# 5.9 Response of Pile Foundations for Fault Dislocation

#### 1) Effect of Fault and Target Bridge

As shown in Chapter 3, in the 1999 Kocaeli and Duzce, Trukey Earthquakes and the Chi Chi, Taiwan Earthquake, destructive damage of bridges occurred as a direct result of fault displacements in addition to the ground motion effects. The vertical offset reached 9 m in the Chi Chi Earthquake. Transportation facilities are likely to cross faults in the areas with active faults. Consequently, it is important to develop appropriate measures for the fault dislocation. An analysis on the effect of a reversed fault is shown here.

Analyzed is a bridge consisted of a 200 m long 5-span continuous plate girders as shown in Fig. 5.90. The soil consists of alternatives of clay and sand, and the natural period of the surface ground  $T_G$  by Eq. (1.1) is 0.37 s. Bottom of the piles reaches a gravel layer at 17 m below the ground surface. Nine 15 m long cast-in-place reinforced concrete piles with a diameter of 1.2m support a footing. A reversed fault with an angle of 45 degree crosses the bridge, perpendicular to the bridge axis, at one of the pile foundations. Although there must be an interaction between the pile foundation where the fault crosses and other pile foundations resulting from the strut action of decks, it is disregarded here for simplicity. Only a structural system consisting of a part of the superstructure, the column and the foundation is considered in the following analysis.



Fig.5.90 Pile Foundation Analyzed

In the idealization of the fault displacement, it is assumed that only the effect of fault displacement is considered with the effect of ground excitation being disregarded. It is also assumed that amplitudes of the ground dislocation along the fault plane are constant in the vertical and horizontal directions. The amount of dislocation along the fault plane is as large as 2 m.

# 2) Idealization of Pile Foundation and Fault Dislocation

Based on Eqs. (5.39)-(5.53), the pile foundation is idealized by a two dimensional discrete model consisting of a series of beams and soil springs as shown in Fig. 5.91. Since the stiffness of superstructure and the inertia force are disregarded, only the footing and piles are idealized in the model with the masses of the superstructure and the column being lumped. To idealize the piles in two dimension, nine piles are merged into three pile lines by simply adding the stiffness and strengths. They are called here as left, center and right piles.

The flexural hysteretic behavior of the piles is idealized by the Takeda degrading model. The moment vs. curvature relation is provided as shown in Fig. 5.92 (a) by the standard fiber element method. The stress vs. strain relation of confined concrete is assumed by the

Hoshikuma et al model, and the stress vs. strain relation of reinforcing bars is assumed as the elasto-plastic bilinear.



Fig.5.91 Analytical Model of Pile Foundation and Fault Dislocation



(a) Moment vs. Curvature Relation (b) Axial Force vs. Axial Displacement Fig.5.92 Model of Piles

When the piles are subjected to the ground displacement as shown in Fig. 5.91(b), the left piles above the fault plane and the right piles below the fault plane are directly subjected to the ground displacement. They are called as the front piles. However, the ultimate bearing capacity and the stiffness of the piles that are behind the front piles (the rear piles) are generally smaller due to the group pile effect. Therefore, the ultimate bearing capacity and stiffness of the rear piles are assumed a half of those of the front piles. Restoring force of soil springs are idealized as shown in Fig. 5.70 (1).

On the other hand, the vertical force vs. vertical displacement relation of the soil springs is assumed as shown in Fig. 5.70 (2). The vertical stiffness of *i*-th soil spring  $K_{Vi}$  is evaluated as

$$K_{Vi} = \alpha \frac{A_P E_P}{L} \cdot \frac{2\pi D \cdot l_{Pi} \cdot f_{Pi}}{\sum 2\pi D \cdot l_{Pi} \cdot f_{Pi}}$$
(5.60)

where  $A_P$ ,  $E_P$ , L and D: section area, elastic modulus, length and diameter of a pile,

respectively,  $\alpha$ : modification factor,  $f_{Pi}$ : skin friction, and  $l_{Pi}$ : distance between soil springs.

#### 3) Pushover Analysis

A pushover analysis is conducted for the pile foundation. The ground above the fault plane moves along the fault plane with 0.1 m interval relative to the ground under the fault plane. The maximum dislocation along the fault plane is 2 m.

Figs. 5.93, 5.94 and 5.95 show how the moment, the shear and the axial force of the piles increase as the fault dislocation along the fault plane  $\Delta_F$  increases from 0.1 m to 2 m. The yield moment  $M_y$ , the ultimate moment  $M_u$ , the ultimate curvature  $\phi_u$ , the shear capacity  $P_S$ , and the ultimate axial force capacity for tension  $R_{uc}$  of a pile are also presented in these figures for comparison. As shown in Fig. 5.93, the curvature reaches the peak values at 6.4 m and 11.4 m below the pile cap in the left and the center piles, respectively, and at the pile cap in the right piles. When the fault dislocation  $\Delta_F$  increases to 2 m, the maxim curvature of a pile reaches 2.5, 3.5 and 5.1 times the ultimate curvature  $\phi_u$  (=0.027/m) at the left, center and right piles, respectively.



Fig.5.93 Moment and Curvature Distributions



The shear force of the piles becomes maximum at the fault plane as shown in Fig. 5.94. The maximum shear force is in the range of 1.2-1.9 MN at the fault dislocation of 2 m, and they are 1.1-1.8 times the shear capacity (=1.1MN).

As shown in Fig. 5.95, tension is induced in the left piles within the ranges about 5 m above and below the fault plane. The maximum tension reaches 1.2 MN at the fault dislocation of 2 m, but it is smaller than the pile tension capacity of 2.3 MN. On the other hand, at the right piles, the compression reaches 1.3MN that is smaller than the compression capacity of 25MN.

Fig. 5.96 shows the lateral force and the lateral relative displacement  $\Delta u$  of soil springs at the fault dislocation  $\Delta_F$  equal to 0.1, 0.3, 1 and 2 m. At the 2 m fault dislocation, the soil springs yield at 5.9-12.9 m, 2.9-11.9 m and 0-9.9 m below the pile caps at the left, the center and the right piles, respectively. On the other hand, Fig. 9 shows the vertical force and the vertical relative displacement  $\Delta v$  of the soil springs. The maximum skin friction and the maximum bearing stress at the bottom of the pile evaluated Eqs. (8) and (9) are presented in Fig. 5.97 for comparison. Above the fault plane, the soil springs yield at all piles, and the vertical relative displacement  $\Delta v$  reaches 1.4 m at the fault dislocation of 2 m. It implies that the vertical displacement resulted from the fault dislocation is absorbed by the vertical relative displacement  $\Delta v$ . Since the skin friction of the piles is larger at below the fault plane than above the fault plane, the inelastic hysteresis of the soil springs occurs at the soil springs above the fault plane.



Fig.5.96 Distributions of Lateral Force and Lateral Relative Displacement

Fig. 5.98 summarizes the progress of failure of the piles. At the fault dislocation of 0.1 m, the flexural yield occurs at five locations. When the fault dislocation reaches 0.3 m, the curvature at the pile cap reaches the ultimate flexural curvature  $\phi_u$  at the right piles. At the fault dislocation of 2 m, the flexural moment reaches the moment capacity  $M_u$  at six locations, and the shear reaches the shear capacity  $P_s$  at five locations. It is apparent that the piles will suffer extensive damage when the fault dislocation reaches 2 m.



Fig.5.97 Distributions of Vertical Force and Vertical Relative Displacement



Fig.5.98 Progress of Failures

# 5.10 Seismic Response of Arch bridges

Arch bridges show quite unique structural response since the longitudinal and vertical modes are coupled. The mode coupling results in a large vertical structural response in arch ribs, which in turn develops a large variation of axial force in the arch ribs. Such a characteristic is described below for a 160 m long reinforced concrete arch bridge as shown in Fig. 5.99 (Kawashima and Mizoguchi 2000).



Fig. 5.99 Arch Bridge Analyzed

The arch bridge has a rise of 27 m. Since the arch span vs. rise ratio is 1/5.6, it is a standard configuration as an arch bridge. Both ends of the reinforced concrete deck are supported by steel movable bearings in the longitudinal direction. The arch rib and the deck are connected together at the arch crown. Fig. 5.100 shows the section of arch rib at the springing. It is a box section. At both springings, 60 deformed bars with 22 mm diameter (D22) are placed at 150 mm spacing along both inner and outer faces of the upper and lower flanges. Twenty two D22 bars are placed at 150 mm spacing along both faces of the web. As a consequence, the longitudinal reinforcement ratio is 1.0-1.5%. D19 ties are placed at 150 mm spacing. D19 cross ties are also provided at 150 mm spacing in the flanges and webs. Both ties and cross ties are anchored by 90 degree bent hooks in the covering concrete.

The arch bridge is idealized by a discrete analytical model as shown in Fig. 5.101. The Takeda degrading model is used for the arch rib. The flexural moment vs. axial force interaction is disregarded in this analysis. Although the yield moment depends on the axial force, it is assumed in this analysis that it is constant, and this value is evaluated for the axial force of the arch rib induced by the dead load. Fig. 5.102 shows the natural mode shapes for major modes. The 1st and the 3rd modes (natural period is 2 s and 0.612 s, respectively) are anti-symmetric horizontal modes, while the 2nd and the 4th modes (natural period is 1.07 s and 0.447 s, respectively) are symmetric vertical modes. It is noted that in each mode, both horizontal and vertical amplitudes exist. For example, in the 1st mode, there exists not only the horizontal amplitude but also large vertical amplitude. This is called that the horizontal and vertical modes are coupled.





Fig. 5.102 Mode Shapes

Static response of the arch bridge is described first. Fig. 5.103 shows the axial force and the bending moment of the arch rib resulted from the static dead loads. The axial force and the bending moment due to load combinations of "dead load + active load" and (dead load + active load + thermal load," which are predominant in design, are presented here. As a consequence, the design axial force and the design bending moment are determined as shown in Fig. 5.103. Approximately 90% of the axial force is induced by the dead load. In the previous design, a seismic load corresponding to the static coefficient of 0.18 was used. Since the allowable stress was extended 1.5 times for a load combination of "dead load + seismic effect," none of the sections of the arch rib requires modification due to the seismic effect. On the other hand, the bending moment induced by the dead load is only 20% of the design value, and the rest is contributed by the active load and the thermal effect. However as is the case of axial force, no modification is required for the arch rib section by the seismic effect.

Rock at the site is idealized by two dimensional equivalent linear model (Lysmer 1975). The computed rock response is shown in Fig. 5.104. Since the shear wave velocity of the rock in the left valley is lower than that in the right valley, amplification of acceleration responses is higher in the left valley. Computed rock accelerations at both springings are used as an input ground motion to the arch bridge. To understand the basic response characteristics of the arch bridge, both uniform excitation and multiple excitation are considered. In the uniform excitation, the acceleration responses at the right valley are applied to the arch bridge at both springings.



Fig. 5.103 Axial Force and Bending Moment for Static Loads



Fig. 5.104 Response Accelerations of Rock

# 1) Uniform Excitation

First, the arch bridge is excited only in the longitudinal direction. Fig. 5.105 (a) and (b) show the peak response acceleration and displacement of the arch bridge when it is subjected to a longitudinal excitation at both springings. The maximum lateral acceleration of 0.8 g occurs at both springings, while the maximum lateral displacement of 0.15 m occurs at the 1/4 and 3/3 points in the arch rib. Over 1 g acceleration and over 0.3 m displacement occur in the vertical direction although the arch bridge is excited only in the longitudinal direction. This results from the significant mode coupling of the arch bridge between the longitudinal and vertical modes.

Fig. 5.105 (c) and (d) show the axial force and bending moment of the arch rib. By comparing Fig. 5.105 to Fig. 5.103, it is known that the axial force increases about 20% by exciting the bridge in the longitudinal direction, and this axial force exceeds the design axial force. The bending moment also exceeds the design bending moment at many locations. However, since the bending moment does not reach the yield moment, the arch rib does not yield.



Fig. 5.105 Peak Response Accelerations and Displacements of the Arch Bridge subjected to Uniform Excitation only in the Longitudinal Direction

Next, Fig. 5.106 shown responses of the arch bridge when it is subjected to both the longitudinal and vertical excitation. The lateral responses do not increase resulted from the vertical excitation, but the effect of vertical excitation on the vertical responses is not less. This effect is more clearly observed in the axial force response. At the left springing, the maximum axial force increases from 60 MN to 70 MN by applying the vertical excitation. The bending moment slightly increases by considering the vertical excitation.



Fig. 5.106 Peak Response Accelerations and Displacements of the Arch Bridge subjected to Uniform Excitation in both the Longitudinal and Vertical Directions

# 2) Multiple Excitation

Fig. 5.107 shows the responses of the arch bridge subjected to multiple excitation both in the longitudinal and vertical directions. It is noted here that a direct comparison of the response displacement cannot be made because the response displacement under the uniform and the multiple excitations represents the relative and the absolute displacements. Since the rock acceleration at the left valley is significantly larger than the response at the left valley, the acceleration response of the arch rib, in particular in the vertical direction, increases significantly by applying the vertical excitation. The important point is the significant increase of the axial force under the multiple excitation. Although the tension is only 10 MN, which is 20% of the design axial force, this is important in the evaluation of seismic performance of this bridge. s shown in Fig. 5.108, flexural yield with a moment ductility factor of 1.1 occurs at the arch rib where the arch rib and the deck is connected together.



Fig. 5.107 Peak Response Accelerations and Displacements of the Arch Bridge subjected to Multiple Excitation in Both the Longitudinal and Vertical Directions

