HU} : Maximum value of the horizontal ground reaction force (kgf/cm^2)
- k_H : Coefficient of horizontal ground reaction (kgf/cm^2), found as stipulated in the Specifications.
- p_U : Passive earth pressure during an earthquake (kgf/cm^2), found as stipulated in the Specifications. It can account for the weight of the soil up to the ground surface which is stable over a long period around a foundation as the overburden load.
- α_k : Coefficient for the coefficient of horizontal ground reaction of a single pile.
- α_p : Coefficient for the maximum value of the horizontal ground reaction force for a single pile.
- η_k : Coefficient for the coefficient of horizontal ground reaction accounting for the pile group effects.
- η_p : Coefficient for the maximum value of the horizontal ground reaction force accounting for the pile group effects.

Table 1 presents the values of the coefficient α_k of the coefficient of horizontal ground reaction and the coefficient α_p of the maximum value of the horizontal ground reaction force of a single pile. This was obtained based on an analysis of the results of the horizontal loading test of a single pile shown in 1).

The coefficient η_k for the coefficient of horizontal ground reaction accounting for the pile group effects is the following value.

$$\eta_k = 2/3 \qquad (3)$$

The coefficient η_p for the maximum value of the horizontal ground reaction force accounting for the pile group effects is the following value.

Cohesive ground : $\eta_p = 1.0$ (4)

Sandy ground : $\eta_p \cdot \alpha_p =$ Interval between the centers of the piles in the direction at right angles to the loading direction / pile diameter ($\leqq \alpha_p$) (5)

In the case of piles other than those in the front pile in sandy ground, the value is 1/2 of that indicated in (5).

The values of η_k, η_p are found from the results of the analysis of the loading tests of group piles in 2).

(3) The Bending Moment - Curvature Relationship of a Pile Body

When using the displacement method, a pile body is treated as an elastic body in order to keep the stress of the reinforcing bars etc. of the pile body from exceeding the allowed stress. When calculating the horizontal strength of a pile foundation, it is important to evaluate the behavior of the pile body after yield and the ultimate strength. It is necessary to appropriately model the bending moment - curvature relationship in the pile body.

The points of inflection in the bending moment - curvature relationship when bending moment acts on a concrete pile are usually treated as the cracking time (C), the time when the reinforcing bars begin to yield (Y), and the ultimate time of the section (U). The analytical model was a trilinear model linking these points found for a circular steel reinforced concrete cross section with the axial direction N and subjected to the bending moment M (Figure 2(d)).

Because points of this sort are not clearly revealed on a steel pipe pile, it is modelled as a bilinear model with the point at which the stress of the outermost edge of the steel pipe pile reaches the yield point considered to be the initial gradient, and the plastic moment treated as the maximum value (same figure (e)).

The horizontal load - horizontal displacement curve obtained based on the horizontal loading testing of cast-in-place piles shown in Figure 3 and the results of its analysis are shown in Figure 4. Here, the ground resistance properties are treated as completely elasto-plastic type. An analysis treating the pile body as linear revealed conformity in the area where the amount of deformation is small, but as the amount of deformation rises, the values diverge. This suggests that when the non-linear properties of the bending stiffness of a pile are considered, it is possible to perform a simulation up to the large deformation stage.

3.2 Loading Tests of Limiting Value of Ductility Factor of Pile Foundations

(1) Test Objectives

In the case of a wall type bridge pier at right angles to the bridge axis or a case where liquefaction will occur, it is rational to design the bridge so that the energy is absorbed by the foundation. Even in these cases, it is necessary to design the bridge so that the response ductility factor of the foundation is within a certain value in order to prevent excessive damage to the foundation body.

As a limiting value for this ductility factor, the safety coefficient for the ultimate was used to determine the allowed ductility factor for a reinforced concrete bridge pier. Even when a member which is part of the pile body of a pile foundation reaches the ultimate state, this does not result in an immediate conspicuous horizontal strength decline in the overall foundation system. It is difficult to define the ultimate for an entire pile foundation system. For this reason, loading tests of a model pile foundation is done to clarify the ductility factor of the foundation and the state of damage to the pile body. This information is used as reference data when determining the limiting value of the ductility factor.

Loading tests were performed using PHC, cast-in-place piles, and steel pipe piles which are used for pile foundations of highway bridges. The following section refers to the tests of cast-in-place piles.

(2) Test Method

Figure 5 shows an overview of the loading method. This testing, which is done to clarify the state of damage to a pile body when horizontal force acts upon pile groups, requires that moment and horizontal force act on the model simultaneously. The loading was performed by simultaneously controlling three jacks to hold the vertical load at a constant level and performing alternating positive and negative loading of a horizontal force with the horizontal force H/moment M ratio constant at M/H = 2.5 m.

The pile foundation models used were six reinforced concrete piles with a diameter of

30 cm and a length of 180 cm; three set in the loading direction and two at right angles to the loading direction. The interval between their centers was 75 cm, which was 2.5 times their diameters.

The displacement of the horizontal loading position at the level where the axial direction reinforcing bars at the outer edge of the pile bodies of all piles yielded in the previous analysis was set at 1 δ_y, and positive - negative alternating loading was performed while increasing the displacement in increments of 1 δ_y. Three cycles of loading were done for each δ_y. The vertical load was 84 tf.

(3) Test Results

The models of cast-in-place piles were made using 16 reinforcing bars of type SD295A, D10 as the axial direction reinforcement. The hoop ties were made of SD295A, D4 installed at 4 cm intervals. The distance from the outer surfaces of the piles to the centers of their axial direction reinforcement was 5 cm. The results of material testing show that the compressive strength of the concrete was 323 kgf/cm^2 while the yield point of the steel reinforcing bars was 3,757 kgf/cm^2.

Figure 6 shows the hysteresis curve of the horizontal force and the horizontal displacement of the cast-in-place piles. A description of the state of damage to the piles follows.

At 2 δ_y, signs that the covering concrete would soon collapse appeared at the top and bottom bases of the piles at the ends and in the bottom of the piles in the middle, and maximum load was reached at 3 δ_y. At that time, some of the covering concrete at the bottom of pile 1 separated. At 4 δ_y, covering concrete at the top of pile 1, the top and bottom of pile 3, and the bottom of pile 2 separated, but this did not expose any of the reinforcing steel rods (Photograph 1). At 6 δ_y, steel reinforcing rods were exposed and buckled at the bottom of pile 1 and at the top and bottom of pile 3, and at 7 δ_y, steel reinforcing rods were exposed and buckled even at the top of pile 1. As the cyclic loading continued, the hoop ties and axial direction reinforcement failed reducing the horizontal strength.

While not described in this report, similar loading testing was performed using pile group made up of PHC piles and steel pipe piles. Based on the results of these tests and calculation results, the limit value for the ductility factor of a pile foundation has been set at 4.

4. CAISSON FOUNDATIONS

4.1 Analytical Model of Caisson Foundations

The Hyogo-ken Nanbu Earthquake caused only slight damage to caisson foundations, and it is believed that foundations designed using the seismic coefficient method provide sufficient ductility to guarantee stability. But because caisson foundations are generally used as independent column shaped bodies, it is necessary to verify the strength of caisson foundations. The resistance elements are studied again and the resistance properties are modeled as shown below based on its convenience as a design calculation method and the precision of the ground constant estimation method (Figure 7).

(1) Horizontal Resistance of the Surface of a Foundation

Displacement of a foundation is accompanied by compression of the ground at the front surface of the foundation and a subgrade reaction p_{Hr}. The upper limit value p_{HU} of the horizontal subgrade reaction force at the front surface of the foundation at depth z is found from the following formula.

$$p_{HU} = \alpha_p p_p \qquad (6)$$

Where

p_p : Passive earth pressure strength of the ground (tf/m^2)

α_p : Correction factor of the upper limit value of the horizontal subgrade reaction force

$$\alpha_p = 1 + \frac{1}{2}\frac{z}{B^*} \ (\leqq 3.0) \qquad (7)$$

z : Depth from the design ground surface (m)

B : Loading width of the foundation (m)

The correction factor α_p, which accounts for the three-dimensional expansion of the ground resistance at the front surface of a foundation, was set from the analysis and the model experiments. The calculated loading width of the foundation is basically the foundation width, but when the plane shape of the foundation is circular, it is multiplied by 0.8.

The horizontal resistance of the side surface of the foundation has, in the case of a conventional caisson foundation, been considered with the ground resistance of the front surface of the foundation increased by 20%, but the horizontal direction shear subgrade reaction force on the side surface is separated and assessed so that it is possible to deal with various plane shapes. The resistance properties are assumed to be elasto-plasticity type with an upper limit value.

(2) Vertical Resistance of the Side Surfaces of a Foundation

Using a conventional design method, the shear resistance τ_v in the vertical direction of the surfaces of a foundation were ignored, but because according to the shape of the foundation and the execution method, its share of the resistance properties of a foundation is large, it is considered to be the elasto-plasticity type resistance.

(3) Resistance of the Bottom Surface of a Foundation

The resistance of the bottom surface of a foundation accounts for the vertical subgrade reaction force p_{vr} and the horizontal shear subgrade reaction force τ_{sr}. Both subgrade reactions have upper limit values. And in some cases, if rotating displacement occurs on the bottom surface of a foundation, part of it is lifted. For this reason, the resistance of the part which lifts is ignored.

(4) Stiffness of a Foundation Body

Using a conventional design method, a main foundation was treated either as a rigid body or as an elastic body, but it is necessary to also consider behavior following the yield of the foundation body in order to evaluate its strength. For this reason, it is treated as an analysis method which can reduce the bending stiffness according to the bending moment produced in the foundation body. But in order to protect the foundation body from severe damage during an earthquake, the design should limit the stress resultant of the foundation body to less than the yield point of the members.

Under the recent revision of the Specifications for Highway Bridges, the resistance elements of the ground around a foundation are, in principle, considered to be the 6 coefficients of subgrade reaction shown in Figure 8.

[1] Vertical coefficient of subgrade reaction at the bottom surface of a foundation k_V

[2] Horizontal shear coefficient of subgrade reaction at the bottom surface of a foundation k_S

[3] Horizontal coefficient of subgrade reaction at the front surface of a foundation k_H

[4] Horizontal shear coefficient of subgrade reaction at the side surface of a foundation k_{SHD}

[5] Vertical shear coefficient of subgrade reaction at the front surfaces of a foundation k_{SVB}

[6] Vertical shear coefficient of subgrade reaction at the side surface of a foundation k_{SVD}

These ground resistance elements are treated as elasto-plastic bodies with upper limit values.

4.2 Analysis of Loading Tests

Analysis of past horizontal loading tests was performed based on the design calculation model described in 4.1 in order to verify this design model. Two examples of this analysis are shown below.

(1) Caisson Foundation Model Loading Test

A model caisson foundation with an embedment length of 4.8m and a diameter of 1.4m installed on ground formed from alternating strata of sandy silt and clay was tested by applying static loading up to about 30 cm in one direction. A comparison of the results of this experiment with the results of an analysis performed using the method in 4.1 is shown in Figure 9. The figure reveals that these conform closely up to a horizontal displacement of about 20cm, but over 20cm, the analytical value of the yield strength is evaluated lower than the corresponding experimental value. However, the overall behavior generally coincides.

(2) In-situ Horizontal Loading Test of a Wall Foundation

Figure 10 shows the results of an in-situ horizontal loading test and the results of an analysis using the method described in 4.1 for an actual size wall foundation with an embedment length of 23.8m, a thickness of 1.2m, and a width of 2.4m constructed on ground consisting primarily of alternating strata of clay and silty fine sand. The analytical values conform extremely closely with the experimental results up to the point where the main foundation yields, but are a little smaller after yield.

In addition, identical loading testing was performed on three wall foundations etc., and these experimental values also coincide closely with the analytical results.

5. SPREAD FOUNDATIONS

5.1 Behavior of Spread Foundations During Large Deformation

Under the revised Specifications for Highway Bridges, foundations are designed based on the check of ductility and dynamic strength. But stability calculations for spread foundations continue to be based on the seismic coefficient method as in the past; verification based on the check of ductility and dynamic strength is not necessary. There is a reason for this exception. Because spread foundations are usually supported by a good quality bearing stratum, the ground provides more than enough bearing strength. This means that it is possible to count on the foundation lifting to absorb energy, and that even in a case where a spread foundation displays such non-linear behavior, excessive damage will not be inflicted on the ground.

Figure 11 shows the moment - angle of rotation relationship when horizontal force has acted on a spread foundation along with the subgrade reaction distribution at that time. At the stage where the horizontal force is small, the subgrade reaction force distribution is trapezoidal. Next, the subgrade reaction force distribution becomes triangular because when the active moment exceeds the lift limit moment of the foundation, tensile force does not act between the foundation and the ground. The lift limit moment of the foundation and the lift limit angle of rotation at that time are found with formula (8) and formula (9) shown below.

$$M_0 = BV/6 \tag{8}$$

$$\alpha_0 = \frac{12M_0}{B^3 D k_v} \tag{9}$$

Where:

M_0 : Lift limit moment (tf m)

α_0 : Lift limit angle of rotation (rad)

V : Vertical force at the foundation bottom surface (tf)

B : Foundation length in the load direction (m)

D : Foundation length in the load right angle direction (m)

k_v : Vertical coefficient of subgrade reaction at the bottom surface of the foundation (tf/m^3)

The subgrade reaction force distribution is assumed to recover its trapezoidal shape when the subgrade reaction force at the top of the foundation reaches its upper limit.

As a result of trial calculations performed using these methods, a spread foundation design with the seismic coefficient method will, even during a Hyogo-ken Nanbu Earthquake class seismic event, present no problems related to its stability calculations. Therefore, under the revised Specifications, spread foundations are designed based on the seismic coefficient method as in the past.

5.2 Check of Strength of Footing

Turning to the design of footing, the existing design method has been supplemented by a method involving concentrating the actions of the subgrade reactions near the edge of the foundation. This method has been introduced because in the event of action by a powerful earthquake with a low probability of occurring during the years of service of a bridge, the foundation would lift concentrating the subgrade reaction forces near the edge of the footing.

During the revision, the results of trial calculations of the subgrade reaction force distribution based on the method shown in 5.1 were organized, and the design is now done by having the subgrade reaction forces act on a location on the interior side whose distance from the edge of the foundation is only 1/20 of the foundation length (see Figure 12).

6. CONCLUSION

The check of ductility and dynamic strength for highway bridge foundations has been proposed based on the results of research into the behavior of foundations during severe deformation. For seismic design method of foundations, analytical model of pile foundations and caisson foundations were proposed accounting for the non-linear properties of the horizontal ground resistance, vertical ground resistance and the bending stiffness of the foundation body. Horizontal loading tests of pile foundation model were performed to clarify the foundation ductility factor - degree of pile body damage relationship. These tests confirmed that at less than 4 times the yield displacement, the pile bodies are spared serious damage.

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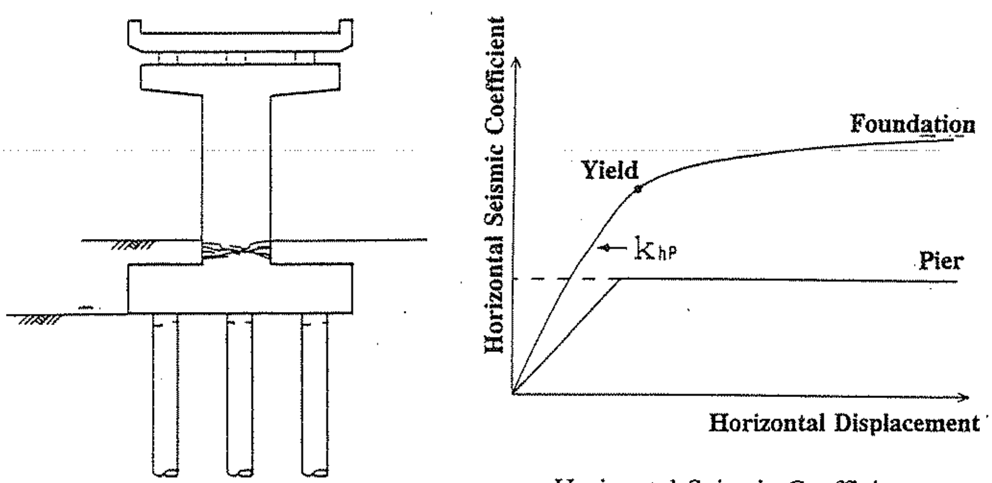
Table 1 Coefficient

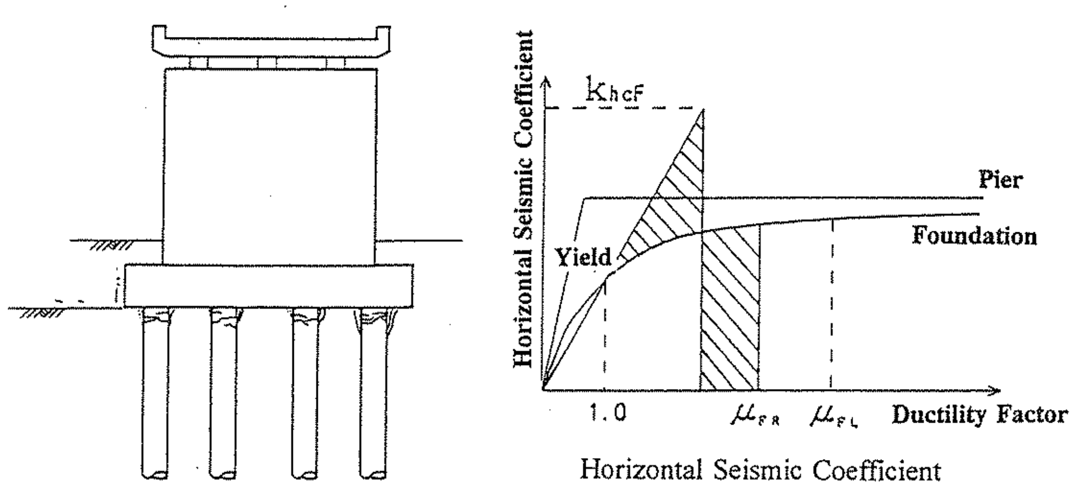
Type of Ground	α_k	α_p
Sandy Ground	1.5	3.0
Cohesive Ground	1.5	1.5

Note: In cohesive ground where N $\leqq$ 2, α_p is assumed to equal 1.0.



Horizontal Seismic Coefficient
- Horizontal Displacement

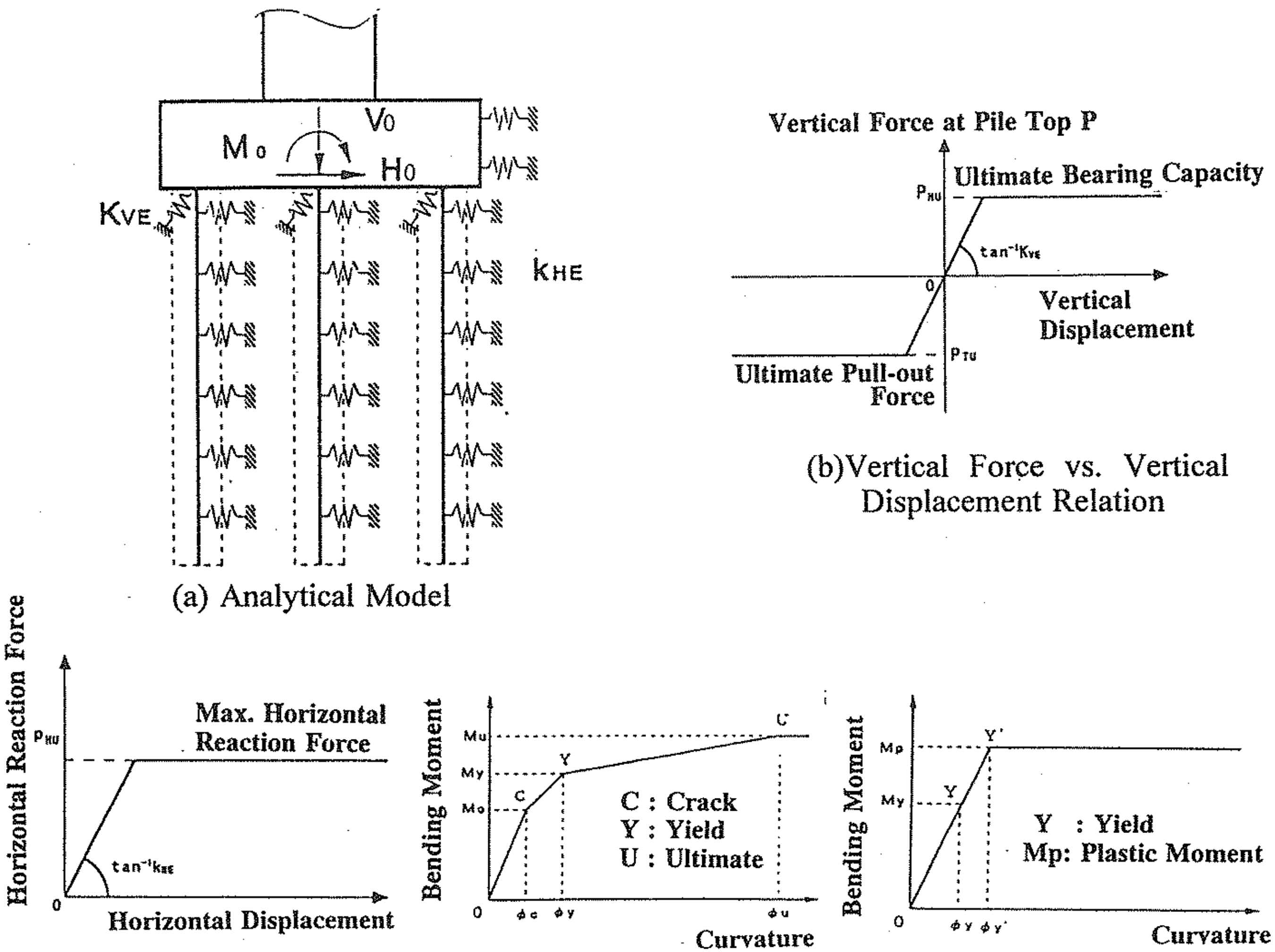
(a) Case of a Principal Plastic Hinge Formed in the Bridge Pier Base



Horizontal Seismic Coefficient
- Ductility Factor

(b) Case of Principal Non-linear Properties in the Foundation - Ground System

Figure 1. Basic Guidelines for the Design of Foundations Using the Check of Ductility and Dynamic Strength



(a) Analytical Model

(b) Vertical Force vs. Vertical Displacement Relation

(c) Horizontal Force vs. Horizontal Displacement Relation

(d) Moment vs. Curvature Relation of Reinforced Concrete Piles

(e) Moment vs. Curvature Relation of Steel Pipe Piles

Figure 2. Analytical Models of Pile Foundations

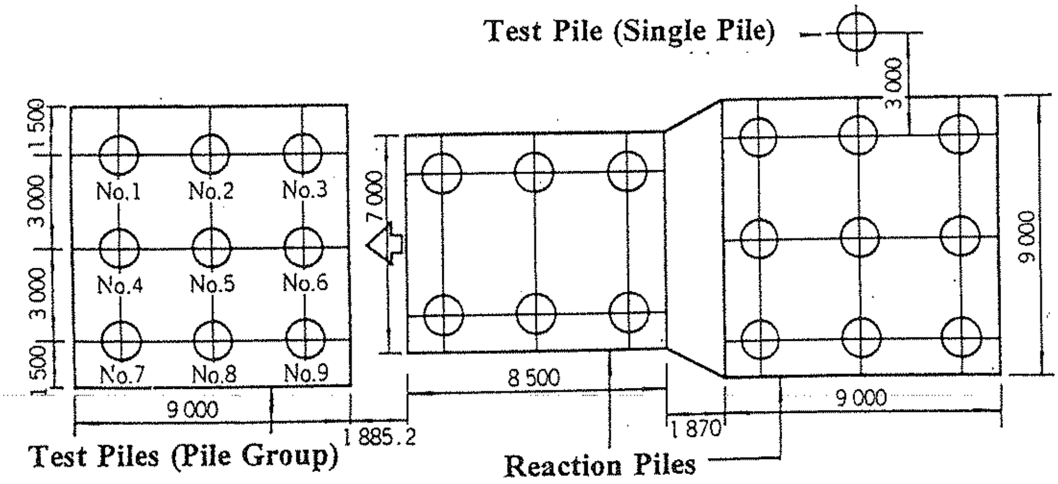


Figure 3. Loading Test of Cast-in-place Pile Foundation

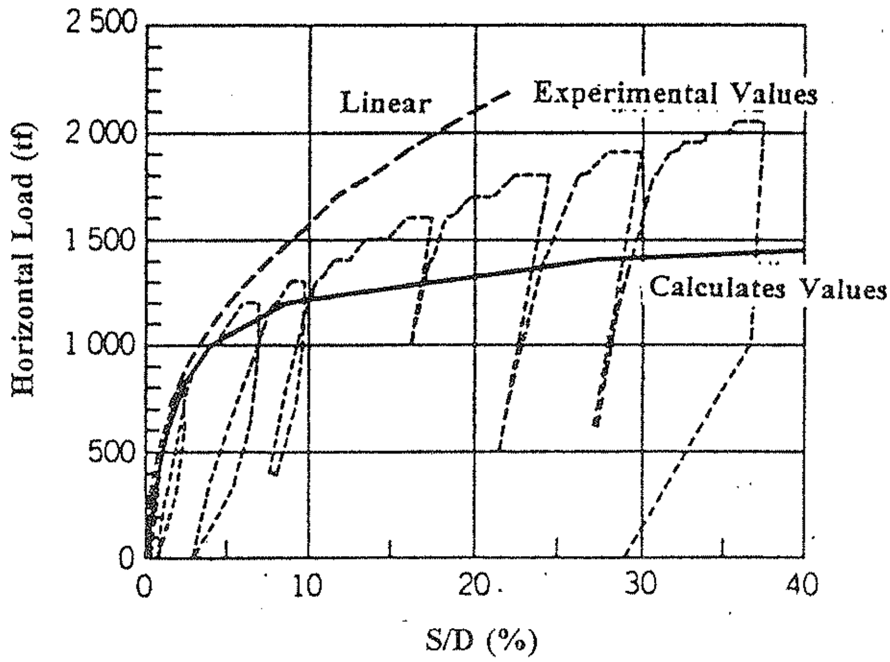


Figure 4. Analysis of the Loading Test of a Cast-in-place Pile Fondation

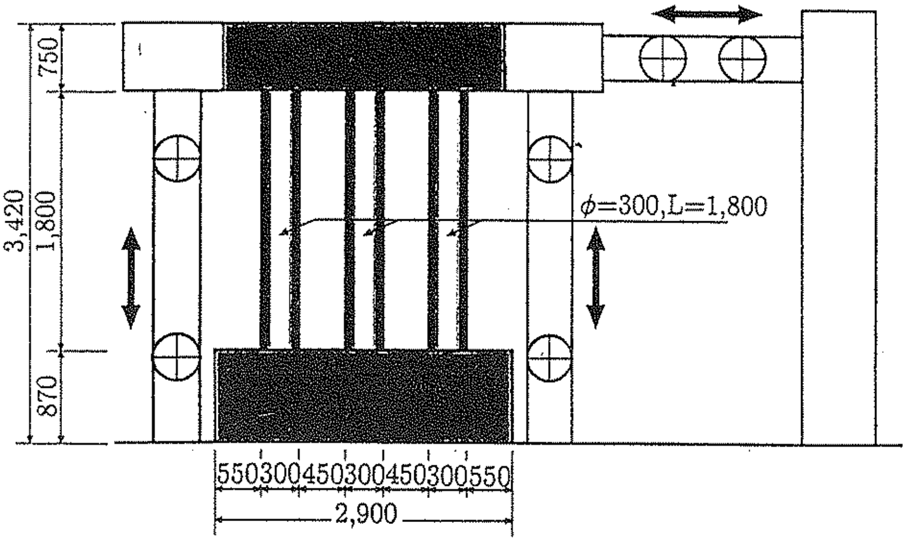


Figure 5. Loading Test

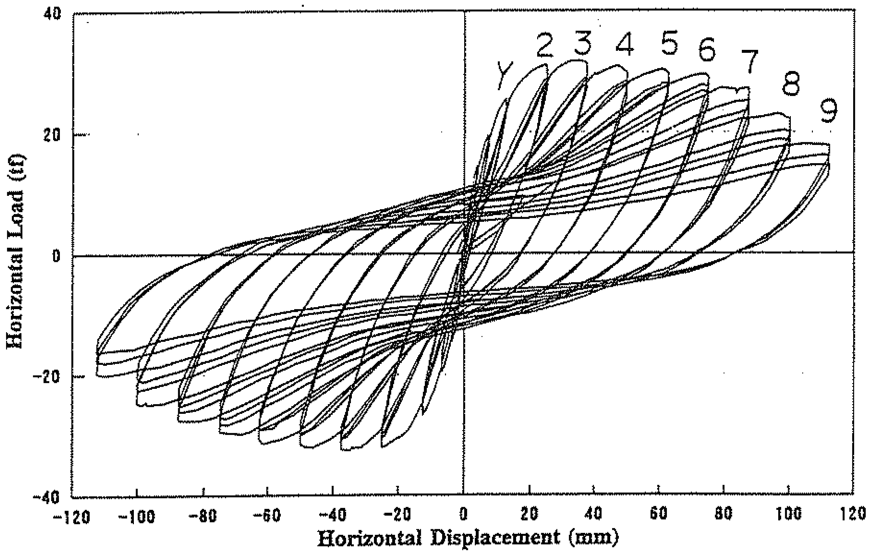
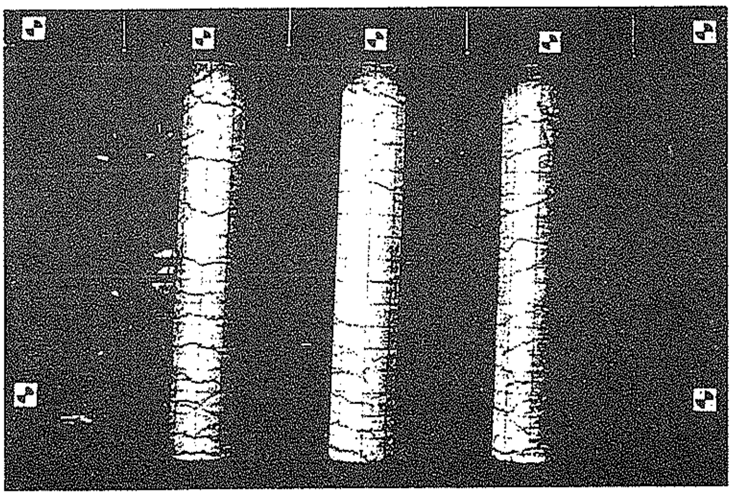


Figure 6. horizontal Load - horizontal Displacement Hysteresis Curve (Cast- in-Place Piles)



Photograph 1. State of Damage to Pile Bodies (Cast-in-place Piles, + 4 δ_y)

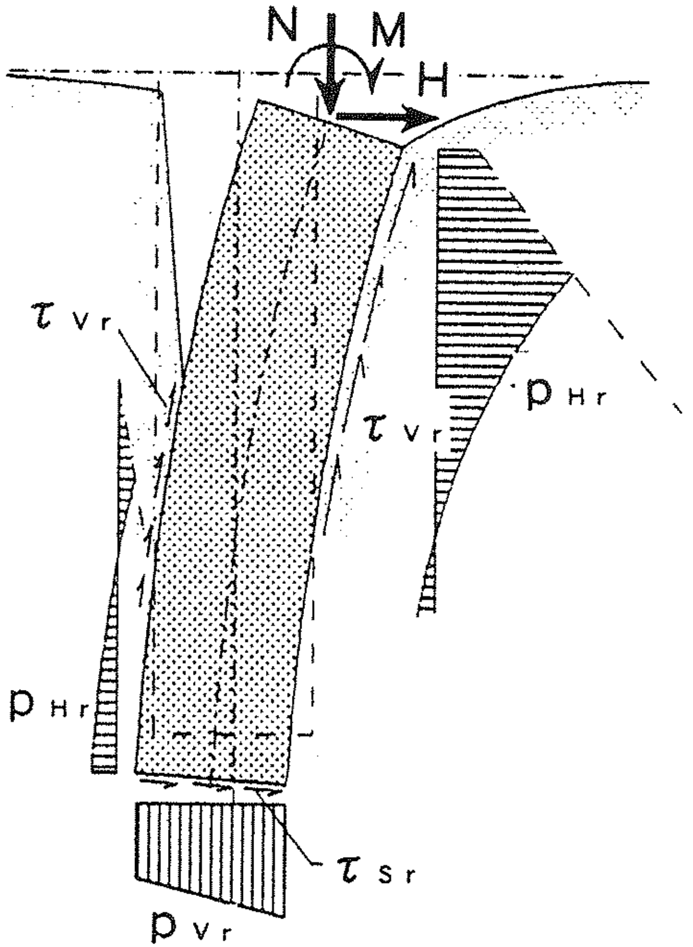


Figure 7. Resistance Elements of Caisson Foundations

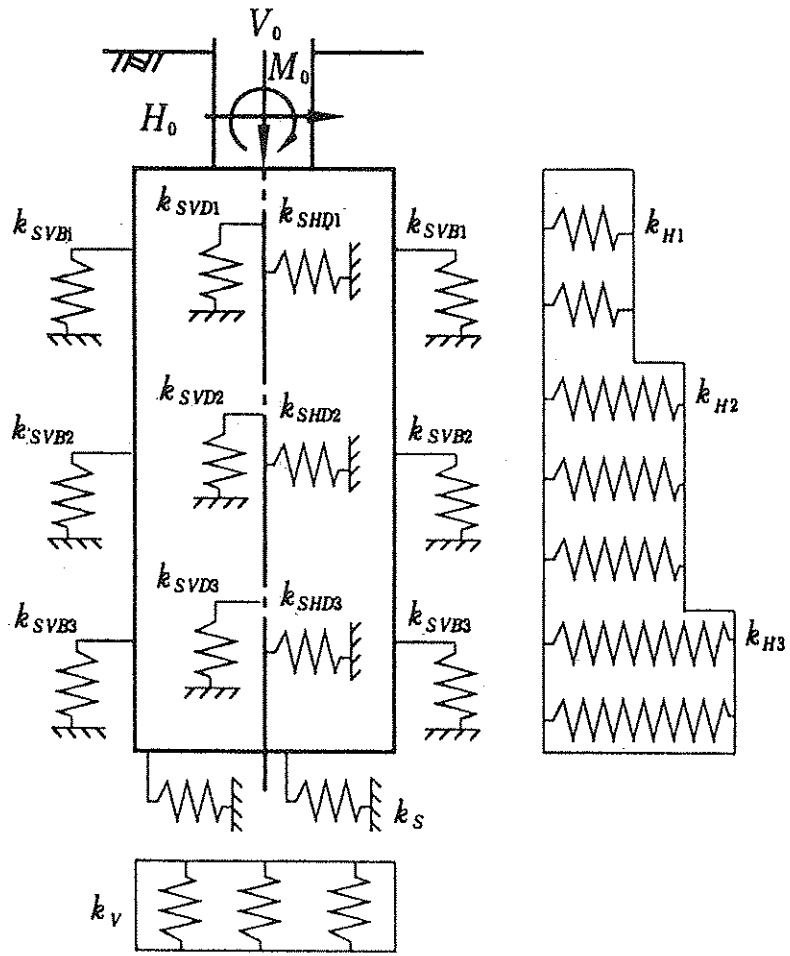


Figure 8. Coefficients of Subgrade Reaction of Caisson Foundations

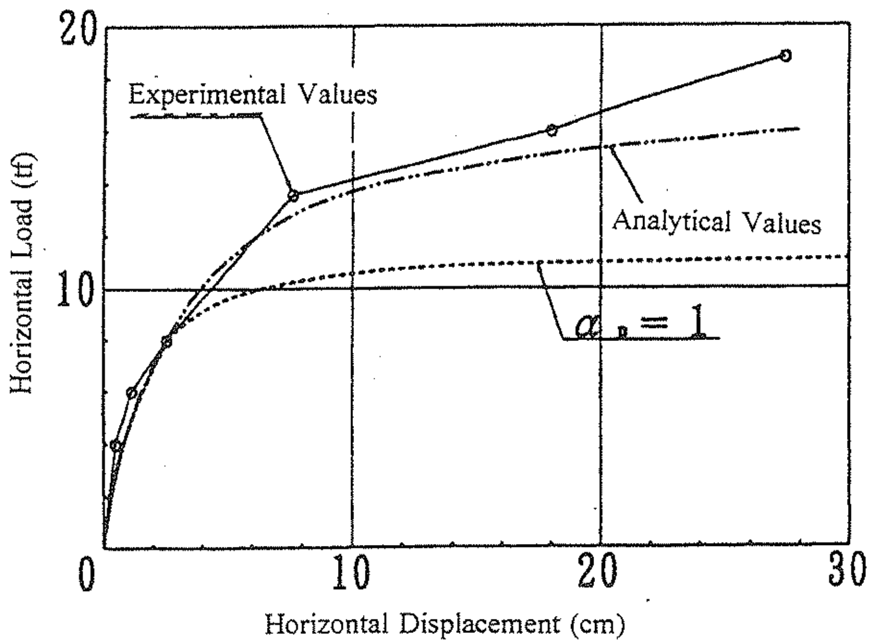


Figure 9. Horizontal Loading Test of a Caisson Foundation

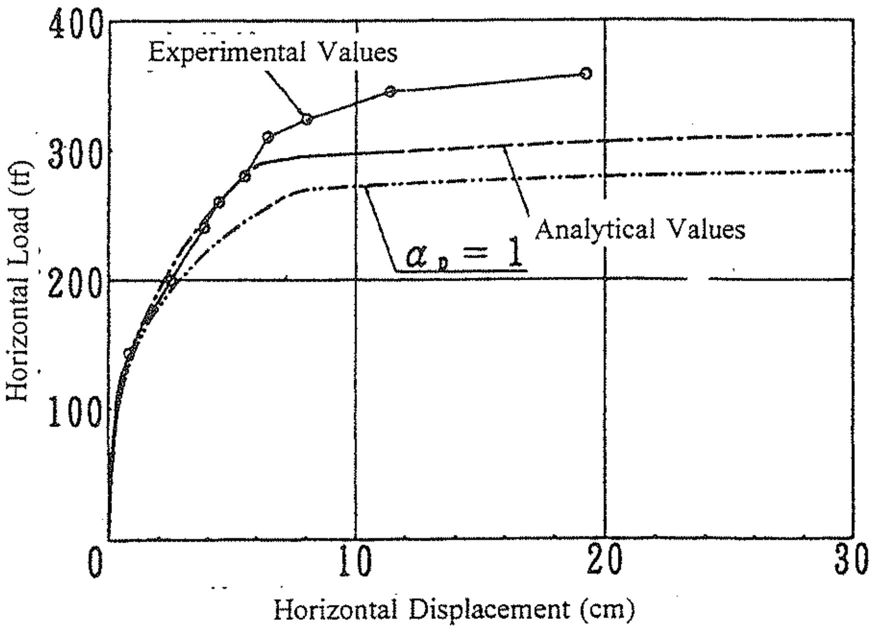


Figure 10. Horizontal Loading Test of a Wall Foundation

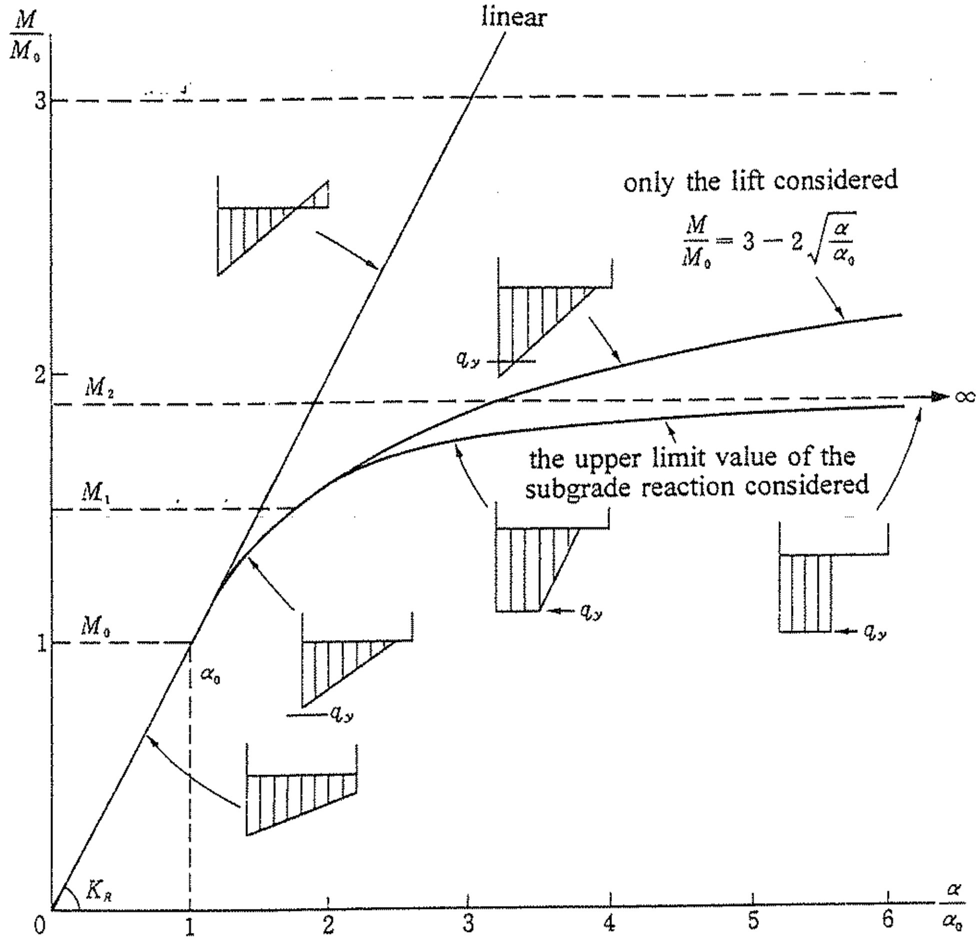


Figure 11. Moment - Angle of Rotation Relationship of Spread Foundation

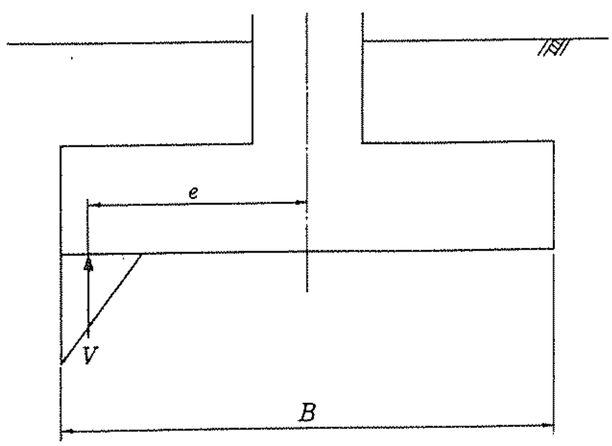


Figure 12. Check of Footing Strength of Spread Foundations