







## 2. Destruction Sequence and Damage Events

When considering the method of the seismic design in particular, it is necessary to confirm the destruction sequence assumed by the design work in order to set the engineering index of the resisting member and to identify the critical member governing the destruction mode. Hence in this section, the destruction sequence of the target structure is assumed with reference to the destruction sequence resulting in the related research [5], also the destruction mode confirmed by past research results[6].

### 2.1 Summary of experiments

In the related research[5], the static experiments were conducted on the miniature model of 3-column pier-pile integral structures supported by sandy and clayey ground to confirm the destruction sequence and incurred damage events.

In this static experiment, the  $P$ - $\delta$  curve at the loading point (column head) obtained from the experimental results and the history of damage events of each member reaching to destruction is confirmed. The  $P$ - $\delta$  curve is shown in Fig.3. Each event that incurred on the members is also shown in Fig.3.

In both sandy and clayey cases, the top of the column reached yield before the underground parts. Finally, the top of the column reached fracture as an ultimate state.

Furthermore, a gradient of the initial  $P$ - $\delta$  curve is indicated by the straight trend up to the initial yield. Afterward, the gradient of the  $P$ - $\delta$  curve became a gentle curve up to the yield of the underground part. After this event, the gradient gets closer to level and only displacement progresses to the ultimate state.

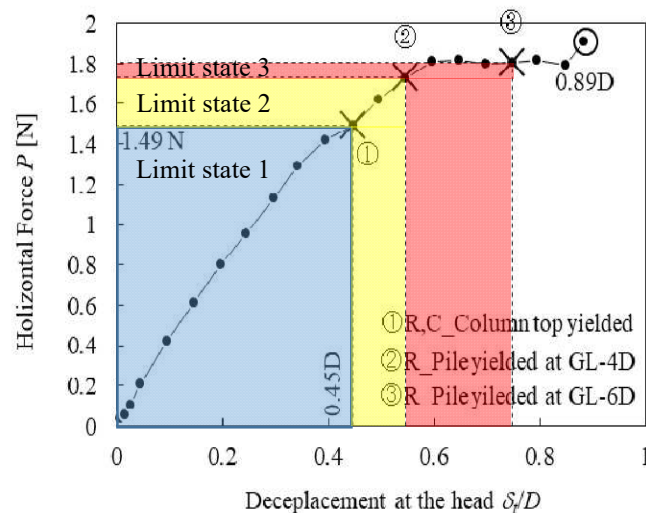


Fig. 3 – Destruction sequence on the  $P$ - $\delta$  curve in clayey ground

### 2.2 Assumption of the design work

Regarding to the type of the destruction sequence for multipolar rigid frame structure consisting of a pier pile integral structure, it was indicated that it can be separated into two types based on the assumption that the pile cannot be pulled out in the past research [6, 7]. These types are the “pier-pile preceding type” in which pier and pile yield in order, and the “ground preceding type” wherein the ground destructs first. The destruction form is shown in Fig.4.

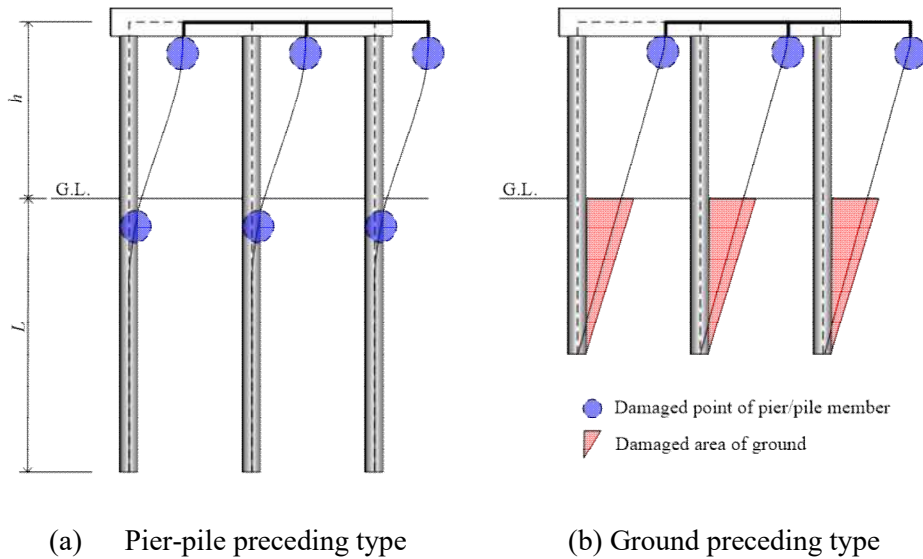


Fig. 4 – Destruction form

The pier-pile preceding type can be said to be superior in toughness because it can avoid the sudden increases in displacement of the structure since the top of the column or the pile leading to yield in order. In other words, it can be said that it is a preferable type of destruction form compared with the ground preceding type in which the displacement suddenly increases once the ground collapses.

Here, the deformation mode of the target structure depends on  $\beta h$ ;  $\beta$  indicating the relationship between rigidity of the ground and stiffness of the pile, and defined as Eq. (1). Herein  $h$  is pier height shown in Fig 4.  $k_H$  = horizontal subgrade reaction coefficient calculated by Eq. (2),  $k_{H0}$  = horizontal subgrade reaction coefficient corresponding to the value of the plate loading test, calculated by Eq. (3),  $B_H$  = the conversion loading width of the foundation calculated by Eq. (4), and  $EI$  = the flexural rigidity of the steel pipe,  $D$  = the diameter of the targeted pier-pile.

Additionally, the determination of the destruction form in cases where each part of the member reaches the plastic region can be assumed to be impacted by the pile length  $L$  since the destruction form is governed by the ratio of the plastic region to elastic region of the ground. Therefore, the relationship between the destruction form and the  $\beta(h+L)$  is investigated based on the idea that the destruction form can be assumed depending on the relationship between the stiffness relationship of ground and pile, and the entire length of a target structure. Fig.5 shows the relationship between the destruction form and  $\beta(h+L)$ .  $\beta(h+L)$  on the vertical axis and ground rigidity  $k_H$  defined as Eq. (2) on the horizontal axis.

From Fig.5, it can be confirmed that as  $\beta(h+L)$  increases in all cases, the destruction form tends to be pier-pile preceding type.

Herein, the boundary of the two types of the destruction form is investigated. The line of the boundary can be expressed to connect the mid points of the two destruction form and shown in Fig.5 as the line approximated by the least squares method using an exponential function. Also, this line can be shown in Eq. (5). Namely, when the Eq. (5) is satisfied, it can be confirmed that the destruction form of this structure can be the pier pile preceding type. Furthermore, the multipolar rigid frame structure can make an economical plan by reducing the length of the pier pile integral structure when the index  $\beta(h+L)$  is kept as close as possible to and above the Eq. (5).

$$\beta = \sqrt[4]{\frac{k_H D}{4EI}} \quad (1)$$



$$k_H = k_{H0} \left( \frac{B_H}{0.3} \right)^{-3/4} \quad (2)$$

$$k_{H0} = \frac{1}{0.3} \alpha E_0 \quad (3)$$

$$B_H = \sqrt{\frac{D}{\beta}} \quad (4)$$

$$\beta (h + L) \geq 0.4 k_H^{1/4} \quad (5)$$

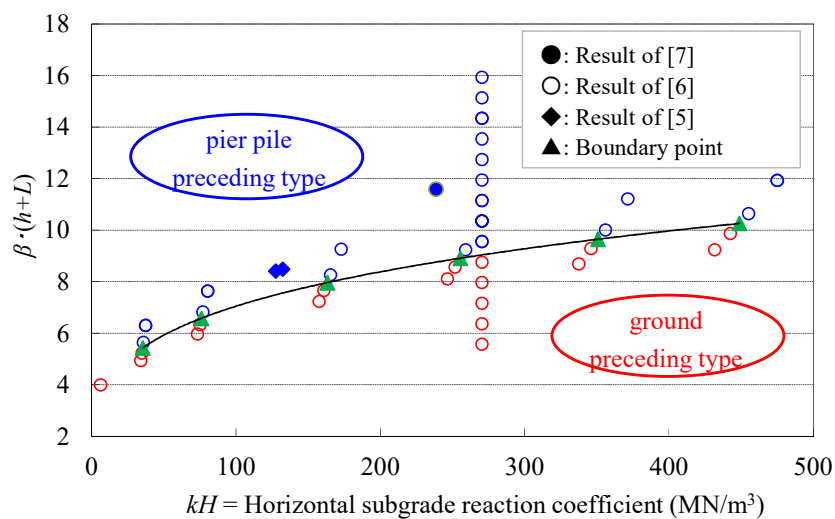


Fig. 5 – The relationship between the destruction form and  $\beta(h+L)$

### 3. Target Performance for the Design Work

When discussing on the type of the destruction form regarding the pier pile integral structure, it can be confirmed from Fig.3 that the destruction sequence goes through the following three states.

#### 3.1 Limit state of the target performance

Each state can be defined as follows;

Limit state 1 can be defined as a state in the elastic region. This state can be defined as the region where the mechanical properties of the whole bridge system do not exceed the elastic region. In other words, the limit state which can be regarded as keeping reversibility.

Limit state 2 can be defined as a state where either the top of the column or the underground part of the pile reach yield. This state can be defined as the region where the plastic region occurs only on the member targeted of the plasticity and its plastic region can easily be repaired. In this state, the displacement does not suddenly increase since the target structure is a statically indeterminate structure in high-order. Because of this, the structure does not become unstable in the case of the state in partial yielded of the structure. Furthermore, since the yielded member of the pile is located in the vicinity of the ground surface, around 0 to 2 m, it can keep the visibility after being damaged by a major earthquake.

Limit state 3 can be defined a state where both the top of column and the underground part of the pile reach plastic region. The deformability of this state can be kept in the plastic region by controlling a certain limit point in the plastic region of a targeted member.



### 3.2 Limit state and suggested limit value

The target value of the limit value for each limit state is proposed based on the related past research. Table 1 and Fig.6 are shown the target values of the limit value and the images of the limit state. Herein,  $\varepsilon_y$  indicates a yield strain of the steel member,  $\varepsilon_a$  = the limit value of the steel member in limit state 2 and 3,  $d_d$  = the limit values of the horizontal direction at a ground surface.

Table 1 – Limit states and suggested limit value

	Super structure	Pier/column (Aerial part)	Pile (Underground part)	Ground surface
Limit state 1	$\varepsilon_y$ Elastic region	$\varepsilon_y$ Elastic region	$\varepsilon_y$ Elastic region	Horizontal direction $d_d$
Limit state 2	$\varepsilon_y$ Elastic region	$\varepsilon_a$ Defined in [5]	$\varepsilon_y$ Elastic region	No limit
Limit state 3	$\varepsilon_y$ Elastic region	$\varepsilon_a$ Defined in [5]	$\varepsilon_a$ Defined in [5]	No limit

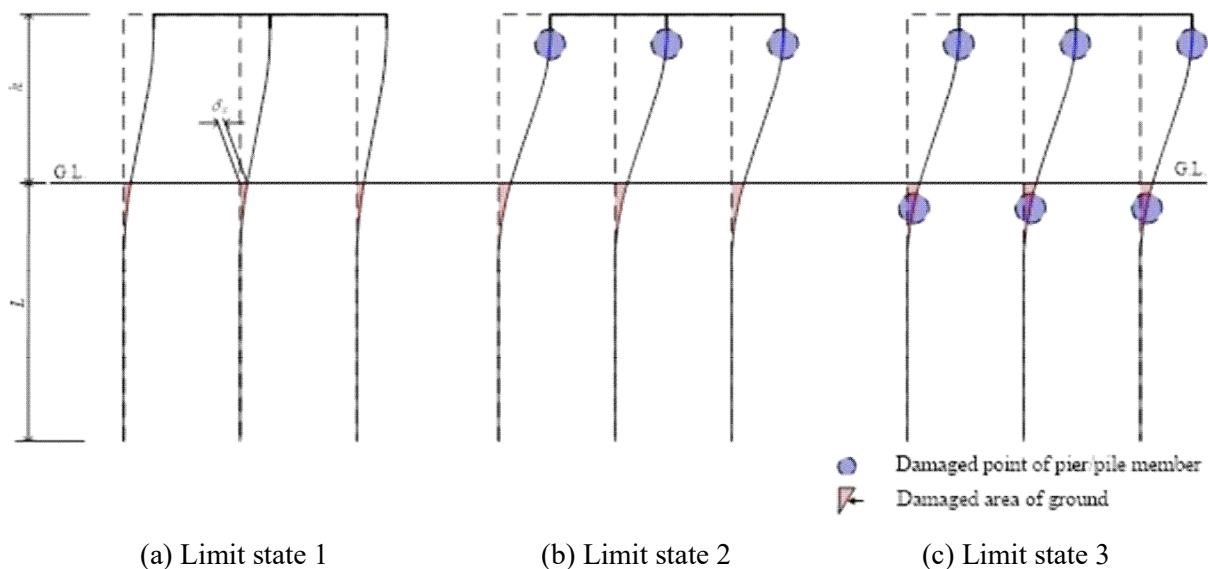


Fig.6 – Damaged image of each limit state

#### 3.2.1 Suggested limit value for limit state 1.

Since the limit state 1 can be regarded as keeping reversibility, the limit value can be defined as the yield strain of the steel members, and the limit point of the elastic region of the ground.

As for the elastic limit point of the mechanical properties of the ground, the newly issued Japan Standard IV [10] can be adoptable. A new limit value calculated by Eq. (6) composed of the partial factor with consideration to the influence of the estimation method of the ground property is proposed.



$$d_d = \xi_l \Phi_Y d_y \quad (6)$$

Herein,  $\xi_l$  is the factor of the investigation and analysis,  $d_d$  is within the range  $14 \leq d_d \leq 50$  mm, and  $d_y$  is the characteristic value of the yield displacement for the pile governed by the ground. These factors are shown in Japan Standard IV 10.5.6 [10]

Furthermore, in the past research [5], the dynamic experiments were conducted on the miniature model of 1-column and 3-column pier-pile integral structures supported by sandy and clayey ground. From these dynamic experiments, the residual displacement could not be confirmed in the case where the maximum displacement at the ground surface is within 5% of the pile diameter.

Based on the above, it is proposed in this research to set Eq. (6) as the limit value of the ground surface for the design work.

### 3.2.2 Suggested limit value for limit state 2.

The limit state 2 assumes the limit state for the major earthquake defined by Japan Standard V [11]. In this study, the limit state of each member can be defined as follows, based on the idea that the condition is set to allow yielding of only the top of the column.

The limit value of the superstructure can be defined as yield strain of the steel members since the limit state can be regarded as maintaining of reversibility. In the case of the multipolar rigid frame structure consisting of the pier-pile integral structure, there is concern that the yield of the superstructure may occur in the vicinity of the rigid portion with the column. However, as this kind of situation is not considered as the destruction form assumed in this research, this kind of situation must be avoided up to the ultimate state.

The limit value at the top of the column of this structure, which assumes a column composed of steel pipes, is set to a limit state that does not exceed the maximum strength point as a member and that the sufficient deformability in plastic region remains. In this context, the past research [5] can be cited as adequate value based on the result of the experiments targeted to elongated steel pile. Furthermore, this experiment was conducted on the steel pile fabricated SKK material (Electric Resistance Welded). Additionally, the target value of the limit value for the pile in underground can be defined as the yield strain since the state of the mechanical properties is defined to not exceed the elastic region.

The limit value of the ground can be cited from the results of past research [5], the result of the case of a seismic wave equivalent to level 2 defined in Japan Standard V [11] can be confirmed to act on the 3-column model. According to the experimental results, the generated displacement of the column at the ground surface is only about 5% of the pile diameter as the maximum displacement, and no residual displacement has been confirmed. In the same series of this study, dynamic experiments with the same seismic wave have also been performed on the single pile model, the generated displacement of the column at the ground surface reaches 10% of the pile diameter as the maximum displacement in both the sandy and clayey cases. Among them, residual displacement was confirmed only for the experimental results of the sandy ground.

Moreover, in the case of the target structure such as a pier-pile structure with a protruding length (= column length  $h$ ), unlike an ordinary pile without a protruding length, the stiffness of the entire structure is governed by the stiffness of the column. Therefore,  $M_{max}$  point (= the point occurring bending moment on the column or pile) generating on the pile is not located on the underground part but rather the vicinity of the ground surface. Namely, this tendency implies the area of ground disturbance is limited to the vicinity area of the ground surface.

From the above, it is proposed that the limit value for limit state 2 is not provided because the column displacement occurring at the ground surface can be controlled by controlling the yield degree occurring at the top of the column, based on the characteristics of this structure.

### 3.2.3 Suggested limit value for limit state 3



Limit state 3 assumes a state not exceeding the maximum strength point as a resistance member, which can be defined by controlling the yield degree of the top and underground part of the column/pile within the limit value.

The limit value of the superstructure can be defined to be the same as limit state 2. In other words, the limit value can be defined as the yield strain of the steel members since the mechanical properties can be regarded as keeping maintaining reversibility.

Also, the limit value of the pile-pile can be defined to be the same as limit state 2. It is aimed to keep the state not exceeding the maximum strength point.

Regarding the limit value of the ground, as mentioned in limit state 2, it is proposed that the limit value is not provided. This is because for limit state 3 it can be estimated that the ground displacement generated to the column at the ground surface is limited within a certain range, since mainly the yield state of columns and piles is proceeding.

#### 4. Analytical Model for Design Work

Steel pier-pile integral structure is an integral structure that consists of a multipolar rigid frame structure using the flexible column supported on the partial elastic foundation, which differs from a typical bridge in terms of using a rigid footing. Thus, Beam on Nonlinear Winkler Foundation (BNWF) model shown in Fig. 7 can be admitted as an appropriate analytical model since that model can reproduce the actual behavior considering the displacement or deflection of the pier-pile due to seismic force. Additionally, in the past research [5], the analysis using the BNWF model has been performed, and the reproducibility of the dynamic and static experimental results has been mentioned. In the course of the above, BNWF model can be proposed as an adequate analytical model in this paper. Also, the model of each member based on relevant research [4] [5] [6] [7] and standards [9] [11] can be applied to the target structure with the exception of the pier pile portion which is the special part. For this reason, the modeling method for special parts of this structure, such as the elongated column member of this structure and specification of the ground spring, are proposed in the following.

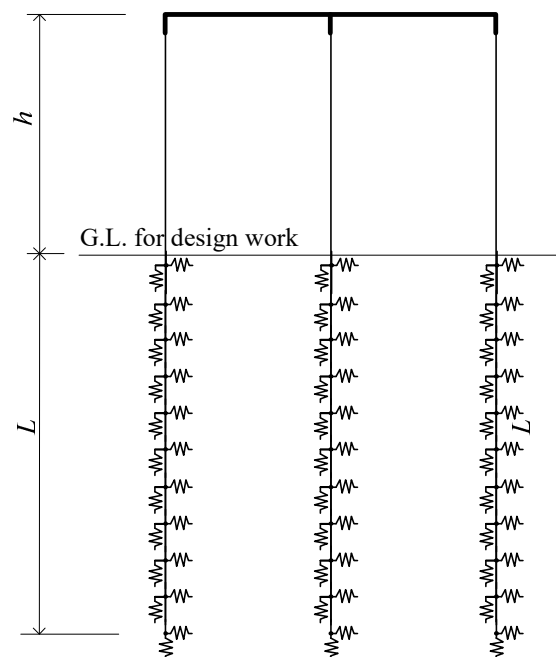


Fig.7 – Proposed analytical model “BNWF” for design work





#### 4.1 Pier and Pile

The slenderness ratio parameter  $\bar{\lambda}$  of this elongated column member can be cited from the past research [5]. In this past research, the targeted range of  $\bar{\lambda}$  is shown in equation (7).

$$0.2 \leq \bar{\lambda} = \frac{1}{\pi} \sqrt{\frac{\sigma_y l}{E r}} \leq 1.0 \quad (7)$$

Herein,  $\sigma_y$  is the yield stress of material.  $l$  = effective buckling length calculated by Eq. (8),  $r$  = cross-section secondary radius.

$$k = \frac{l_{cr}}{h} = 1\alpha_1 \left\{ 1 + \frac{1}{\alpha_3 \beta h} \tan^{-1} \left( \frac{1}{1+2\alpha_2 \beta h} \right) \right\} \quad (8)$$

Herein,  $k$  indicates the ratio of the effective buckling length  $l_{cr}$  to column height  $h$ . In detailed is shown in [4, 7].

The modeling of the pier-pile member can be applied to the  $M$ - $\phi$  model [11] shown in Fig.8. Also, since the suggested limit value is derived from experimental results [5] using same method as a method [12] of deriving the applicable range and limit value of Japan Standard V [11], it can be applied to this structure.

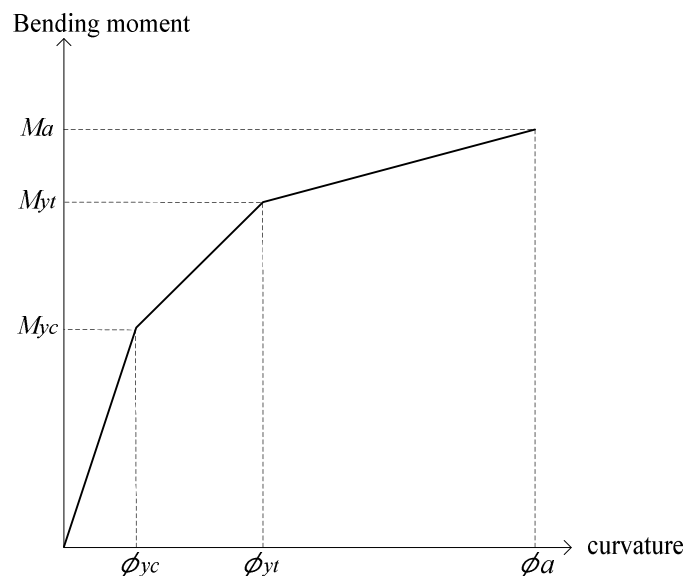


Fig.8 –  $M$ - $\phi$  model by trilinear model

Herein,  $M_a$  and  $\phi_a$  indicate the limit value of bending moment and curvature of the column accordingly.  $M_{yt}$  and  $\phi_{yt}$  indicate the bending moment and curvature at the yielded in the tension side accordingly.  $M_{yc}$  and  $\phi_{yc}$  indicate the bending moment and curvature at the yield on the compression side accordingly.

#### 4.2 Ground spring

Regarding ground spring in static analysis, the applicability of Eq. (1) to (4) has been confirmed in the past research [5]. For the spring applied to dynamic analysis for the BNWF model, a nonlinear hysteretic rule for Winkler type soil-pile interaction springs that considers loading pattern dependency has been proposed in the research [13][14] as the model showing the dynamic



characteristics of the pile foundation. This spring has been adopted as a fitting model in past research [5].

## 5. Conclusion and Next Subject

The results revealed in this section are summarized below.

- (1) The destruction form of the multipolar rigid frame structure consisting of the pier pile integral structure can be expressed by the line approximated using an exponential function, composed of the length of the pier pile and stiffness relationship of rigidity of ground and pier pile. Furthermore, when the proposed Eq. (5) is satisfied, it can be confirmed that the destruction form of this structure can remain the pier-pile preceding type which is superior in toughness.
- (2) It was shown that the limit state for the target structure can be defined into three limit states based on the results of past pertinent research. Furthermore, the idea of the limit values for each state was proposed from the engineering perspective.
- (3) Adequate analytical models which can be reproduced the actual behavior of the structures was proposed for the design work of this structure.

The following subjects are remaining in order to evaluate the reliability of the proposed analytical model to design work.

- (4) The amount (greater or smaller) of the outcome using the proposed analytical model will be evaluated through the work of trial design based on this policy. Therefore, it is necessary to conduct verification work through trial design according to various cases. Specifically, it can be compared by the analysis using the experimental model in the past research [5], with the actual section force confirmed by the experiments, and estimated cross sectional force calculated by the proposed design model in this paper.

## 6. References

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