APPLICATION OF ROOT PILES TO FOUDATIONS AND REINFORCING STRUCTURES IN JAPAN

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ABSTRACT

Micropiles with small diameters that were developed in Europe are now widely accepted for use in the reinforcement of structures and earth all over the world. In Japan as well, root piles that are categorized as a type of micropile have accumulated their share of performance records since their initial application in underpinning an observatory in 1980. It can be said that there may be various types of micropiles, depending upon different pile structures and piling methods for construction that are due mainly to varied ground characteristics as well as differing construction systems favored by each nation. The purpose of this paper is to present special features of root piles in their applications to structural foundations and reinforcements.

1. INTRODUCTION

When applied to structural foundations and reinforcements, because root piles themselves are small in diameter and less stiff, they are expected to function together with the reinforced earth that comprises piles and ground as a whole. Therefore, piles are deemed to be able to demonstrate their high earthquake resisting performance as a flexible substructure.

The authors' approach to this subject was made to clarify the bearing-capacity characteristics of structural foundations driven by root piles using mainly impact assessment conducted on root piles stricken by the Great Hanshin-Awaji Earthquake as well as using case studies on root pile application. This paper includes the following report items:

1) Bearing-capacity characteristics of root piles as reinforced structural foundation
2) Applicability as verified by case studies on the use of piles in rockfall prevention cokerages and underpinning bridge abutments
3) Evaluation on the earthquake resistibility of piles from results of field surveys conducted on damaged root piles after the Great Hanshin-Awaji Earthquake

2. ROOT PILE FEATURES

Root piles have found their way into the world, not to mention Europe, where they were originally developed during the 1950's. Root piles are used as a sort of reinforced earth for the ground, forming a composite structure of both foundation and piles. They are mainly designed to be friction piles regardless of their end-bearing capacity since individual piles serving as reinforcing material are uniformly of a small diameter.

2.1 Root pile form and specifications

In Japan, a root pile is composed of reinforcing bars as the core material and grout, such as cement milk or mortar, as shown by the cross section in Table 1. The standard specifications provide that root piles are cast in place with diameters of 60 to 135mm, lengths of 4.0 to 30.0m, and proper strengths of 170 to 420kN.

2.2 Casting of piles

Piles are formed by casting reinforcing bars into casing holes at the site drilled by a boring machine, with mortar grout coating those bars as shown in Figure 1. As compared with machines used for casting ordinary piles with large diameters, the small boring machine is operable for casting root piles. Therefore, root piles have such advantages as providing a higher operability in casting at the site within an extremely limited working space under complex ground conditions that are difficult to excavate or at any urban area where such public nuisances as noise or vibration are restricted.

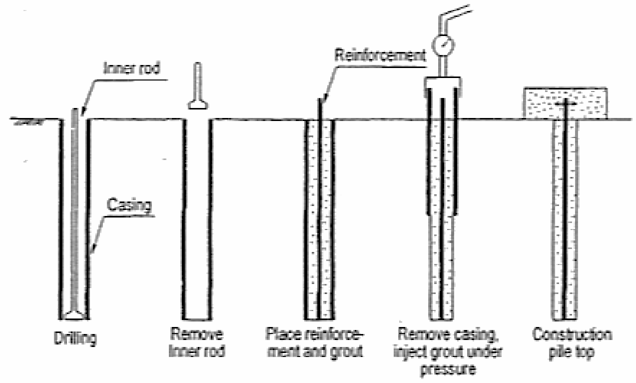

Figure 1 Casting Method of Root Piles

2.3 Micro pile categories and divisions of application

According to piling arrangement patterns, piles are divided into three categories: a single unit pile as shown in Figure 2, piles that are effective in their grouping into one unit, and piles that are effective in their forming of a network. With regard to effects of the latter two, it is verified by Lizzi's model test[1] that the bearing capacity of piles has been increased at a larger rate compared with those of individual piles of a single unit. Here, micropiles may be divided into such categories as specified in Table 1.

The share ratio of root piles by their applications in Japan is shown in Figure 3, according to divisions of micropile applications formulated by Bruce[2]. It is noted that the application of micropiles to structural foundations for reinforcement accounts roughly for 20 percent of the total; in particular, retaining walls take the largest share among all structures being reinforced by root piles.

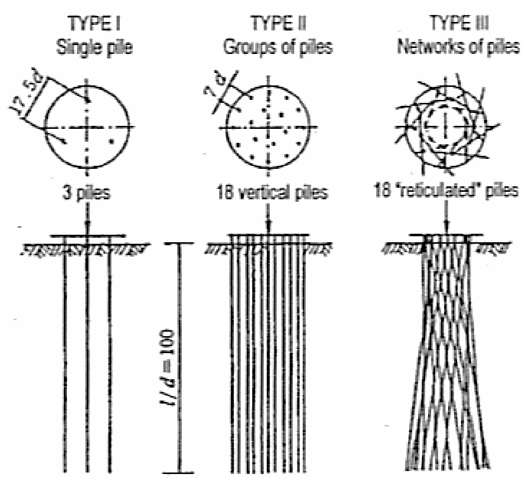

Figure 2 Arrangement of Micro Piles[1]

Table 1 Categorization of Micro Piles

	High capacity	⟺	Low capacity
Reinforcement	Steel pipe · Steel bar	Steel pipe	Steel bar
Bearing capacity	≥1000kN	1000kN >	420kN ≥
	Steel bar / Grout / Steel pipe	Grout / Steel pipe	Steel bar / Grout
Pile diameter	300mm ≥ D ≥ 150mm	250mm ≥ D ≥ 100mm	150mm ≥ D
Arrangement method	I, II		II, III

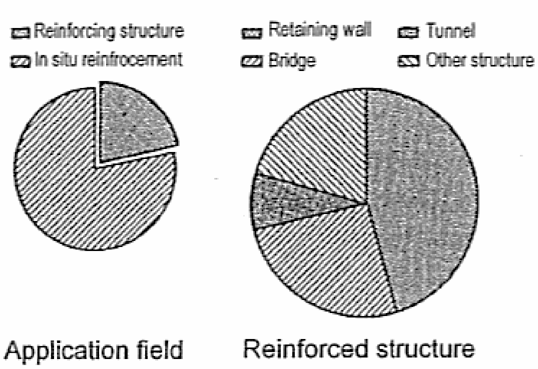

Figure 3 Divisions of Root Pile Applications

3. BEARING CAPACITY CHARACTERISTICS

Bearing capacity characteristics of structures reinforced by root piles are characterized by their load-sharing between both piles and the ground, unlike pile foundations, which are supported by individual piles. Because of such characteristics, root piles are sometimes regarded in Japan a means of reinforcing the ground itself.

Root piles reinforce the ground by reducing the load on it. Piles are driven into the ground and thus made to bear part of the load by acting against external force. Thus, calculations can be made on three types of stress-sharing that is composed of earth and piles (steel bars and mortar) within the elastic deformation region, on the assumption that ground reinforced by root piles is a component of the linear elastic region as composite·ground (reinforced earth):

On the basis of the elastic performance in the body of reinforced earth, Equation (1) below can be used to calculate the equivalent section area covered with mortar for each pile. Equations (2) and (3) also apply to calculations of the equivalent section area and the equivalent geometrical moment of inertia from the reinforced earth structure in this study:

$$A_{pile} = n \cdot As + (Ac - As) = (n-1) \cdot As + Ac \ (m^2) \quad (1)$$

$$A_{ERP} = m \cdot A_{pile} \cdot S + A \ (m^2) \quad (2)$$

where,

A_{pile}: Mortar-equivalent section area of pile (m^2)
n: Elastic modulus ratio of mortar and reinforcing bar (n=15)
Ac: Sectional area of pile (m^2)
S: Number of piles contained in the datum plane under study
A_{ERP}: Soil-equivalent section area of reinforced earth structure
m: Elastic modulus ratio of pile versus its surrounding earth
As: Sectional area of core material (m^2)
A: Net sectional area of reinforced earth structure (m^2)

$$I_{ERP} = I + m \cdot A_{pile} \cdot \Sigma x_i^2 \ (m^4) \quad (3)$$

where,

I_{ERP}: Soil-equivalent geometrical moment of inertia of reinforced earth (m^4)
I: Net geometrical moment of inertia of reinforced earth structure (m^4)
X_i: Distance to each pile as measured from the neutral axis of the datum plane under study (m)

According to the calculation above, the maximum compressive stress of each component member can be calculated as follows against external force acting upon the datum plane under study for the reinforced earth structure, with reference to the equivalent sectional area and equivalent geometrical moment of inertia:

$$\sigma_{ERPXi} = \frac{N}{A_{ERP}} \pm \frac{M}{I_{ERP}} \cdot xi \quad (kN/m^2) \quad (4)$$

where,

$\sigma_{ERP\,x}$: Compressive stress acting upon the pile location on the datum plane under study (kN/m^2)
N: Vertical force acting upon the reinforced earth structure on the datum plane under study (kN)
M: Bending moment acting upon the reinforced earth structure on the datum plane under study (kN·m)
x_i: Distance from the neutral axis on the datum plane under study (m)

$$\sigma_{c\ max} = m \cdot \sigma_{ERP\,x\,max} \times 10^{-1} (N/mm^2) \quad (5)$$

$$\sigma_{s\ max} = n \cdot \sigma_{c\ max} \ (N/mm^2) \quad (6)$$

where,

$\sigma_{ERP\,x\,max}$: Max. compressive stress acting upon the reinforced earth structure (kN/m^2)

$\sigma_{c\,max}$: Max. compressive stress acting upon the mortar-coated portion of piles (N/mm^2)

$\sigma_{s\,max}$: Max. compressive stress acting upon core material (N/mm^2)

Figure 4 shows the relationship between the occupancy rate of root pile placement α and the specific unit stress of component member σ_{ERP}, reflecting disparities in the elastic modulus of the ground, assuming only the vertical load acts upon the composite ground finished by placing root piles of such variable parameters as shown in Table 2.

As the occupancy rate rises with the number of piles in the ground, the load on ground may be reduced, but the degree of reduction tends to fall despite the greater number of piles, if the ground is characterized by a low degree of elastic modulus.

At the design stage, an adequate number of piles must be determined for their placing in situ, within the lower limit of placement density, by the compressive stress value acceptable to the ground of the lowest strength among three types of component members.

Here, the "lower limit of placement density" refers to the density in which both piles and ground can be united by surface friction to take behaviors into deformation.

For the sake of design optimization, piles with a rigidity near that of the ground must be arranged in the proper density, based on the bearing capacity characteristics of the ground underlying the structure and reinforced by root piles.

Table 2 Root Pile Parameters

Pile diameter	ϕ135mm
Steel bar	D38
A	1.0m × 1.0m = 1.0m²
N	100kN
M	0 kN·m
α	Σ Ac/A

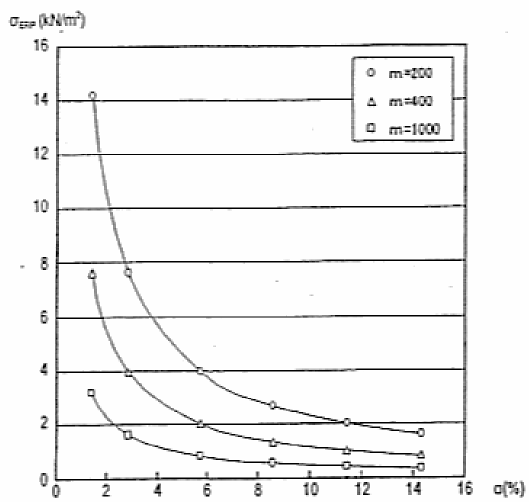

Figure 4 Diagram Showing Occupancy Rate α of Root Pile Placement and Component Member Stress σ_{ERP}

4. APPLICABILITY AND CASE STUDIES

This is to introduce two examples of root-pile applications, taking advantage of their own characteristics, each respective case is of the application of root piles to either structural foundations or reinforced earth.

4.1 Root-pile applications

4.1.1 Underpinning an existing bridge abutment (Osaka city)

The existing abutment was set up over the back face of the former abutment. The superstructure of that bridge was then supported by the existing abutment in place of the old one.

Later, with the development of a railway conversion project onto the elevated track, an implementation plan envisaged the new construction of a portal-framed abutment for the viaduct, by breaking up the existing abutment. Furthermore, it was planned to have the abutment taken out while maintaining train operations and keeping bridge girders in place.

The plan then included the temporary shifting of superstructure loads from the existing abutment back to the old abandoned one.

The casting of root piles in place was proposed for the purpose of the underpinning to be well matched with the train loads that were imposed upon the former abutment. This construction method was therefore adopted because it met such requirements as allowing work to be done 5.1m under the bridge girders, which would not be removed, and train operations to continue unaffected in the least by any vibrations arising during the construction of the new abutment.

The substructure ground was composed of sand layers with relative looseness down to a depth of 4.0 m as shown in the boring log. A sand layer of N = 20 or larger in value was set up as the pile-bearing layer.

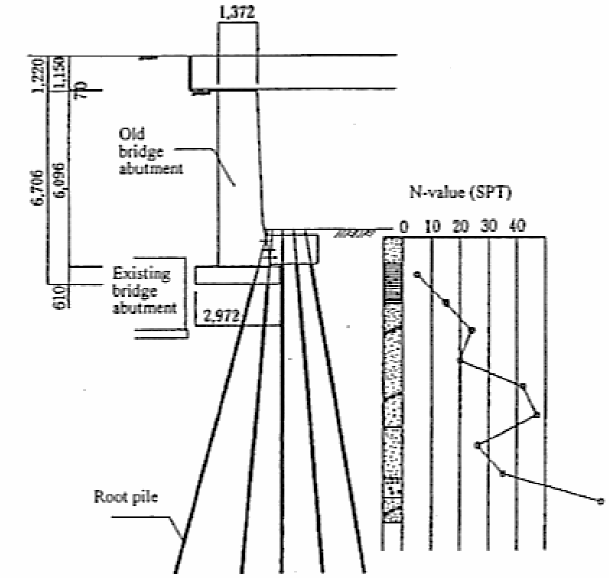

Figure 5 Cross Section Drawing

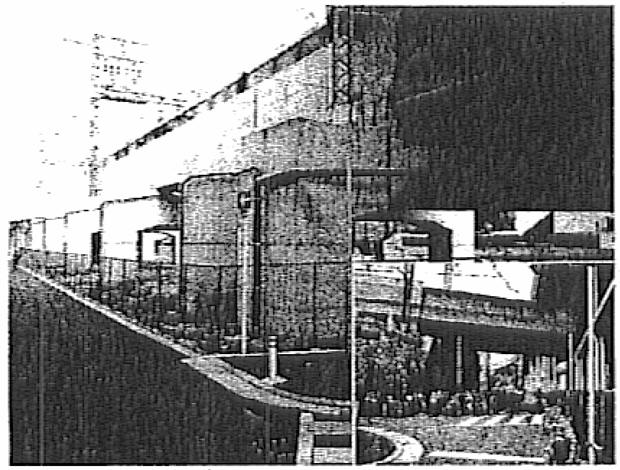

Photo 1 Whole View after Construction
Whole View before Construction

4.1.2 New foundation of rockfall prevention cokerages (Yamanashi Prefecture)

The lower subgrade of the Fuji-Subaru Line, a toll road, at the halfway point of Mt. Fuji, consists of a scoria stratum of high permeability, which is spread over the whole piedmont of the mountain.

Water seepage throughout the scoria stratum in the summer tends to form frozen earth in winter. This caused debris to flow at three points on the Fuji-Subaru Line, apparently originating from the slip surface in the boundary between frozen and melted layers. There was still a great concern over the probable recurrence of debris flow as the total sediment volume of unstable nature was estimated to have reached as much as 100,000m^3 per site[3]. In the Oniwa District, one of the six stricken districts, the construction of a tunnel that was designed to let the unstable sediments flow down to facilitate restoration from damages was planned to include the use of root piles in the tunneling ground foundation as shown in Figure 6.

In this approach, for the purpose of evaluating the frictional performance of piles in the scoria stratum, vertical loading tests were conducted using both the compression and tension of a 3.0m-long test pile to verify the degree of surface friction stress. Because the maximum loading weight was given as 300kN/pile as shown in Figure 7, the distributed axial force at loading in both compression and tension failed to reach the ultimate limit, but it was confirmed that the value rose up to 0.23N/mm^2, far beyond the estimated 0.10N/mm^2 for ultimate surface friction stress.

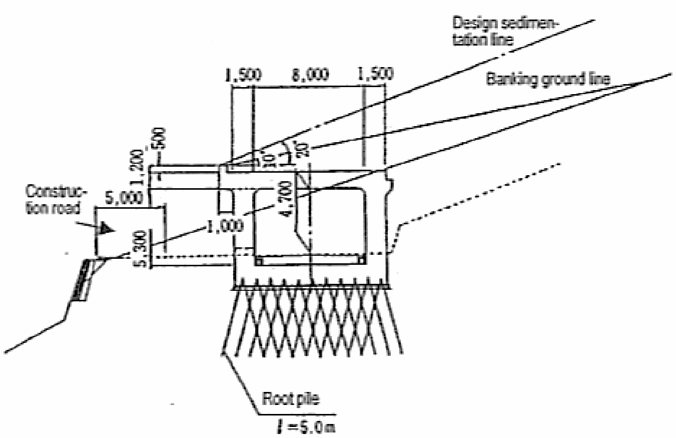

Figure 6 Cross Section Drawing

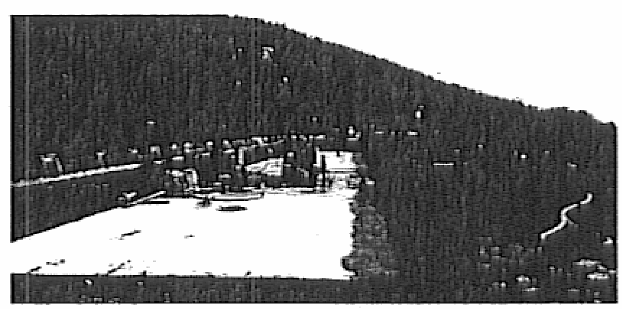

Photo 2 Constructed as Drawn

4.2 Consideration of applications

As noted in the preceding two examples, applications of root piles can be advantageous because of their small diameter and operable driving machines as well as their ability to be spliced together to form a short length even in a narrow or limited working space. Furthermore, they produce limited vibration and noise when then are cast in place. These are the main reasons root piles are preferred.

On the other hand, because their load-bearing capacity is less than those of larger-diameter conventional piles, root piles may be less economical if the implementation plan does not have any restricted operating conditions.

Moreover, field measurement data on root piles remain less available than those on conventional piles. Therefore, the further development of the designing method for a broader application of root piles in the construction field is needed.

Hence, in effort to promote reinforced structural foundations using root piles, it is important to address some of the following future tasks:

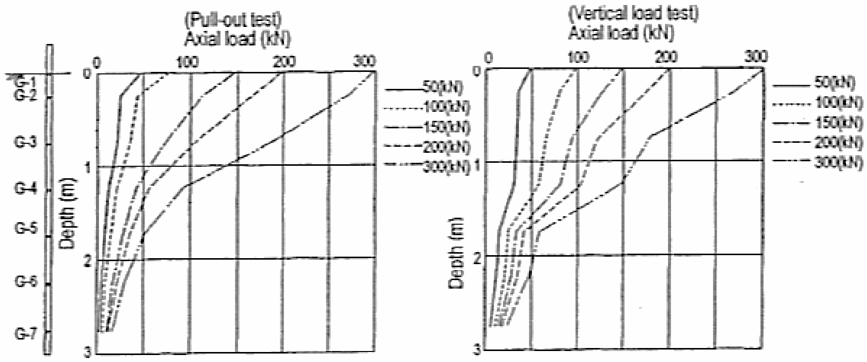

Figure 7 Loading Test Results

1) The improved strength and surface friction force for small-diameter piles leading to a successful performance of higher bearing capacity
2) The development of an innovative drilling machine that is more compact but has a higher drilling efficiency
3) The collection and assembly of data from tests in situ

5. EARTHQUAKE-RESISTING PERFORMANCE

After the 1995 Great Hanshin-Awaji Earthquake, damage investigation of reinforced earth structures with steel reinforcements was carried out in the southern part of Hyogo Prefecture, Japan. The investigated area is located within a circle with a radius of 70km at the epicenter. The damage investigation concerns on the 10 reinforced natural ground structures, which are located within area having maximum acceleration of more than 100gal. In this paper, the damage performance of reinforced earth structures is first described, together with a comparison of the damage performance of unreinforced and reinforced natural ground structures. Also, seismic integrity of reinforced earth structures is discussed.

5.1 Outline of investigation

The reinforced earth structures investigated are shown in Figure 8, which consist of reinforced natural ground structures with root piles. Table 3 shows the descriptions of characteristics of steel reinforcements.

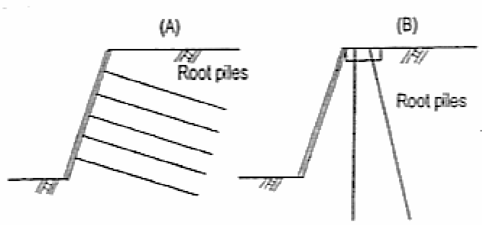

Figure 8 Reinforced Earth Structures Investigated

Table 3 Characteristics of Steel Reinforcements

Rein-force-ment	Material characteristics · Dimension
Mortar	ϕ100mm - 135mm, f ck = 24N/mm^2
Steel bar	Deformed reinforcing bar: SD295A with norm JIS D25 - D32

The investigation items are damage condition of wall surface, surrounding areas and the boundary, which are visually investigated, and their damage is ranked. According to the investigation, no complete collapse or destruction in reinforced natural ground structures with root piles. However, some examples of minor damage and deformation were observed, which might characterize the general performance of reinforced earth structures during earthquake.

5.2 Damage performance of Root Piles-Reinforced Structures

Figure 9 shows the relationship between wall height of reinforced earth structures and estimated maximum horizontal acceleration[4] at each investigated site, together with their damage degree. The height of reinforced earth structures ranges from 2.5m to 16.0m. The damage degree tends to be moderately higher, as increasing the maximum horizontal acceleration. However, in the region of similar maximum acceleration, higher structures do not necessarily suffer more extensive damage. Structures required some repair (two cases marked by ■) are located within area of maximum horizontal acceleration of greater than 500gal and their wall heights are lower than 8m. In other words, there is no trend that the damage of structures becomes more severe, as increasing wall height and maximum acceleration. It is difficult to generalize the seismic performance of reinforced earth structures, because the ground motion at each site is not available and the topography and geological and geotechnical properties differ at each site. Therefore, seismic performance of reinforced earth structures are estimated based on their damage condition of wall surface and surrounding ground. The reinforced earth structures investigated have shotcrete wall, thin RC plate , which makes comparatively flexible wall structures. Suggesting that the reinforcements absorbed a significant earth pressure acting on walls during earthquake with no meaningful damage, such as failure and overturning of walls. However, the wall surfaces deform following deformation of whole reinforced earth and, in consequence, this could result in damage to joints.

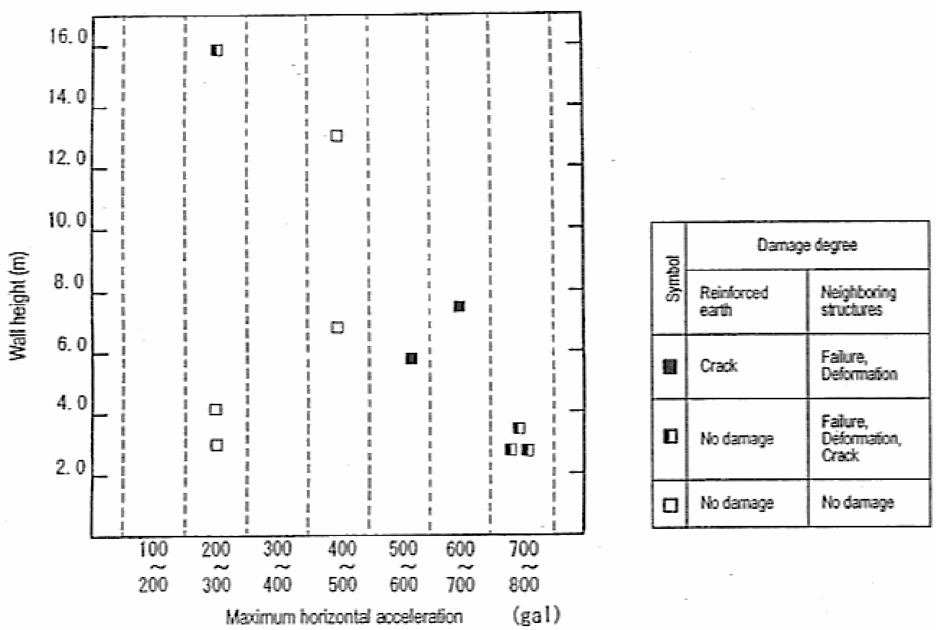

Figure 9 Relationship between Wall Height and Estimated Maximum Horizontal Acceleration as Related to Damage

5.3 Results of damage investigation of reinforced natural ground structures

As shown in Figure 9, all sites where neighboring structures around reinforced natural ground structures were damaged were located within area of maximum horizontal acceleration greater than 300gal. Four sites among them are greater than 700gal. The reinforced natural ground structures investigated, however, have slightly damaged. In two case histories, only slight damage of cracks was observed in the reinforced natural ground structures of Type B (see Figure 8), which were located on inclined ground experienced by maximum horizontal acceleration as high as 500gal (see Figure 9). In these two sites, unreinforced natural ground structures, which were significantly damaged, are available adjacent to reinforced natural ground structures. This will make the comparison of damage between unreinforced and reinforced natural ground structures. These examples are described in detail below.

5.3.1 Case history 1 (Kobe city)

As shown in Figure 10, two types of facing were used in the construction of the reinforced natural ground structures (6m to 7m in height). One consists of a retaining wall of piled stones and the other a shotcrete wall. Adjacent to the shotcrete wall, an unreinforced retaining wall (3m or less in height) was constructed by piled stones.

Cracks were observed in the ground surface at the top of shotcrete wall and hair cracks were also detected on the face of the shotcrete wall. An opening of 5cm was observed at the facing boundary of reinforced earth structures (Photo 3). The adjacent unreinforced retaining wall was destroyed after falling forward, in spite of lower wall height. Houses located in front of reinforced structures suffered some significant damage.

Significant damage was observed in the facing of shotcrete, which has less rigidity than in piled stone retaining walls. Comparing the performance of the unreinforced and reinforced structures above-mentioned, it can be confirmed that the advantage of seismic integrity of reinforced earth structures.

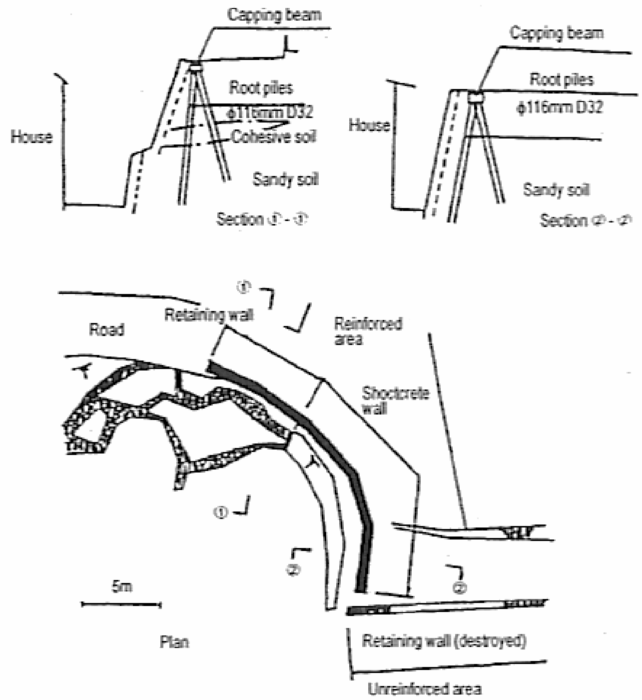

Figure 10 Location of Case History 1

5.3.2 Case history 2 (Takarazuka city)

As shown in Figure 11, an investigation of the post-earthquake condition of the site involved two locations, namely the reinforced retaining wall (H=7.5m) of Type B in Figure 8 and an unreinforced retaining wall which is located orthogonal to the reinforced wall.

The unreinforced retaining wall deformed forward with an opening of 20cm at most between wall top and backfills, together with cracks on ground surface along the wal. The ground caved-in parallel cracks by 7cm difference in areas of 20m off the top of retaining wall. This suggests that the natural ground might slightly slide in the unreinforced area.

In the reinforced area, several cracks on ground surface were detected and the ground mass deformed forward in the front of the reinforced retaining wall, although no cracks or deformations were found at the backfills. In this site, the direction of ground motion is approximately orthogonal to the reinforced retaining structure, suggesting that more severe seismic force acted on the reinforced retaining wall. Consequently, it is suggested that the reinforced earth structure has a strong seismic resistance.

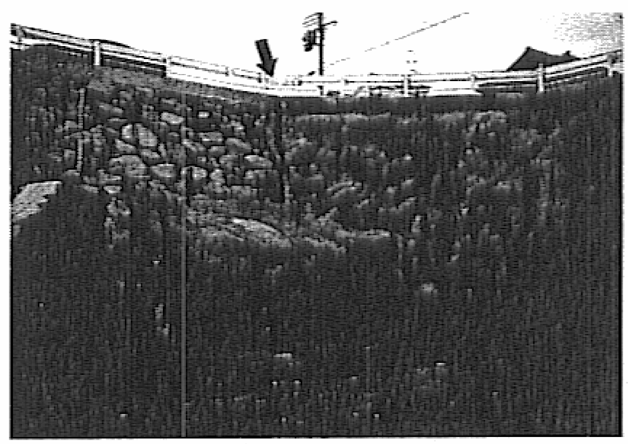

Photo 3 Damage Example at Facing Boundary of Reforced Earth Structure

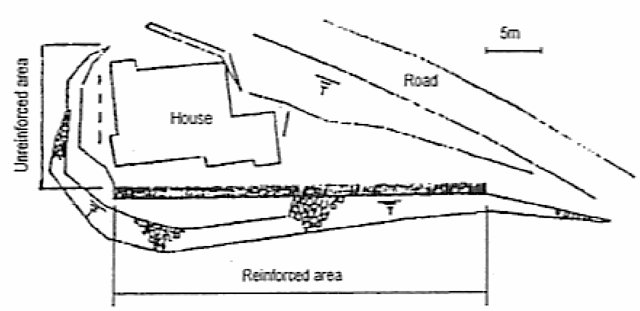

Figure 11 Location of Case History 2

5.4 Seismic performance of Root Piles-Reinforced Structures

All reinforced earth structures investigated were checked their aseismicity by the seismic coefficient method. Structures judged to need repair (marked with ■ in Figure 9) had not been designed to withstand the real seismic motion in the earthquake, even though they did not reach whole failure. Also, the shape of reinforcements was determined through the specification at normal or detail of structure. Therefore, the seismic integrity of the structures judged to need repair is examined below, using the seismic coefficient method.

5.4.1 Method for examining performance during earthquake

Two cases of reinforced earth structures were selected for examination herein; that is, Sections ① and ② in the reinforced natural ground structure in Case history 1 as shown in Figure 10. The seismic integrity of reinforced natural ground structures was evaluated using Equations (7) and (8) respectively, using horizontal seismic coefficient k_h.

$$F_s' = \frac{\sum(W_i \cos\theta_i - k_h W_i \sin\theta_i)\tan\varnothing + cl_i)}{\sum(W_i \sin\theta_i + k_h W_i \cos\theta_i)} \tag{7}$$

$$P' = (F_{sp}' - F_s')\sum(W_i \sin\theta_i + k_h W_i \cos\theta) \tag{8}$$

where

F_s': Minimum safety factor of unreinforced natural ground during earthquake
P': Sliding force acting on reinforcement (reinforced earth) during earthquake (kN/m)
F_{sp}': Design safety factor of reinforced natural ground structure during earthquake
c: Cohesion of soil (kN/m^2)
Ø: Internal friction angle of soil (°)
l: Bottom length of a slice (m)
W: Total weight of soil above sliding plane (kN/m)
θ : Angle between sliding plane and horizontal plane (°)

For reinforced natural ground structure, the failure mode is assumed by the pull-out or rupture of reinforcement. The tensile force of reinforcement is calculated as the smaller value of the two resisting forces based on both failure modes.

5.4.2 Evaluation of seismic integrity

In all reinforced earth structures, the safety factor due to failure mode of pull-out was smaller than that of rupture, and the tensile force of reinforcement was calculated by the pull-out resistance of reinforcement. Figure 12 show the relation between the horizontal seismic coefficient k_h for design and the safety factor Fg to minimum pull-out resistance of reinforcement for and reinforced natural ground structure respectively. It is seen in this figure that the safety factor Fg decreases as increasing horizontal seismic coefficient, and becomes less than 1.0 for the range of k_h = 0.24 to 0.3. A comparison between Section ① and ② of Case history 1 of reinforced natural ground structure revealed that a lower safety factory was obtained in more severely damaged structure. Considering the seismic performance of these reinforced earth structures, the horizontal seismic coefficient of about 0.3 in the seismic coefficient method corresponds to the estimated maximum horizontal acceleration of about 500 to 600gal.

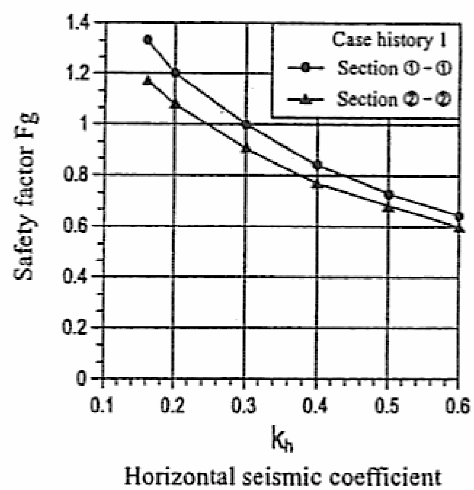

Figure 12 Relationship between Safety Factor and Horizontal Seismic Coefficient in the Reinforced Natural Ground Structure

6. CONCLUSION

The application of root piles to structural foundation and reinforcement has been tried and proven in the field many times in Japan. Many researchers have verified that root piles are fully able to perform their structural function without suffering damage in the Great Hanshin-Awaji Earthquake, which had the largest magnitude of any land-based earthquake in Japan.

Results of damage inspections revealed that all structures reinforced by root piles were deformed in response to the earthquake motion as observed from their residual configuration of deformation. Those reinforced structures were shown to be sufficiently flexible and highly earthquake resistant.

In Japan, where the risk of damage to structures from seismic activity is high and compounded by highly dense urban populations and facilities, root piles — with its bearing capacity characteristics, earthquake-proof performance, and efficiency in execution — are a promising means of reinforcing existing structures and as well as useful as foundation piles for new structures built under adverse construction conditions.

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